DESIGN CRITERIA FOR SUPPORTS AND LININGS OF UNDERGROUND STRUCTURES FOR WATER RESOURCES PROJECTS - SOME CASE STUDIES

A DISSERTATION

Submitted in partial fulfilment of the requirements for the award of the degree of

MASTER OF ENGINEERING

in

WATER RESOURCES DEVELOPMENT

By

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CANDIDATE'S DECLARATION

I hereby certify that the work which is being presented in this dissertation entitled, **"DESIGN CRITERIA FOR SUPPORTS AND LININGS OF UNDERGROUND STRUCTURES FOR WATER RESOURCES PROJECTS - SOME CASE STUDIES"** in partial fulfillment of the requirement for the award of the **MASTER OF ENGINEERING** in **Water Resources Development** submitted to the **Water Resources Development Training Centre, University of Roorkee, Roorkee (India)** is an authentic record of my original work carried out during the period from July 16, 1997 to December 19, 1997 under the supervision of Prof. Gopal Chauhan, Professor and Director, WRDTC, University of Roorkee and Dr. M.C. Goel, Formerly Professor, WRDTC and Engineer in Chief of U.P. Irrigation Department.

The matter embodied in this dissertation has not been submitted by me for the award of any other degree.

19/12/97

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Date-: December 19, 1997

Place : Roorkee

This is to certify that the above statement made by the candidate is complete and correct to the best of our knowledge.

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SYNOPSIS

Inspite of extensive research performed in the field of rock engineering, innovation in design has not proceeded as rapidly as in other engineering fields. It is interesting to note that rock bolts and shotcrete represented the last major innovations in underground excavation support technologies. While rock bolts were enormously successful since their introduction in 1940's, even today rock bolt parameters and layout are specified primarily on the basis of empirical procedures and practical experience.

In this dissertation twenty case studies of power houses and water tunnels related to water resources projects , have been taken up. The support requirements have been worked out by using Terzaghi's method, Wickham's RSR method, Bieniawski's RMR method and Barton's Q system. The minimum support pressure has been compared with the support pressure accommodated by the actual supports and observed pressures provided at site.

It can be inferred that in case of massive to moderately jointed rocks, Q system gives the minimum support pressure. For very seamy and closely jointed rocks, RMR method gives the lowest support pressures. In the case of squeezing and swelling rocks also, the RMR method give the minimum support pressures. Terzaghi's method gives the maximum support pressures for 100 percent cases of large excavations and for more than 95 percent cases of water tunnels. The observed rock pressures are either nearer or these are enveloped by calculated rock pressures by Q system.

Hence based on this study it can be inferred that Q system of rock classification and rock support design, can be safely used.

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CHAPTER 1
INTRODUCTION

1.1 GENERAL

So far in India the excavation of large underground cavities have been restricted to underground power houses. The first underground power house was taken up at Maithon for Damodar Valley Development, way back in 1953, followed by Koyna in Maharashtra. Since then, a number of underground excavations for hydropower development, are coming up. All the rocks are under stress and strain as a result of its geologic history. During excavations, restraints at the boundaries are removed or changed and the rock adjacent to the boundaries is in effect being unloaded. Immediately following the exposure of new surface, redistribution of stresses take place. With advancement of tunnel heading, the radial field stress at the new surface becomes zero and the tangential stress parallel to the surface will increase. As a result of these stress changes the rock deforms and moves towards opening, thereby creating the instability to the underground opening.

1.2 FACTORS AFFECTING STABILITY OF UNDERGROUND STRUCTURES

In planning and construction of water resources projects, the design of underground excavations brings together the geotechnical risk and economic considerations in a direct way. The specifications of supports and linings can have very significant impact on placement and design of machine hall and other underground excavations and greatly affect the cost of pressure tunnel itself. Moreover the stability of underground excavation is to be ensured by way of providing the required support system. The stability of underground excavations depends on the following factors.

- Pre and post excavation stresses.
- Size and shape of the opening.
- Orientation or the axis of the opening with respect to the maximum principal stress.
- Nature and type of rocks and their engineering properties.
- Groutability of Rock
- Depth of overburden/rock cover over the opening.
- Extent of discontinuities like faults, number of joints sets, fracture planes etc.

- Frequency of joints, their orientations, joint filling materials, and permeability characteristics.
- Presence of shear zones or overstressed zones.

1.3 DESIGN METHODS IN USE

The methods generally in use are limited to empirical and numerical methods. In the empirical methods the first engineering approach was developed by the great genius Karl Terzaghi in 1946 followed by some other semi-empirical methods such as that of Beirbaumer, Protodyakonov etc. Deere (1963) gave his concept of Rock Quality criteria on the basis of the drill core recovery of core lengths of 10 or more than 10 cm, obtained per m run of the drillhole. On that basis he designated the rock into various classes. Lateron in 1970's some developments in the field of rock engineering took place with the evolution of classifications systems like rock structure rating (RSR) by Wickham et.al (1972), rock mass rating by Bieniawski et. al. (1974) and Q system by Nick Barton et. al. (1974). These RMR and Q systems were immediately adopted for design of support by the engineers working in the field of rock mechanics.

1.4 SCOPE AND OBJECTIVE

Though the Bieniawski's Rock Mass Rating and Barton's Q systems are being used by the Geotechnical engineers for the determination of support requirements, a lot of work needs to be done in this field. Still the judgement of the site engineer is of more importance.

In the present study some Indian Water Resources Projects have been taken up. Different methods or techniques available for the evaluation of supports and linings in underground water resources project openings have been applied to find the support requirements for various types of underground structures in water resources projects. The support systems actually provided have also been listed. Support Pressures accommodated by the actually provided supports have been compared with the minimum required one. The observed instrumented pressure for seven case studies have also been compared with the anticipated rock pressures.

CHAPTER 2
DESIGN APPROACHES IN ROCK ENGINEERING

The rock support methods used in tunnels and large rock caverns vary to a large extent with the purpose of excavation and the intended working lives of these constructions. Power stations and major highway tunnels require, for instance far more safety than water tunnels, temporary tunnels and openings. In search of economy and safety, one must try to find out the support measures most appropriate for the given excavation and rock mass quality. With respect to support methods used, there is no unanimity, rather say opinions and approaches vary.

2.1 BEHAVIOUR OF ROCK MASSES

The behaviour of rock mass in an underground opening depends upon the depth and quality of rock. Varying degrees of instability can be broadly classified into the following categories.

1. Stable conditions

   Stable condition refers to rock to a state of equilibrium attained with or without assistance from support or reinforcement. Either the deformations are no longer taking place or these are decreasing with the time.

2. Gravitational failure condition

   Condition in which roof and sometimes also the walls progressively collapse by raveling or caving, such as the loosening or falling of blocks leading to the failure of rock mass.

3. Squeezing condition

   Condition in which the crown, the side walls and sometimes the invert of the excavation converge slowly and continuously by mechanisms of stress induced visco-plastic flow. There is substantial distortion of intact rock material often accompanied by creep along joints within the zone of overstressed rock mass.

4. Swelling condition

   When rocks exposed near the wall expands by physico-chemical mechanisms associated with the absorption of water by clay minerals or anhydrite. The swelling is the result of mineralogical changes whereas squeezing is caused by overstressing.

5. Bursting Condition

Rock fails by bursting when the rock ruptures explosively by propagation of fractures through previously solid rock. Stored energy is released suddenly and violently.

2.2 METHODS FOR EVALUATING ROCK SUPPORT REQUIREMENTS

The application of design methodology to rock engineering has not received as much attention as in other engineering fields. This has resulted in excessive safety factors for many civil and mining engineering works.

The design methods which are currently available in rock engineering can be categorised as follows.

A) Analytical Methods

B) Observational Methods

C) Empirical Methods

2.3 ANALYTICAL METHODS

Analytical methods utilise the stresses and deformations around excavations. They include techniques such as closed form solutions, numerical methods (finite element, finite difference, boundary element), analog simulations (electric and photoelestic) and physical modeling.

Methods to compute linear stresses include elastic closed form solutions, beam spring models and beam continuum models such as those based on the finite element method. Their merits and demerits have been reviewed by ASCE Technical Committee on Tunnel Lining Design (O'Rourke, 1984). Charts that assist in predicting liner stresses have been published by Detournay and St. John (1988). An advanced theoretical model to predict ground stresses, strains and displacements around a circular tunnel is provided by Histake et. al. (1989); it includes realistic peak and residual strength criteria and non-linear stress strain relationships. Grimstad and Barton (1988) describe use of the Universal Distinct Element Code (UDEC) with input determined on the basis of the parameters of Q-system. They claim that the numerical procedures provide a wealth of information concerning stresses, deformations, the joint displacements, which assist the designer in assessing the effect of chosen rock strength.

2.4 OBSERVATIONAL METHODS

These methods rely on actual monitoring of ground movement during excavation to detect measurable instability, and on the analysis of ground support interaction. Although considered as separate methods, observational approaches are the only way to check the results and prediction of other methods.

2.5 EMPIRICAL METHODS

These methods are frequently used in rock engineering practice. They assess the stability of underground excavations by use of statistical analysis of observations. Engineering rock mass classifications are the best known empirical approaches for assessing the stability of excavations in rock.

All the above mentioned methods of designing the supports for underground excavations require geological input, engineering properties of rock and considerations of statutory safety regulations. Empirical and semi-empirical design methods are discussed in detail here.

Empirical design solves the problem of our limited experience by making available the accumulated experience of others. It requires the following three steps beyond simple judgement.

- description of ground quality by a quantitative classification system.
- To provide a universal language whereby the global experience gained working in ground of many different qualities can be related to future projects.
- Description of ground performance by a formalised quantitative system, which defines parameters as unsupported time and support requirements and correlation of ground quality to performance by comparison of results from a variety of projects over full spectrum of ground condition.

Purely empirical methods are those that predict tunnel support requirements from knowledge of ground conditions using a pre-established correlation. Semi-empirical methods predict rock loads empirically as an intermediate step and then go from the rock loads to support requirements via simple theoretical model of rock behaviour.

2.5.1 Deere's Method

Deere et. al. (1963, 1969) gave criteria for rock support evaluations in rock tunnels based on Rock Quality Designation (RQD), the ratio of sum of sound rock pieces which are greater or equal to 10 cm in length to the core run. For RQD values greater than 60 they recommended support consisting of rock bolts, mesh and strapping whereas for RQD values less than 40, steel sets or ribs were specified. RQD values of between 40 and 60 called for linear interpolation of support requirements. The RQD method is of interest as it can be used for the preliminary choice of support, as well as a constitutive parameter for more elaborate systems. Table 2.1 can be referred for Deere's support recommendations for tunnels.

2.5.2 Rock Structure Rating (RSR) System

The Rock Structure Rating (RSR) system developed in USA by Wickham et. al. (1972), was based on the supports provided for the tunnels supported mainly by the steel sets and designed using the Terzaghi's tunnel support classifications. RSR is determined by adding three weighted parameters. Parameter A, represents geological conditions (Rock type, rock quality, degree of weathering and geological structure); parameter B, depends on the joint spacings and orientations with respect to the tunnel axis whereas parameter C, rates the ground water inflow and conditions of the joints. Tables and charts are used to determine these parameters. Table 2.2 to 2.4 show the rock structure rating for various grades of rocks.

The support load (KPa) can be calculated by using the following relation :

$$W_r = 0.26(B + H)\left[\frac{8880}{RSR + 30} - 80\right]$$

Where

B = Width of the cavern

H = Height of cavern

The rock bolts can be designed by using the following relations:

Spacing of 25.40 mm bolts in m $= 0.3048\ (24/W_r)^{1/2}$

Spacing of 19.04 mm bolts in m $= 0.3048\ (13.5/W_r)^{1/2}$

Shotcrete thickness in mm $= 25.42\ (1 + W_r/1.25)$

TABLE 2.1 : SUPPORT RECOMMENDATIONS FOR TUNNELS IN ROCK (6-12 M IN DIAMETER), DEERE ET.AL.,1969

Rock Quality	Tunnelling Method	Alternate Support Systems		
		Steel Sets[a]	Rock Bolts[b]	Shotcrete[c]
Excellent[a] RQD>90	A. Boring Machine	None to occasional light set. Rock load 0.0 to 0.2 B[d]	None to Occasional	None to occasional local application
	B. Conventional	None to occasional light set. Rock load 0.0 to 0.3 B	None to Occasional	None to occasional local application 50 to 75 mm
Good[a] 75<RQD<90	A. Boring Machine	Occasional light sets to pattern on 1.5 to 1.8 m center. Rock load 0.0 to 0.4 B	Occasional to pattern on 1.5 to 1.8 m center	None to occasional local application 50 to 75 mm
	B. Conventional	Light sets to 1.5 to 1.8 m center. Rock load 0.3 to 0.6 B	Pattern 1.5 to 1.8 m center	Occasional local application 50 to 75 mm
Fair 50<RQD<75	A. Boring Machine	Light to medium sets, 1.5 to 1.8 m center. Rock load 0.4 to 1.0 B	Pattern 1.5 to 1.8 m center	50 to 100 mm on crown
	B. Conventional	Light to medium sets, 1.2 to 1.5 m center. Rock load 0.6 to 1.3 B	Pattern 1.5 to 1.8 m center	100 mm or more in crown and sides
Poor[b] 25<RQD<50	A. Boring Machine	Medium circular sets on 0.9 to 1.2 m center. Rock load 1.0 to 1.6 B	Pattern 1.5 to 1.8 m center	100 mm to 150 mm on crown and sides. Combine with bolts.
	B. Conventional	Medium to heavy sets on 0.6 to 1.2 m center. Rock load 1.3 to 2.0 B	Pattern 1.5 to 1.8 m center	150 or more on crown and sides. Combine with bolts
Very Poor RQD<25 (Excluding squeezing or swelling grounds)	A. Boring Machine	Medium to heavy circular sets on 0.6 m center. Rock load 1.6 to 2.2 B	Pattern 0.6 to 1.2 m center	50 mm or more on whole section. Combine with medium sets
	B. Conventional	Heavy circular sets on 0.6 m center. Rock load 2.0 to 2.8 B	Pattern 0.9 m center	150 mm or more on whole section. Combine with medium to heavy sets
Very poor (Squeezing or swelling)	A. boring Machine	Very heavy circular sets on 0.6 m center. Rock load upto 75 m	Pattern 0.6 to 0.9 m	150 mm or more on whole section. Combine with heavy sets
	B. Conventional	Very heavy circular sets on 2 foot center. Rock load upto 75 m	Pattern 0.6 0.9 m center	150 mm or more on whole section. Combine with heavy sets.

[a] In good and excellent rock, the support requirement in general be minimal but will be dependent on joint geometry, tunnel diameter, and relative orientation of joints and tunnel. [b] Lagging requirements will usually be zero in excellent rock and will range from upto 25% in good rock to 100% in very poor rock. [c] Mesh requirements usually will be zero in excellent rock and will range from occasional mesh (or straps) in good rock to 100% mesh in very poor rock. [d] B = tunnel width in m.

TABLE 2.2 : ROCK STRUCTURE RATING A, GEOLOGICAL CONDITION (AFTER WICKHAM & TIEDMANN, 1974)

Basic Rock Type*	Massive RQD > 75	Slightly folded or faulted RQD 50-75	Moderately folded or faulted RQD 25-50	Intensely folded or faulted RQD < 25
Type I	30	22	15	9
Type II	27	20	13	8
Type III	24	18	12	7
Type IV	17	15	10	6

*Basic Rock Type

Basic Rock	Rock Condition			
	Hard	Medium	Soft	Decomposed
Igneous	I	II	III	IV
Metamorphic	I	II	III	IV
Sedimentary	II	III	IV	IV

2.5.3 Rock Mass Rating (RMR) System

Rock Mass Rating (RMR) System was developed in South Africa by Bieniawski (1974 and 1979). Bieniawski's classification system considers the following six properties.

- Uniaxial Compressive Strength (UCS)
- Rock Quality Designation (RQD)
- Spacing of Discontinuities
- Condition of Discontinuities
- Ground Water Conditions
- Orientation of Discontinuities

RMR is the sum of all the above ratings as specified by Bieniawski. These ratings are as shown in Table 2.5. Tables allow determination of parameters as a guide to the solution of excavation and support procedures for openings. The average stand up time for the opening can be estimated using the Fig. 2.1. The support pressures (P_r) can be found by using the following empirical formula as proposed by Unal (1983).

TABLE 2.3 : ROCK STRUCTURE RATING B, JOINT SPACING CONDITION (AFTER WICKHAM & TIEDMANN, 1974)

Average Joint Spacing	Strike Perpendicular to Axis					Strike parallel to Axis		
	Direction of Drive					Direction of Drive		
	Both	With Dip		Against Dip		Both		
	Dip of Prominent Joints					Dip of Prominent Joints		
	Flat	Dipping	Vertical	dipping	Vertical	Flat	Dipping	Vertical
Very closely Jointed < 2"	9	11	13	10	12	9	9	7
Closely Jointed 2"-6"	13	16	19	15	17	14	14	11
Moderately Jointed 6"-1'	23	24	28	19	22	23	23	19
Moderate to Blocky 1'-2'	30	32	36	25	28	30	28	24
Blocky to Massive 2'-4'	36	38	40	33	35	36	34	28
Massive > 4'	40	43	45	37	40	40	38	34

Flat 0° - 20° Dipping 20° - 50° Vertical 50° - 90°

TABLE 2.4 : ROCK STRUCTURE RATING C GROUND WATER JOINT CONDITION (WICKHAM & TIEDMANN, 1974)

Anticipated Water Inflow	Sum of Parameters A & B					
	13-44			45-75		
	Joint Condition					
Gallons/min/1 m	Good	Fair	Poor	Good	Fair	Poor
None	22	18	12	25	22	18
Slight < 200 gpm	19	15	9	23	19	14
Moderate 200-1000 gpm	15	11	7	21	16	12
Heavy > 1000 gpm	10	8	6	18	14	10

Good - Tight or cemented Fair - Slightly weathered Poor - Severely weathered or open

TABLE 2.5 : RMR/CSIR GEOMECHANICS CLASSIFICATION OF JOINTED ROCK MASSES (BIENIAWSKI, 1979)

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

	PARAMETER		RANGES OF VALUES				
1	Strength of intact rock material	Point load strength index	> 8 MPa	4 - 8 MPa	2 - 4 MPa	1 - 2 MPa	For this low range - uniaxial compressive test is preferred
		Uniaxial compressive strength	> 200 MPa	100 - 200 MPa	50 - 100 MPa	25 - 50 MPa	10-25 MPa / 3-10 MPa / 1-3 MPa
	Rating		15	12	7	4	2 / 1 / 0
2	Drill core quality RQD		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%
	Rating		20	17	13	8	3
3	Spacing of joints		> 3 m	1 - 3 m	0.3 - 1 m	50 - 300 mm	< 50 mm
	Rating		30	25	20	10	5
4	Condition of joints		Very rough surfaces Not continuous No separation Hard joint wall rock	Slightly rough surfaces Separation < 1 mm Hard joint wall rock	Slightly rough surfaces Separation < 1 mm Soft joint wall rock	Slickensided surfaces or Gouge < 5 mm thick or Joints open 1 - 5 mm Continuous joints	Soft gouge > 5 mm thick or Joints open > 5 mm Continuous joints
	Rating		25	20	12	6	0
5	Ground water	Inflow per 10 m tunnel length	None		< 25 litres/min	25 - 125 litres/min	> 125 litres/min
		OR Ratio $\frac{\text{joint water pressure}}{\text{major principal stress}}$	0		0.0 - 0.2	0.2 - 0.5	> 0.5
		OR General conditions	Completely dry		Moist only (interstitial water)	Water under moderate pressure	Severe water problems
	Rating		10		7	4	0

B. RATING ADJUSTMENT FOR JOINT ORIENTATIONS

Strike and dip orientations of joints		Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
Ratings	Tunnels	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100 — 81	80 — 61	60 — 41	40 — 21	< 20
Class No	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK MASS CLASSES

Class No	I	II	III	IV	V
Average stand-up time	10 years for 5 m span	6 months for 4 m span	1 week for 3 m span	5 hours for 1.5 m span	10 min for 0.5 m span
Cohesion of the rock mass	> 300 kPa	200 - 300 kPa	150 - 200 kPa	100 - 150 kPa	< 100 kPa
Friction angle of the rock mass	> 45°	40° - 45°	35° - 40°	30° - 35°	< 30°

TABLE 6 - THE EFFECT OF JOINT STRIKE AND DIP ORIENTATIONS IN TUNNELLING

Strike perpendicular to tunnel axis				Strike parallel to tunnel axis		Dip 0° - 20° irrespective of strike
Drive with dip		Drive against dip				
Dip 45° - 90°	Dip 20° - 45°	Dip 45° - 90°	Dip 20° - 45°	Dip 45° - 90°	Dip 20° - 45°	
Very favourable	Favourable	Fair	Unfavourable	Very unfavourable	Fair	Unfavourable

$$P_r = \frac{100 - RMR}{100} \gamma B$$

Where,

RMR = Rock Mass Rating

γ = rock density

B = width of opening

Bieniawski also proposed a relation between RMR and Q values as:

RMR = 9 log Q + 44

2.5.4 Size Strength System

The size strength system although used as a general purpose classification was developed in 1973 mainly to predict New Austrian Method of Tunnelling (NATM) support requirements in tunnels (Franklin, 1976). Fig. 2.2 gives the predicted requirements for shotcrete, rock bolts and rib as a function of the degree of support number obtained from the size strength diagram.

2.5.5 Q System

Q system was developed at Norwegian Geotechnical Institute (NGI) Norway by Barton et. al. (1974). The original Q system based on 212 case studies was updated by Grimstad et. al. (1993) on the basis of more than 1000 case records. The Q chart illustrating the supports to the underground openings has also the option to apply wet shotcreting, fibre reinforced shotcrete, and rock bolts. The fibre reinforced concrete provides flexibility and design can be modified as tunnelling progresses. Six rock mass parameters have been considered for the evaluation of Q.

Block Sizes

1. Rock Quality Designation (RQD)
2. Joint Set Number (J_n)

Shear Strength

3. Joint Roughness Number (J_r)
4. Joint Alteration Number (J_a)

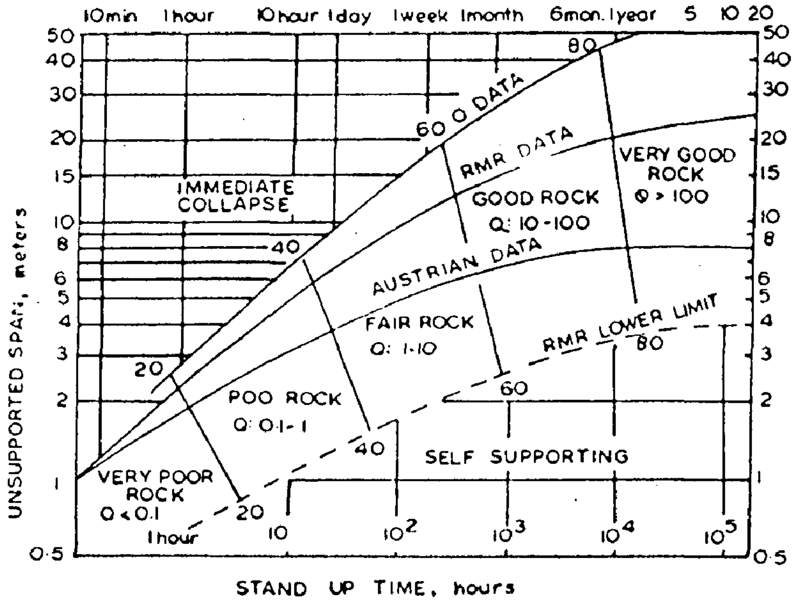


FIG 2.1: COMPARISON BETWEEN STAND-UP TIMES FOR UNSUPPORTED EXCAVATION SPANS PREDICTED BY THE Q SYSTEM, RMR & AUSTRIAN ROCK MASS CLASSIFICATION SYSTEM. RATINGS ARE FOR THE GEOMECHANICS CLASSIFICATION (RMR)

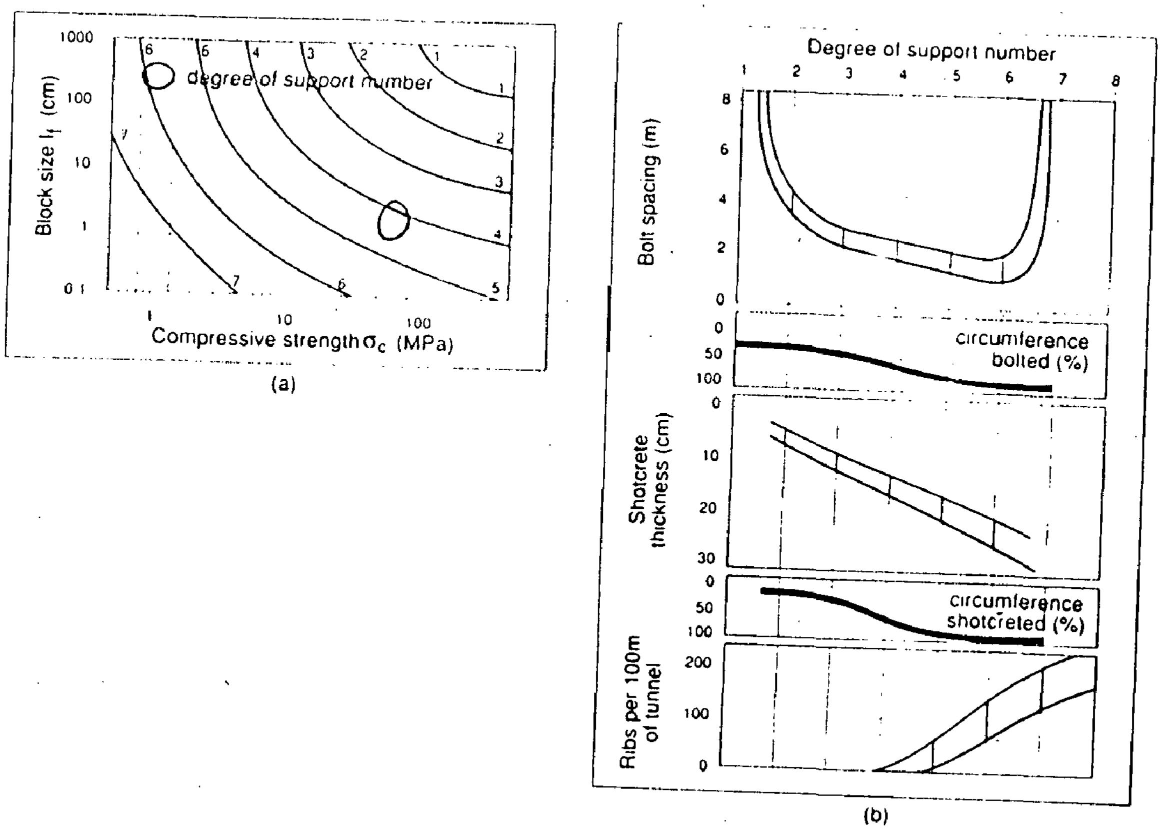


FIG. 2.2 : RELATIONSHIP BETWEEN DEGREE OF UPPORT NUMBER AND REQUIREMENTS FOR NATM SUPPORT (A) SIZE STRENGTH CLASSIFICATION CONTOURED TO SHOW DEGREE OF SUPPORT NUMBER (B) PRIMARY SUPPORT REQUIREMENT AS A FUNCTION OF ROCK MASS QUALITY (FRANKLIN 1976)

Active stresses

5. Joint Water Number (J_w)

6. Stress reduction Factor (SRF)

Table 2.6 gives the numerical values of each of the above parameters. Fig. 2.3 can be used to workout the support needs. Tunnelling quality is expressed as the product of ratios of pairs of above parameters as follows :

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

The values of Q ranges from 0.001 to 1000 as per modified Q charts. Depending upon the value of Q and the ratio of excavation to ESR (Excavation Support Ratio) the support system is determined from the charts. The value of ESR may be taken as 1.0 for underground power stations, 1.0 to 1.3 for road tunnels and 1.6 for water tunnels. Q system gives a better forecast of the support quantities as compared to other methods. The support pressure can be calculated using the following equations.

$$P_{roof} = \frac{2}{J_r} Q^{-1/3} \quad \text{for three or more set of joints}$$

$$P_{roof} = \frac{2 J_n^{1/2} Q^{-1/3}}{3 J_r} \quad \text{for less than three joint sets}$$

where P_{roof} = Roof Support Pressure

J_r = Joint Roughness

J_n = Joint Number

Use Q' instead of Q for wall support pressure.

where $Q' = 5Q$ for $Q > 10$

$= 2.5\,Q$ for $0.1 < Q < 10$

$= Q$ for $Q < 0.1$

TABLE 2.6 : RATING GUIDE FOR Q SYSTEM (BARTON et. al.1974)

Parameter	Item and Description	Value
J_n	**Number of Sets of Discontinuities**	
	Massive	0.5
	One set	2.0
	One set plus random	3.0
	Two sets	4.0
	Two sets plus random	6.0
	Three sets	9.0
	Three sets plus random	12.0
	Four or more sets	15.0
	Crushed rock	20.0
J_r	**Roughness of Discontinuities**	
	Noncontinuous joints	4.0
	Rough and wavy	3.0
	Smooth and wavy	2.0
	Rough and planar	1.5
	Smooth and planar	1.0
	Slick and planar	0.5
	Filled discontinuities	1.0
J_a	Filling and Wall-Rock Alteration, Essentially Unfilled	
	Healed joints	0.75
	Staining only, no alteration	1.0
	Slightly altered joint walls	2.0
	Silty or sandy coatings	3.0
	Clay coatings	4.0
	Filling and Wall-Rock Alteration, Filled Joint	
	Sand or crushed rock filling	4.0
	Stiff clay filling less than 5 mm thick	6.0
	Soft clay filling less than 5 mm thick	8.0
	Swelling clay filling less than 5 mm thick	12.0
	Stiff clay filling more than 5 mm thick	10.0
	Soft clay filling more than 5 mm thick	15.0
	Swelling clay filling more than 5 mm thick	20.0
J_w	**Water Conditions**	
	Dry, or inflow < 5 litres/min. locally	1.0
	Medium water inflow	0.66
	Large inflow, unfilled joints	0.5
	Large inflow, filled joints with washout	0.33
	Large inflow, filled joints, high transient inflow	0.2 to 0.1
	Large inflow, filled joints, high continuous inflow	0.1 to 0.05
SRF	**Stress Reduction Class**	
	Loose rock with clay-filled discontinuities	10.0
	Loose rock with open discontinuities	5.0
	Shallow depth (50 m or less) rock with clay-filled discontinuities	2.5
	Rock with tight unfilled discontinuities, medium stress	1.0

ROCK QUALITY AS EXPRESSED BY THE Q-SYSTEM.

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

RQD = degree of jointing

J_n = number of joint sets

J_r = joint roughness

J_a = joint alteration or filling

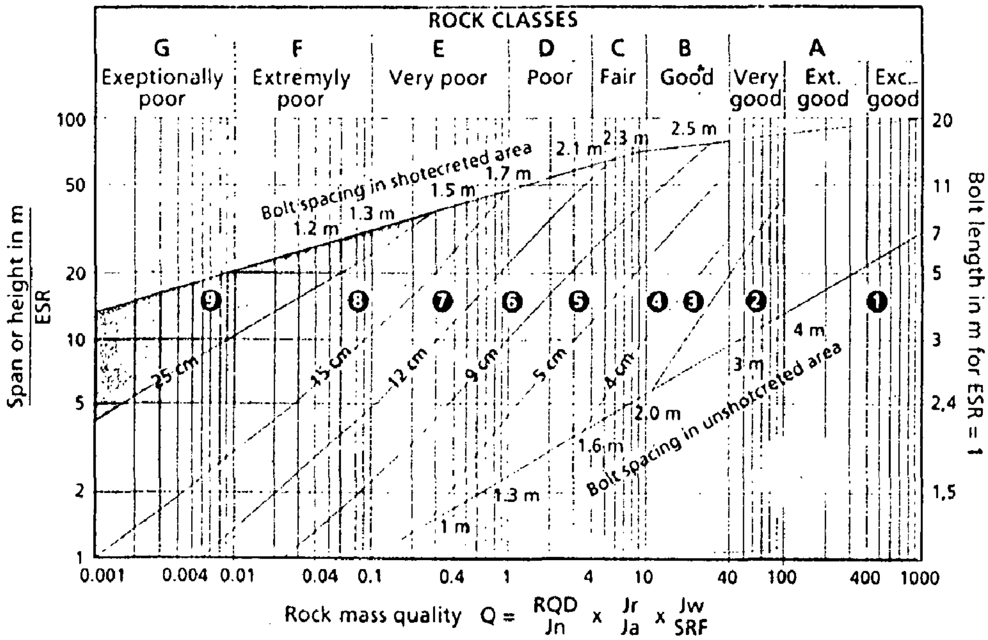
J_w = joint water leakage or pressure

SRF = Rock stress conditions

$\frac{RQD}{J_n}$ is a measure of block size

$\frac{J_r}{J_a}$ is a measure of inter-block friction angle

$\frac{J_w}{SRF}$ is a measure of active stresses



The diagram above shows the various classes of rock mass qualities, each requiring different types of rock support. The thickness of the shotcrete applied depends on the "quality" of the rock. Poorer rock mass quality requires thicker layers of shotcrete in addition to rock bolts.

REINFORCEMENT CATEGORIES:

1. Unsupported
2. Spot bolting, **sb**
3. Systematic bolting, **B**
4. Systematic bolting (and unreinforced shotcrete, 4–10 cm), **B(+S)**
5. Fiber reinforced shotcrete and bolting, 5–9 cm, **Sfr+B**
6. Fiber reinforced shotcrete and bolting, 9–12 cm, **Sfr+B**
7. Fiber reinforced shotcrete and bolting, 12–15 cm, **Sfr+B**
8. Fiber reinforced shotcrete, > 15 cm, reinforced ribs of shotcrete and bolting, **Sfr, RRS+B**
9. Cast concrete lining, **CCA**

FIG. 2.3 : Q SYSTEM OF ROCK CLASSIFICATION AND SUPPORT NEEDS (GRIMSTAD AND BARTON, 1993)

Length and spacing of Rock bolt and cable anchors can be found by using the following relationships :

$L_{roof} = 2 + 0.15*B/ESR$ for rock bolts in roof

$L_{roof} = 0.4*B/ESR$ for cable anchors in roof

$S = (0.001*C/P_{roof})^{0.5}$ for spacing of anchors in roof

$L_{wall} = 2 + 0.15*H/ESR$ for rock bolts in walls

$L_{wall} = 0.35*B/ESR$ for cable anchors in walls

$S = (0.001*C/P_{wall})^{0.5}$ for spacing of anchors in walls

Where P_{wall} = Wall support Pressure

B & H = Width and height of opening respectively

C = Load exceeding yield strength of rock bolts.

2.6 Semi Empirical Methods

Semi-empirical or Rock load methods make use of rock mass classification to predict rock loads acting on the tunnel support, allowing design of supports to resist these loads. Separate predictions are often made for vertical and horizontal load components.

The first attempts at estimating the rock loads were made by Ritter in 1879 and by Kommerell and Bierbaumer in early twentieth century (Steinner et. al., 1980). Better known are the predictions of Karl Terzaghi (1946) who as well as being a pioneer of soil mechanics, was a major contributor to rock engineering. The rock load calculation methods are described as below.

2.6.1 Terzaghi's Method

A liner has to support the entire weight of the overlying rock and soil only in the extreme case of shallow tunnel where the rock contains smooth vertical joints and where a little or no horizontal stress acts to enhance friction. Stresses are redistributed around opening by dilation and mobilisation of strength along the joints in a mechanisms known as arching. The lining has to support only these stresses not carried by rock arch.

The rock burden can be visualised as the weight of the potential rock fall bounded by the arched rock above the tunnel crown below. Dimensions and rock loads that are related to type of rock, Jointing and width and diameter of the opening. Terzaghi expresses loads acting on the liner in terms of the opening width `B', height `H' and rock mass characteristics. Table 2.7 shows the Terzaghi's classification of rocks.

2.6.2 Cording's Method

Terzaghi's method has been revised by Cording el al. (1971) to suit better for modern support systems for use in large caverns. Cording's method compares support measures and monitored displacements in a number of large caverns, and gives typical crown and sidewall support pressures applied by tensioned anchors and bolts) as functions of width and height of opening. As the size of the opening increases, the support pressures required to maintain stability also increase, and large bolts and tensioned anchors are needed.

Bolts lengths in anchored crown typically range from 0.2 to 0.4 times the cavern width B. In planner walls, bolt lengths range from 0.1 to 0.5 times the cavern height H. The following empirical relationships apply for support pressures in the crown and side walls, expressed as a function of rock unit weight and cavern height and width.

$$P_v = n * B * \gamma$$
$$P_h = m * B * \gamma$$

P_v and P_h are the crown and side wall pressures, γ is the unit weight of rock, n & m are empirically determined coefficients and varies with rock quality.

Value of n ranges from 0.1 to 0.3

Value of m ranges from 0.05 to 0.15

2.6.3 Triangular Method

In this method it is assumed that the rock loads would correspond to the weight of the rock confined within a triangle whose sides are sloping at an angle ($45^\circ - \phi$) with the vertical. This method is analogous to the design of lintels in buildings and is used for a very rough estimate of rock load (Fig. 2.4).

TABLE 2.7 : ROCK LOAD CLASSIFICATION PREDICTION OF ROCK LOADS ON STEEL SETS AND LAGGING (TERZAGHI ET. AL. 1946)

S.No	Rock condition	Rock Load	Remarks
1	Hard and Intact	Zero	Light lining, required only if spalling or popping occurs.
2	Hard, stratified, or schistose	0 - 0.25 B	Light Support
3	Massive, Moderately Jointed	0 - 0.50 B	Load may change critically from point to point
4	Moderately Blocky and seamy	0.25B - 0.35 (B+H)	No side pressure
5	Very Blocky and Seamy	0.35 - 1.10 (B+H)	Little or no side pressure
6	Completely crushed but chemically intact	1.10 (B+H)	Considerable side pressure - required support for lower end of ribs, or circular ribs.
7	Squeezing rock, moderate depth	1.10 - 2.10 (B+H)	Heavy side pressure -invert struts required, circular ribs recommended
8	Squeezing rock, great depth	2.10-4.50 (B+H)	Heavy side pressure - invert struts required, circular ribs recommended
9	Swelling rocks	Up to 76 m irrespective of (B+H)	Circular ribs required - in extreme cases use yielding supports

B = Width of Opening H = Height of Opening

Note : The table relates to saturated rocks,; load values for cases 4 through 6 can be halved if the tunnel is permanently above the water table.

2.6.4 Bierbaumer's Method

This theory was developed during the construction of the great Alpine tunnels. According to this theory the tunnel is acted upon by a parabola of height, $h = \alpha H$, where H is the height of overburden above the crown of the excavated cavity (Fig. 2.5).

For determining the value of reduction coefficient a, it is assumed that upon excavation of the tunnel, the rock material tends to slide along rupture planes inclined at $(45^\circ + \phi/2)$, f being the angle of internal friction. The base width B of the parabola of rock load is then computed by using the formula:

$$B = b + 2m (\tan 45^\circ - \phi/2)$$

where, b = excavated width of the cavity

m = excavated depth of the tunnel

$\alpha = 1 - \tan f \tan^2 (45^\circ - \phi/2)H/B$

The reduction coefficient has two limiting values:

a) For very small overburden depths; $\alpha = 1$

b) For very large overburden, when $H \geq 5B$, the magnitude is no longer affected by depth and becomes:

$\alpha = \tan^4 (45 - f/2)$

The maximum vertical pressure coming on the roof of the tunnel could then be assumed as, $p = w_r h$, where, w_r = bulk density of rock.

The Beirbaumer's theory usually gives very high values of rock loads. This takes into account the effect depth of rock cover, but the correctness of Bierbaumer's formulae has not been verified in practice. Best results as per this theory were however obtained for cavities excavated at great depths in materials displaying high internal friction and shear strength.

2.6.5 Protodyakonov's Method

Protodyakonov's theory is similarly founded on the determination of natural arching in the rock. The theory which has gained wide popularity in practice following favourable experiences in Soviet Tunnels and underground construction, is very useful within certain limits.

Protodyakonov assumes the development of arch above the cavity of which the equilibrium is not ensured, unless the stresses along the line AOB in Fig. 2.6 are purely compressive and are not associated with bending. The arch produced under this assumption will follow a parabolic line with goodman approximation.

The equation of the parabola ; $Y = 2X^2/(b*f)$

Where f is the strength factor of the material.

The height of the load carrying arch; $h = b/2 \tan\phi$

The area of the parabola = 2bh/3

Hence the load per unit length, $p = 2\gamma h/3$

or total load = $\gamma b^2/(3 \tan\phi)$

His theory was tested by model experiments, which revealed that, with the exception of small overburden depth, pressures are not affected by the depth at which the tunnel is located. These experiments involved granular materials but the theory was applied to cohesive soil as well by using an appropriate value for the coefficient, f, the strength factor (Table 2.8).

In rock, $f = \sigma_K/100$

Where σ_K = cube strength of rock.

When selecting the value for the strength coefficient, the condition as the type of rock must also be taken into account.

The theory has been found to yield satisfactory results at depths from $b/(2 \tan\phi)$ to $b/\tan\phi$.

2.6.6 Eszto's Method

The effect of tunnel width is also taken into consideration in the rock pressure theory developed by Eszto on the basic observation made in mining that excavation is followed by development of rupture surfaces outcropping to the ground surface. The rupture surfaces became gradually steeper as fissures appearing at the ground surface have been observed to start almost vertically, their inclination decreasing with depth. Rupture failure, thus takes place along a curved surface rather along a plane and the profile of this surface is according to Eszto's curve of

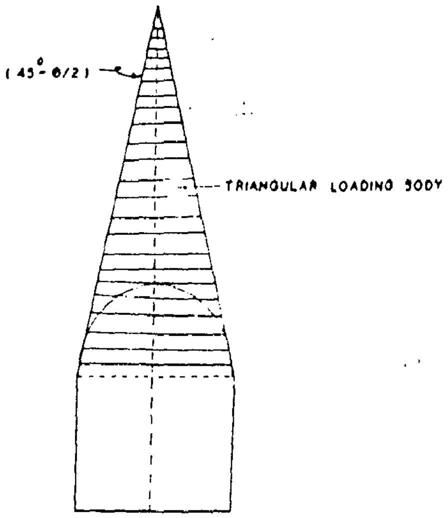


FIG. 2.4 : TRIANGULAR LOADING METHOD

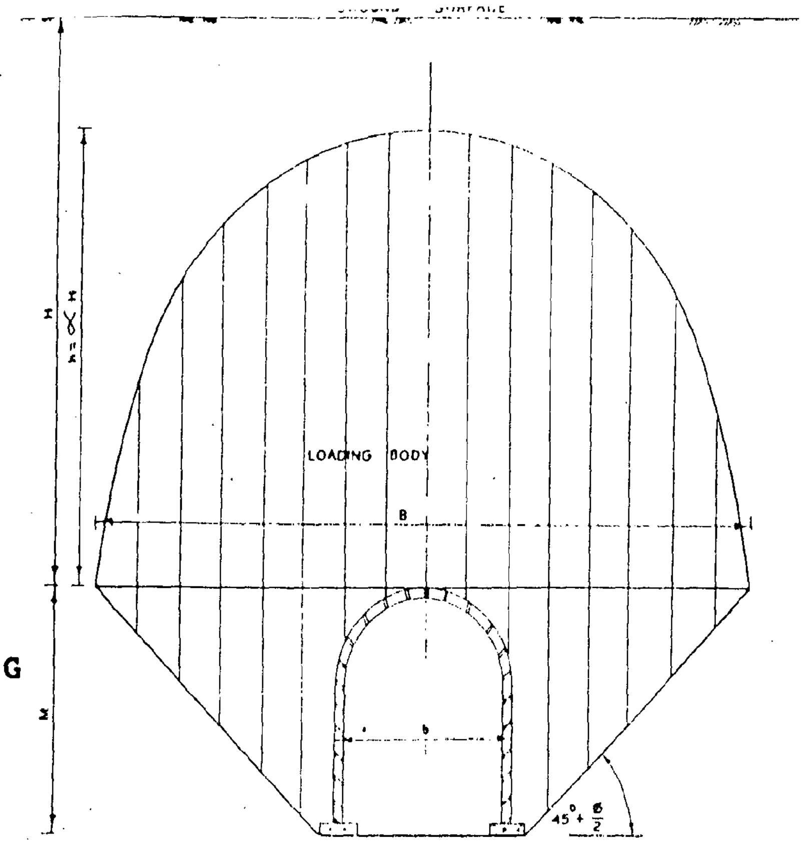


FIG 2.5 : COMPUTATION OF ROCK LOADS BY BIERBAUMER'S METHOD

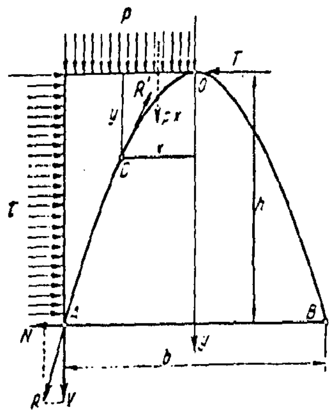


FIG. 2.6 : ANNOTATIONS AND ASSUMPTIONS OF PROTODYAKONOV'S THEORY

TABLE 2.8 : STRENGTH FACTORS FOR PROTODYAKONOV'S THEORY

Category	Strength	Denotation of rock/Soil	Unit Wt. kg/cm^2	Crushing strength, kg/cm^2	Strength factor, f
I	Highest	Solid, dense quartzite, basalt & other solid rocks of exceptionally high strength	2800-3000	2000	20
II	Very High	Solid granite, quartzite porphry, silica shale, highly resistive sandstones & limestones	2600-2700	1500	15
III	High	Granite & alike, very resistive sand & limestones, quartz, solid conglomerates	2500-2600	1000	15
IIIa	High	Limestone, weathered granite, solid sandstone, marble	2500	800	8
IV	Moderately strong	Normal sandstone	2400	600	6
IVa	Moderately strong	Sandstone shales	2300	500	5
V	Medium	Clayshales, sand & limestones of smaller resistance, loose conglomerates	2400-2600	400	4
Va	Medium	Various shales & slates, dense marble	2400-2800	300	3
VI	Moderately loose	Loose shale & very loose limestones, gypsum, frozen ground, common marl, blocky sandstones, cemented gravel, hard clay	2200-2600	200-150	2
VIa	Moderately loose	Gravelly ground, blocky & fissured shale, compressed boulders, gravel & hard clay	2200-2400	--	1.5
VII	Loose	Dense clay, cohesive ballast, clayey ground	2000-2200	--	1.0
VIIa	Loose	Loose loam, loose gravel	1800-2000	--	0.8
VIII	Soils	Soil with vegetation, peat, soft loam, wet sand	1600-1800	--	0.6
IX	Granular soils	Sand, fine gravel, upfill	1400-1600	--	0.5
X	Plastic soils	Silty ground, modified loose & other soils in liquid condition	--	--	0.3

the second order parabola (Fig. 2.7).

In Eszto's theory, it is assumed that cavity created at depth H and of width b, would not be called upon to carry the weight of the entire rock prism extending to the surface, but one part of the rock load would be transmitted by friction and cohesion i.e. by its internal strength to the intact rock. The correctness of this assumption is highly questionable as it would follow from the principle of rupture plane that instead of rock mass between the boundaries verticals of the cavity , it is the weight between the rupture surfaces which is to be distributed.

This weight would be partly resisted by friction along rupture surfaces. Also as pointed out by Eszto himself, the pressure calculated according to his theory should not be used as design criteria in practice. The only merit of this theory is that it provides a better inside upto the influence of factors governing the magnitude of rock pressures.

$$p = \gamma b \tan \nu \frac{\left[\log \frac{(H \tan \nu)}{b} - 1 - \frac{b}{H \tan \nu}\right]}{1 - \left[\frac{b}{H \cot \nu}\right]^2}$$

where $\nu = 45 + \phi/2$

γ = unit weight of rock

H = rock cover

b = width of cavity

ϕ = angle of internal friction.

2.6.7 Fenner's Ellipse Method

In this method it is assumed that failure occurs along an elliptical surface enveloping the opening and passing through the springing of the arch. A typical sketch of Fenner's ellipse is shown in Fig. 2.8. The rock loads on the roof are, thus due to the weight of the rock between the ellipse and the roof intrados. For massive, moderately jointed rock mass, the ellipse may be drawn for no tension condition. For blocky and seamy rock, the ellipse may be drawn for the boundary tangential stress equal to that existing prior to the excavation of the cavity. In a biaxial stress field, boundary stress in vertical axis is given by:

$\sigma_t = \sigma_h (1 + 2q/p) - \sigma_v$

where σ_h and σ_v are the horizontal and vertical stresses and p and q the horizontal and vertical axes of the ellipse respectively.

If $\sigma_h = N \sigma_v$

then $\sigma_t = \sigma_v [N (1 + 2q/p) - 1]$

for Fenner's ellipse of no tension

$\sigma_t = 0$ or

$q/p = (1 - N)/2N$

for Fenner's ellipse with $\sigma_t = \sigma_h = N \sigma_v$

$q/p = 1/2N$

2.6.8 Norwegian Method

The Norwegian method consists of drawing a parabola from the springing of the roof and considering the rock as due to the weight of the rock between the parabola and roof intrados. In most cases, angle of tangent at the abutment is about 40° (Fig. 2.9)

Equation of the Parabola : $y^2 = 4 a x$

2.7 HOEK AND BROWN EMPIRICAL FAILURE CRITERION

Hoek and Brown (1980,1983) developed an empirical failure criterion relating to the major and minor principal stresses at failure to the uniaxial compressive strength of the intact rock and two empirical constants m and s. The criterion can be used to express the rock mass strength in Mohr Coulomb terms also, i.e. the cohesion and friction angle of rock mass at failure. Hoek not only described the practical procedures by which the empirical constants could be determined, but also suggested a table of values for them based on RMR and Q values of rock mass. These tables rapidly came into common use for estimation of rock strength parameters for incorporation in, for example, numerical analysis of single or multiple underground excavations, thereby providing a means of establishing support requirements for such openings by analysis. Subsequently Hoek (1988) modified the relationship between m and s and RMR or Q values of rock mass. Table 2.9 can be referred for the values of m and s. They also indicated that criterion

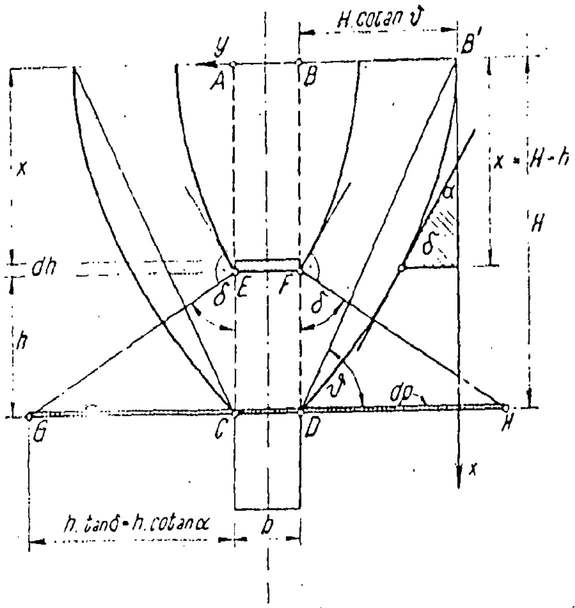


FIG. 2.7 : ASSUMPTION MODEL OF ESZTO'S ROCK PRESSURE THEORY

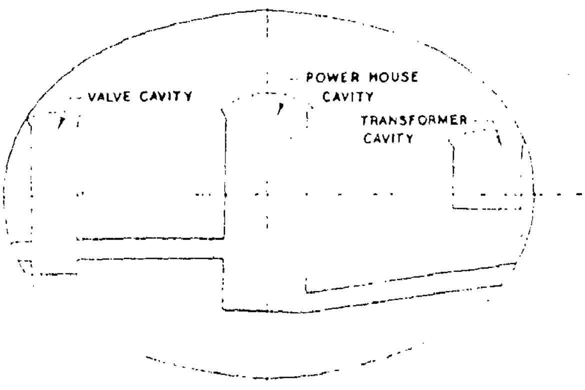


FIG. 2.8 : TYPICAL SKETCH OF FENNER'S ELLIPSE AROUND CLOSELY SPACED CAVITIES

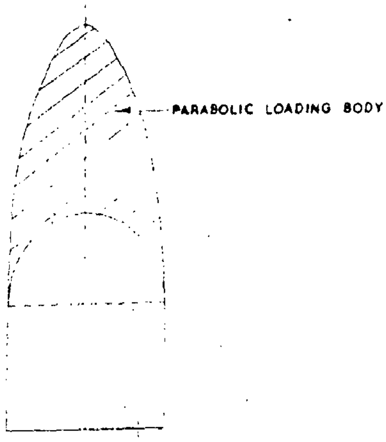


FIG. 2.9 : NORWEGIAN METHOD

TABLE 2.9 : APPROXIMATE RELATIONSHIP BETWEEN ROCK MASS QUALITY AND MATERIAL CONSTANTS FOR HOEK AND BROWN FAILURE CRITERION (AFTER HOEK & BROWN, 1988)

EMPIRICAL FAILURE CRITERION $\sigma_1 = \sigma_3 + \sqrt{m\,\sigma_3 + S\sigma_c^2}$ σ_1 = major principal stress σ_3 = minor principal stress σ_c = uniaxial compressive strength of intact rock, and m and s are empirical constants		CARBONATE ROCKS WITH WELL DEVELOPED CRYSTAL CLEAVAGE dolomite, limestone and marble	LITHIFIED ARGILLACEOUS ROCKS mudstone, siltsatone, shale and slate (normal to cleavage)	ARENACEOUS ROCKS WITH STRONG CRYSTALS AND POORLY DEVELOPED CRYSTAL CLEAVAGE sandstone and quartzite	FINE GRAINED POLYMINERALLIC IGNEOUS CRYSTALLINE ROCKS andesite, dolerite, diabase and rhyolite	COARSE GRAINED POLYMINERALLIC IGNEOUS & METAMORPHIC CRYSTAL-LINE ROCKS amphibolite, gabbro gneiss, granite, notrite, quartz-diorite
INTACT ROCK SAMPLES						
Laboratory size specimens free	m	7.00	10.00	15.00	17.00	25.00
from discontinuities	s	1.00	1.00	1.00	1.00	1.00
CSIR rating: RMR = 100	m*	7.00	10.00	15.00	17.00	25.00
NGI rating: Q = 500	s*	1.00	1.00	1.00	1.00	1.00
VERY GOOD QUALITY ROCK MASS						
Tightly interlocking undisturbed rock	m	2.40	3.43	5.14	5.82	8.56
with unweathered joints at 1 to 3m	s	0.082	0.082	0.082	0.082	0.082
CSIR rating: RMR = 85	m*	4.10	5.85	8.78	9.95	14.63
NGI rating Q = 100	s*	0.189	0.189	0.189	0.189	0.189
GOOD QUALITY ROCK MASS						
Fresh to slightly weathered rock,	m	0.575	0.821	1.231	1.395	2.052
slightly disturbed with joints at 1 to 3m	s	0.00293	0.00293	0.00293	0.00293	0.00293
CSIR rating: RMR = 65	m*	2.006	2.865	4.298	4.871	7.163
NGI rating: Q = 10	s*	0.0205	0.0205	0.0205	0.0205	0.0205
FAIR QUALITY ROCK MASS						
Several sets of moderately weathered	m	0.128	0.183	0.275	0.311	0.458
joints spaced at 0.3 to 1m.	s	0.00009	0.00009	0.00009	0.00009	0.00009
CSIR rating: RMR = 44	m*	0.947	1.353	2.030	2.301	3.383
NGI rating: Q = 1	s*	0.00198	0.00198	0.00198	0.00190	0.00198
POOR QUALITY ROCK MASS						
Numerous weathered joints at 30-500mm	m	0.029	0.041	0.061	0.069	0.102
some gauge.Clean compacted waste rock	s	0.00003	0.000003	0.[illegible]03	0.000003	0.000003
CSIR rating: RMR = 23	m*	0.447	0.639	0.[illegible]	1.087	1.598
NGI rating: Q = 0.1	s*	0.00019	0.00019	0.00[illegible]	0.00019	0.00019
VERY POOR QUALITY ROCK MASS						
Numerous heavily weathered joints spaced	m	0.007	0.0010	0.015	0.017	0.025
<50mm with gauge. Waste rock with fines	s	0.0000001	0.000000	0.000000	0.000000	0.0[illegible]
CSIR rating: RMR = 3	m*	0.219	0.313	0.469	0.532	0.78[illegible]
NGI rating: Q = 0.01	s*	0.00002	0.00002	0.00002	0.00002	0.00002

* Disturbed rock mass m and s values

Undisturbed rock mass m and s values

should only be used when rock mass is essentially homogeneous such as without continuous joints or when four or more comparable joints sets were present.

2.8 TERZAGHI'S MODIFIED ROCK CLASSIFICATION (SINGH ET. AL. 1995)

Bhawani Singh et. al. (1995) developed a simple classification system for rock masses on the basis of 23 case studies. He concluded that the support pressures does not increase directly with the size of excavation as suggested by Terzaghi (1946). Terzaghi's rock load classification does not provide reliable support pressure values for large tunnels and caverns under non squeezing or hard rock conditions. Further the estimated pressures for squeezing and swelling ground conditions fall in a large range for a meaningful application. Table 2.10 shows modified rock classification and support recommendations.

Various methods are available for determining the rock load on the roof arch. However, there is no way of ascertaining which method is accurate. With the different values of rock loads obtained by above methods, it is left to the judgement of the designer to adopt the design load in a particular situation.

TABLE 2.10: BHAWANI SINGH'S RECOMMENDATION ON SUPPORT PRESSURE FOR ROCK TUNNELS (SINGH ET. AL. 1995)

Category	Rock condition	Rock Load		Remarks
1	Hard and Intact	0	0	
2	Hard, stratified, or schistose	0-0.4	0	
3	Massive, Moderately Jointed	0.4-0.7	0	
4	Moderately Blocky, seamy, very jointed	0.7-1.0	0-0.2P_v	Inverts may be required
5	Very Blocky & Seamy, shattered, highly jointed, thin shear zone or fault	1.0-2.0	0-0.5P_v	Inverts may be required, roof preferred
6	Completely crushed but chemically unaltered, thick shear & fault zone	2.0-3.4	0.3-1.0P_v	Inverts essential, arched roof essential
7	**Squeezing Rock Condition**: A. Mild squeezing u/a upto 3 %	3.0-4.0	Depends on primary stress value P_h may exceed P_v	Invert Essential in excavation, flexible Support
	B. Moderate squeezing u/a 3-5%	4.0-6.0		preferred circular ribs recommended
	C. High squeezing u/a>5 %	6.0-14.0		
8	**Swelling rocks** : A. Mild swelling	3.0-8.0	Depends on type and content of swelling	Inverts essential in excavation, arched roof essential
	B. Moderate swelling	8.0-14.0		
	C. High swelling	14-20.0		

Notations : P_v=vertical support pressure, P_h=Horizontal support pressure, B=Height of opening, u=radial tunnel closure, a=B/2, Thin shear zone = upto 2 m thick

CHAPTER 3
DETERMINATION OF ROCK LOADS

A number of underground excavations in rock relating to water resources projects were taken up for the calculation of rock loads or support pressures using different methods. The methods widely applied have been used to work out the support requirements for underground openings. These methods include the following.

- Terzaghi's Method (1946)
- Deere's Rock Quality Designation (1963 and 1967)
- Wickham's Rock Structure Rating (1972)
- Bieniawski's Rock Mass Rating system (1974 and 1979)
- Barton's Q system (1974 and 1993)

3.1 NATHPA JHAKRI H.E. PROJECT, H.P.

Nathpa Jhakri H.E. Project envisages the utilisation of about 488 m drop in river Sutlej between Nathpa and Jhakri in Himachal Pradesh on the Indo-Tibet National Highway about 150 km from Shimla. The fully underground project consists of concrete gravity dam, four underground desilting chambers, 10.15 m dia and 27.3 km long head race tunnel, 301 m deep underground surge tank, three pressure shafts, underground power house 222 m (L) x 20 m (W) and 49 m (H), underground transformer hall and 10.15 m diameter and 960 m long tail race tunnel with a downstream surge gallery.

3.1.1 Geology

The pre-cambrian rocks belong to the Wangtu-Jeory Gneissic Complex in the eastern margin of Rampur window. They are surrounded by Jutog series of carbonaceous slates, limestones, quartzites and schist separated by Main Central Thrust (MCT) which is prominent and well known shear zone in the Himalayan region. The weaker rocks (mainly schists) are folded with more than two generations of folds and are intersected by steeply dipping faults and shear zones.

The area encompassing the power house site contains essentially quartz-mica schist. These rocks are moderately jointed and at places slightly to moderately weathered. The rocks are intruded by quartzite veins of varying thickness often forming boundaries which follow the foliation trend. Fig. 3.1 shows the geological map of Nathpa Jhakri Project.

3.1.2 Rock Mass Classification and Rock Pressures

Depending upon the geological and engineering properties of the rock, RMR and RSR have been calculated (Appendices 3A and 3B respectively). The rock loads have been presented in Table 3.1. For details of tunnel supports refer Bhasin et.al.(1996a).

TABLE 3.1 : ROCK CLASS AND SUPPORT PRESSURES AT NATHPA JHAKR POWER HOUSE, H.P.

Method of Rock Classification	Rock Type :Quartz Mica schist
Rock Class by Terzaghi	Massive Moderately Jointed (Class 4)
Supp. Pr. (kg/cm^2)	1.35 to 6.5
Wickham' RSR (Appendix 3A)	52
Supp. Pr., kg/cm^2	5.07
Bieniawski's RMR(Appendix 3B)	60 (Fair)
Support Pr. (kg/cm^2)	2.16
Barton's Q Value	2.7 (Fair)
Supp. Pr.(kg/cm^2), P_{roof}	0.42
P_{wall}	0.31

3.1.3 Supports Recommended

Wickham's RSR method 25.40 mm diameter bolts at 0.65 m centers should be provided. As per Bieniawski's RMR the systematic bolts 4m long at 1.5-2.0 m in crown & walls with shotcrete 5 to 10 cm in crown and 3 cm on walls are adequate support. However by Q system 5m long untensioned bolts in roof and 10 m long bolts in the walls at 2.1 m spacing alongwith 9 cm thick fibre reinforced shotcrete would be required.

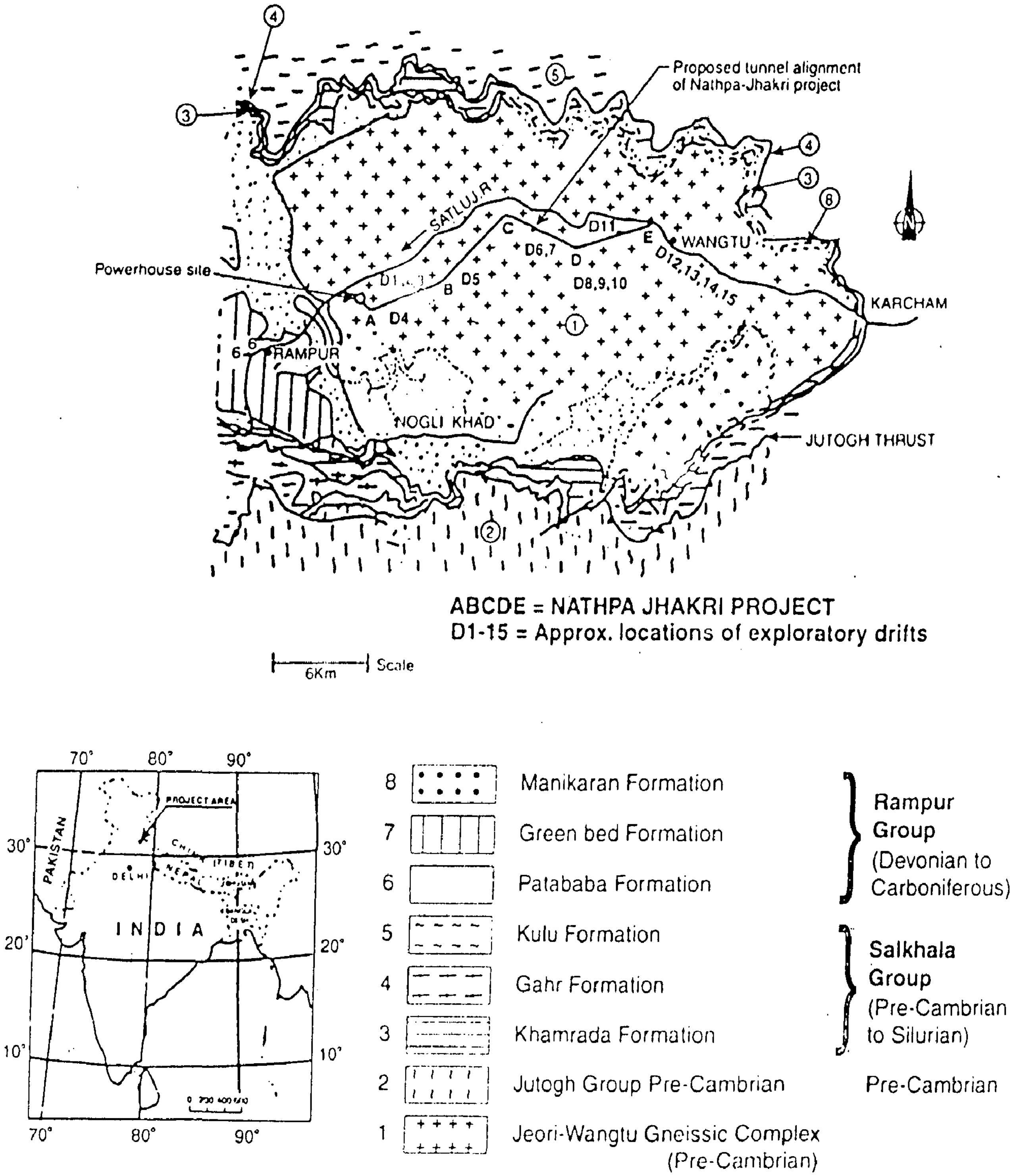


FIG. 3.1 : GEOLOGICAL MAP OF NATHPA JHAKRI PROJECT

3.1.4 Support Actually Provided

Roof Arch

Crown portion : 6 m long, 25 mm dia rock bolts both ways with 8 m long, 32 mm dia bolts at 2 m spacing

Remaining part : 32 mm dia, 6 m & 8 m long bolts at 3 m spacing

Walls :

Below springing Level : Rock Bolts of 32 mm dia, 7.5 m and 9 m long, at 3m spacing

Central Portion : 32 mm dia, 11 m and 9 m long, at 3 m spacing

Lower Portion : 32 mm dia, 7.5 m and 9 m long, at 3 m spacing

In addition to the rock bolts, two layers of shotcrete of 5 cm thickness with welded wire mesh in between have also been provided. Fig. 3.2 shows the support system adopted in the machine hall.

3.2 SARDAR SAROVAR PROJECT, GUJARAT

The multipurpose Sardar Sarovar Project, on river Narmada in the state of Gujarat of 1450 MW installed capacity (including river bed and canal bed power houses), is presently under construction. The fully underground power house 212m (L) x 23 m (W) x 58 m (H) cavern will house six turbines of 200 MW each will work under a head of 100 m and is situated immediately downstream of the 128 m high and 1210 m long concrete gravity dam across river Narmada.

3.2.1 Geology

The bed rock within the area around the power house consists of sub-horizontal lava flows of basalt with intrusive dolerite sills and lenses of agglomerates. Mainly three joint sets have been identified along with some randomly oriented joints. Bedding is sub horizontal. The orientation of the joints are as follows.

1) NNW/60° - 80° SE,SW

2) ENE/60° - 80° SE/NW

3) ENE/30° - 45° NW

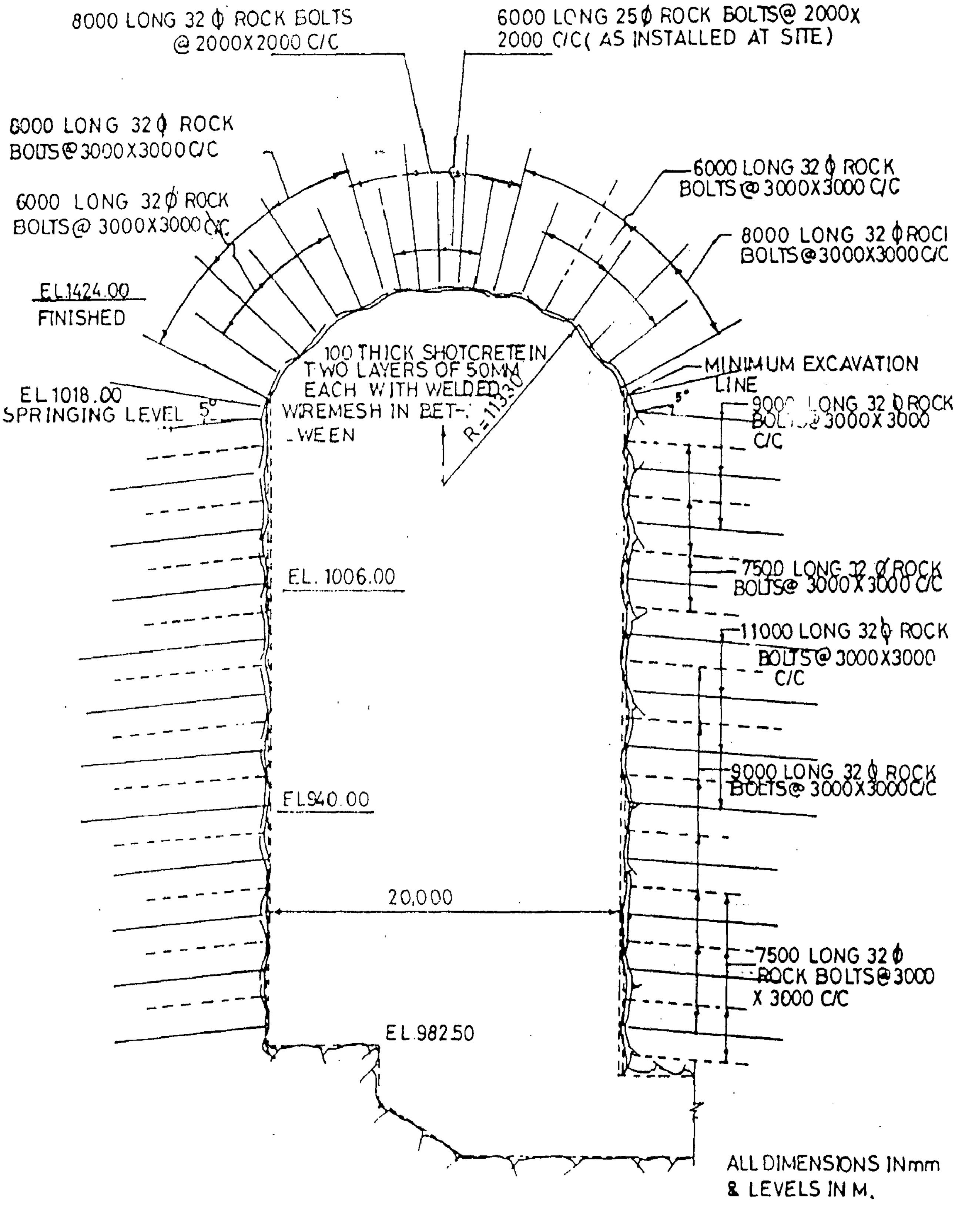


FIG. 3.2 : ROCK SUPPORTS ADOPTED IN NATHPA JHAKRI POWER HOUSE

Joint striking ENE have been found to contain thin fillings of calcite and chlorite. In general the joints can be described as having a rough surface, very narrow aperture and having medium persistence. These characteristics have been found to be favourable for constructing a cavern of such dimension.

A shear zone of 1 to 2 m wide dipping 60° to 65° south rows across the cavern along the contact of a dolerite dyke at the southern end. It consists of rock fragments of dolerite with little clay and is calcified. The porphyritic basalt which covers 85% of the cavern roof, is traversed by two shear zones 0.1 to 0.8 m thick running across the cavern roof.

3.2.2 Rock Mass Classification and Rock Pressures

The rock pressures have been presented in Table 3.2.

TABLE 3.2 : ROCK CLASS AND SUPPORT PRESSURES AT SARDAR SAROVAR POWER HOUSE, GUJARAT

Method of rock classification	Rock Type		
	Basalt	Dolerite	Sheared rock mass
Rock Class By Terzaghi	Very Blocky & Seamy,Class 4	Very Blocky & Seamy,Class 4	Completely crushed but chemically inert,Class 5
Supp.Pr., kg/cm^2	1.6 to 4.4	1.6 to 4.4	2.2 to 6.8
Bieniawski's Rock Mass Rating	63 (Good)	72 (Good)	40-21 say 30 (Poor)
Supp.Pr.,kg/cm^2	2.26	1.71	4.57
Barton's Q Values (Bhasin et.al., 1996)	9.2 to 14.4 (Av. 11.8)	14.4 to 18.5 (Av. 16.45)	0.33
Supp.Pr., kg/cm^2 P_{roof}	0.63 - 0.73	0.58 - 0.63	1.1

3.2.3 Support Recommended

The recommended supports have been presented in Table no 3.3.

3.2.4 Supports Actually Provided

The roof of the underground power house, located within the basaltic rocks, nave been stabilised with Willium's hollow core, mechanically anchored grouted bolts, 6-7 m long bolts at 1.75 m staggered spacing, with two layers of 7.5 cm thick shotcrete wire mesh. Across shear zone, three rows of 8-10 m long inclined bolts were installed with minimum 1.5 m length grouted in sound rock.

TABLE 3.3 : RECOMMENDED SUPPORT FOR SARDAR SAROVAR POWER HOUSE

Rock Type		
Basalt	Dolerite	Shear Zone
As per Bieniawski's RMR Method		
Locally 20 mm dia rock bolts 3 m long at 2.5m + Shotcrete 5 cm in crown where applicable		20 mm dia Systematic bolts at 1-1.5m + Shotcrete 10-15cm thick in crown and 10cm on sides
As per Barton's Q System		
Systematic rock bolts, 6m long in roof and 11 m in walls at 2.3m + Shotcrete of 5 cm thickness		Rock bolts at 1.5 m spacing + 15 cm thick Fibre reinforced shotcrete.

The side walls of P.H. were initially reinforced using 6 m long bolts at 2 x 2 m pattern, pre tensioned to 14 T and grouted. The distressed area have been reinforced with 40 tendons, pre-tensioned to 50 T. Numerical studies showed stress concentration around the junction which were required to be strengthened using 32 mm diameter, 10 m long pre tensioned grouted bolts.

3.3 SANJAY VIDYUT PARIYOJNA, H.P.

Sanjay Vidyut Pariyojna is located underground in a hill which runs nearly east-west in Sungra in Kinnaur district of Himachal Pradesh. This hill is flanked by the river Sutlej in the south and Bhaba in the north. The size of the power house is 71m (L) x 20.5 m (W) x 12.25 m (H). Power house capacity is 3x40 MW (Pelton Turbines) operating under a head of 988 m.

3.3.1 Geology

The site of power house lies on the crystalline rocks belonging to the Jutog Group. The Jutog formation comprises phylites, carbonaceous schists, sericite mica schists with tiny garnets, quartz-biotite schists and amphibolites. The site has been explored by a drift 200 m long with its axis running between N35°E and N40°E cuts across the rocks belonging to the Jutogs.

3.3.2 Rock Mass Classification and Rock Pressures

The rock pressures have been worked out using various methods and are presented in Table 3.4.

TABLE 3.4 : ROCK CLASS AND SUPPORT PRESSURES AT SANJAY VIDYUT PARIYOJNA, H.P.

Method of Classification	Augen Gneiss Rock
Rock Class by Terzaghi	Very blocky and seamy (Class 5)
Supp. Pr. (kg/cm^2)	5.34 to 16.8
Bieniawski's RMR Value (Agarwal et. al. 1985)	44 (Poor)
Supp. Pr. (kg/cm^2)	3.03
Stand-up time	Immediate Support is required
Barton's Q Value (Agarwal et. al. 1985)	13 (Good Rock)
Supp. Pr,(kg/cm^2), P_{roof}	0.16
P_{wall}	0.09

3.3.3 Supports Recommended

Bieniawski's Method suggests the systematic bolts at 1.5 to 2.0 m spacing in crown and walls with wire mesh in crown and shotcrete 10-15 cm in crown and 10 cm on walls. Whereas as per Q system, 5m long bolts at 1.1 m (untensioned) & 5 cm thick mesh reinforced shotcrete are the adequate support.

3.4 BASPA H.E. PROJECT, H.P.

Underground power house complex for Baspa Hydro-electric Project is located on the left bank of river Sutlej about 800 m u/s of confluence of river Sutlej and Baspa. The power house cavity of Baspa is 92 m (L) x 18 m (W) x 39.75 m (H). It will house 3 pelton turbines and generating units of 100 MW each. spherical inlet valves of 1.5 m diameter, service bay at one end and control block on the other end. Transformer Hall cavity 75 m (L) x 13 m (W) x 20.4-m

(H) is aligned parallel to the power house cavity at a distance of 31 m in the downstream direction.

3.4.1 Geology

One central adit has been excavated along the full length (92 m) of the power house cavity in N82°W - S82°E direction. In this adit the rock show a general strike of N10°E - S10°W to N20°E - S20°W and dip of 45° in S70°E - S80°E direction, whereas in the exposed cliff face the strike ranges from N-S to N10°E to S10°W and dip varies from 45° to 50° in easterly direction (Fig. 3.3). In the adit quartzite has been met from chainage 0-37m and 51-53m (42.4% length) while quartzite mica schist from 37-51 and 53 to 92 m (57.6% length). Average Q values are above 4 (Singh et. al. 1995b) except in reaches 17-50 m where the average Q value is 3.

3.4.2 Rock Mass Classification and Rock Pressures

The rock pressures and the rock quality has been presented in Table 3.5.

TABLE 3.5 : ROCK CLASS AND SUPPORT PRESSURES AT BASPA POWER HOUSE, H.P.

Method of rock Classification	Description
Rock Class by Terzaghi	Massive Rock (Class 3)
Supp. Pr. (kg/cm^2)	0 to 2.39
Wickham's RSR (Appendix 3C)	78
Supp.Pr., kg/cm^2	0.2
Bieniawski's RMR	54-62 (Fair Rock)
	1.74 - 2.1
Stand-up time	Immediate Support is required
Barton's Q Value (Singh et. al., 1995b)	3 to 8 (Fair Rock)
Supp. Pr,(kg/cm^2), P_{roof}	0.66 to 0.92
P_{wall}	0.49 to 0.68

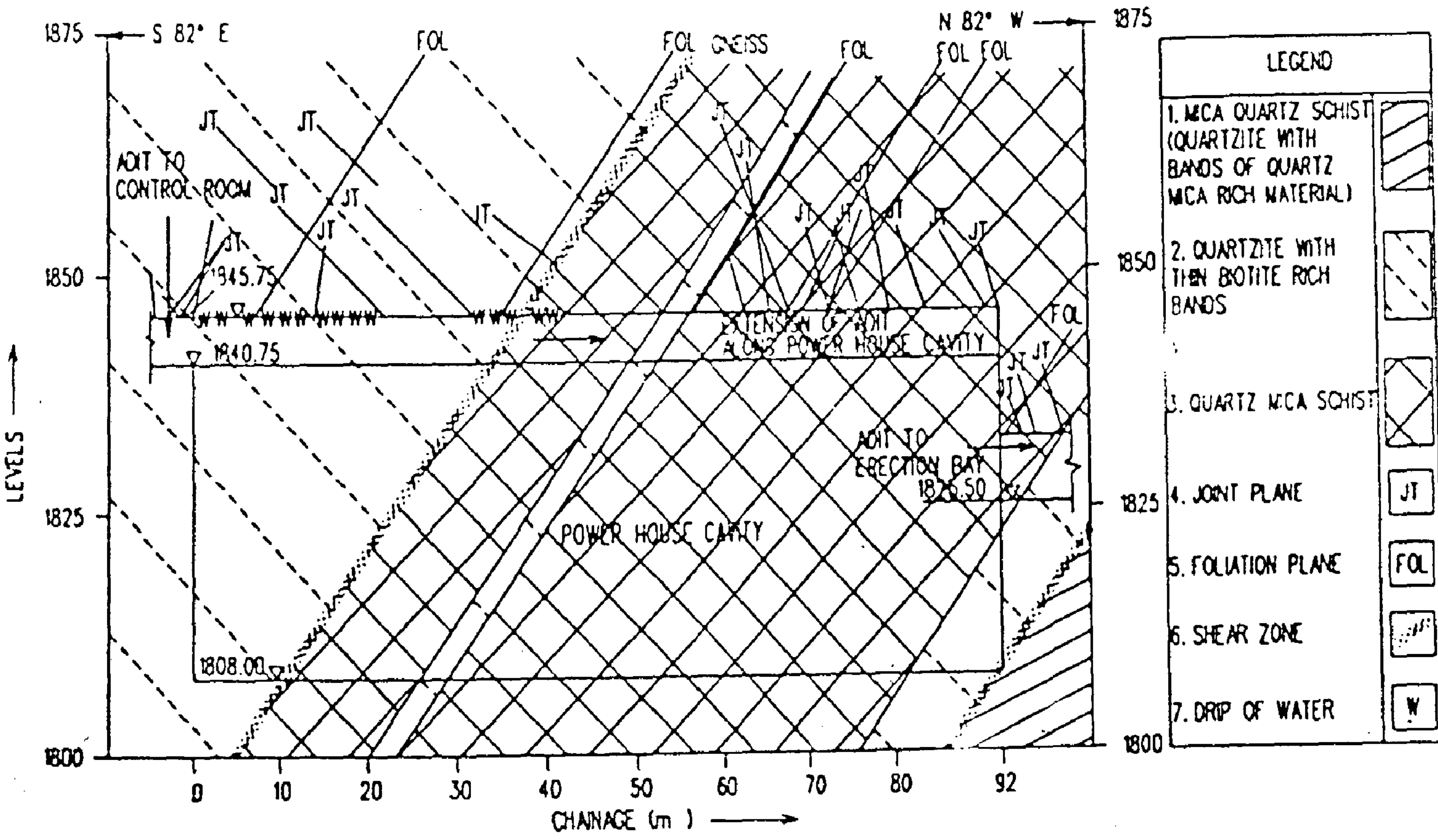


FIG. 3.3 : GEOLOGICAL SECTION ALONG CENTRE LINE OF BASPA POWER HOUSE CAVITY

3.4.3 Support Recommended

As per RMR method 20mm diameter, 4 m long systematic bolts at a spacing of 1.5 to 2.0 m in crown and walls with wire mesh in crown should be provided. Shotcreting 5 to 10 cm on the crown and 3 cm on sides is also required. Q system recommends the use of 5 m long rock bolts in roof and 8 m in walls spaced at 2.1 m and shotcrete of 5 cm thickness.

3.4.4 Support Actually Provided

Chainage 0-24 and 51-92 m.

1. 10 cm thick shotcrete with weld mesh (100 mm x 100 x4.2 mm)
2. 25 mm diameter bolts with grid spacing of 1.5 m. Depth of rock bolts will be 5m and 6 m staggered.

Chainage 0-24 and 51-92 m.

1. 15 cm thick shotcrete with weld mesh (100 mm x 100 x4.2 mm)
2. 25 mm diameter bolts with grid spacing of 1.25 m. depth of rock bolts will be 5 m and 6 m staggered.

   Rock bolts are tensioned and grouted to load of 15 Tonnes.

3.5 LAKHWAR H.E. PROJECT, U.P.

Lakhwar Vyasi Project envisages the construction of 204 m high concrete gravity dam across river Yamuna and an underground Power House located inside right abutment of the dam near village Lakhwar about 80 km from Dehradun in Uttar Pradesh. The power house will have an installed capacity of 3 x 100 MW and shall utilise a drop of 166 m. The power house cavity of 130 m (L) x 20 m (W) x 43.5 m (H) size comes almost in the line of the dam axis. Layout of the underground power house is shown in Fig. 3.4.

3.5.1 Geology

At the power house location there exists a narrow strip of Jaunsar traps (Basic rock) having maximum width of about 300 m along the flow of river which is just sufficient to accommodate the base width of solid gravity dam. The basic rock ranges in composition from

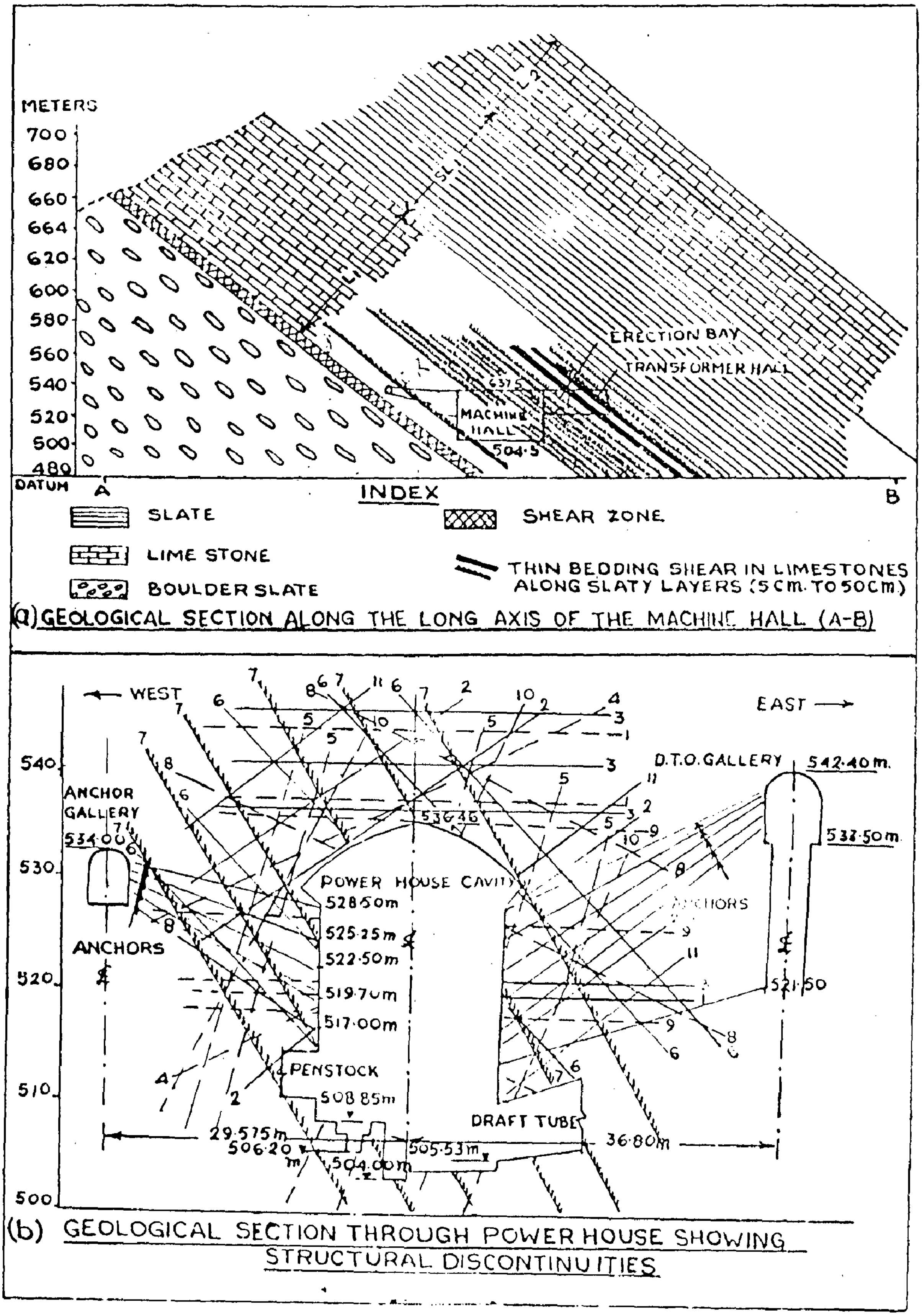


FIG. 3.6 : GEOLOGICAL SECTION OF CHHIBRO POWER HOUSE

3.6.3 Support Recommended

The support requirements have been worked out as 25 mm diameter rock bolts of 5 m length in the roof and 8 m in the walls at a spacing of 1.0 m with 10 cm thick shotcrete.

3.6.4 Support Actually Provided

The roof of the power house has been supported by steel arches and the walls have been supported by 350 prestressed anchors of average length 23.5 M of 600 KN capacity at 2-5 m spacing and reinforced shotcrete 7.5 Cm thick has been used where found necessary. In the roof R.S. Joists of 250 x 125 mm with cover plates of 250 x 20 mm at top and 150 x 20 mm at bottom spaced at 25 cm centres have been used as rock support. Backfill concrete of M-150 strength has been used. Fig. 3.7 shows the roof and wall supporting arrangements in the power house.

3.7 CHAMERA H.E. PROJECT, H.P.

The project is located near Chaurah village in Chamba district of Himachal Pradesh on approximate latitude N32^{o} 36$'$ and longitude E75^{o} 56$'$. The project is linked to the nearest rail head Pathankot by a 97 km road. The underground power house of 112 m (L) x 24 m (W) x 37 m (H) size is located at Khairi. The total installed capacity of the project is 540 MW (3 nos of francis turbines of 180 MW each). The project comprise of an 9.5 M finished diameter head race tunnel of 6414 m long.

3.7.1 Geology

The power house complex comprising of two caverns viz. The machine hall and transformer hall and other ancillary components have been excavated in metamorphosed andesite basalt. Rock mass in the power house may be categorised as blocky to foliated and intersected by five sets of discontinuities of different orientations, predominant being foliation joints and shears. Foliation joints are continuous and undulating and generally moderate to closely spaced. Most of the joint sets have low persistence. Water seepage in caverns during excavation was negligible. Geological section of the project is shown in Fig. 3.8.

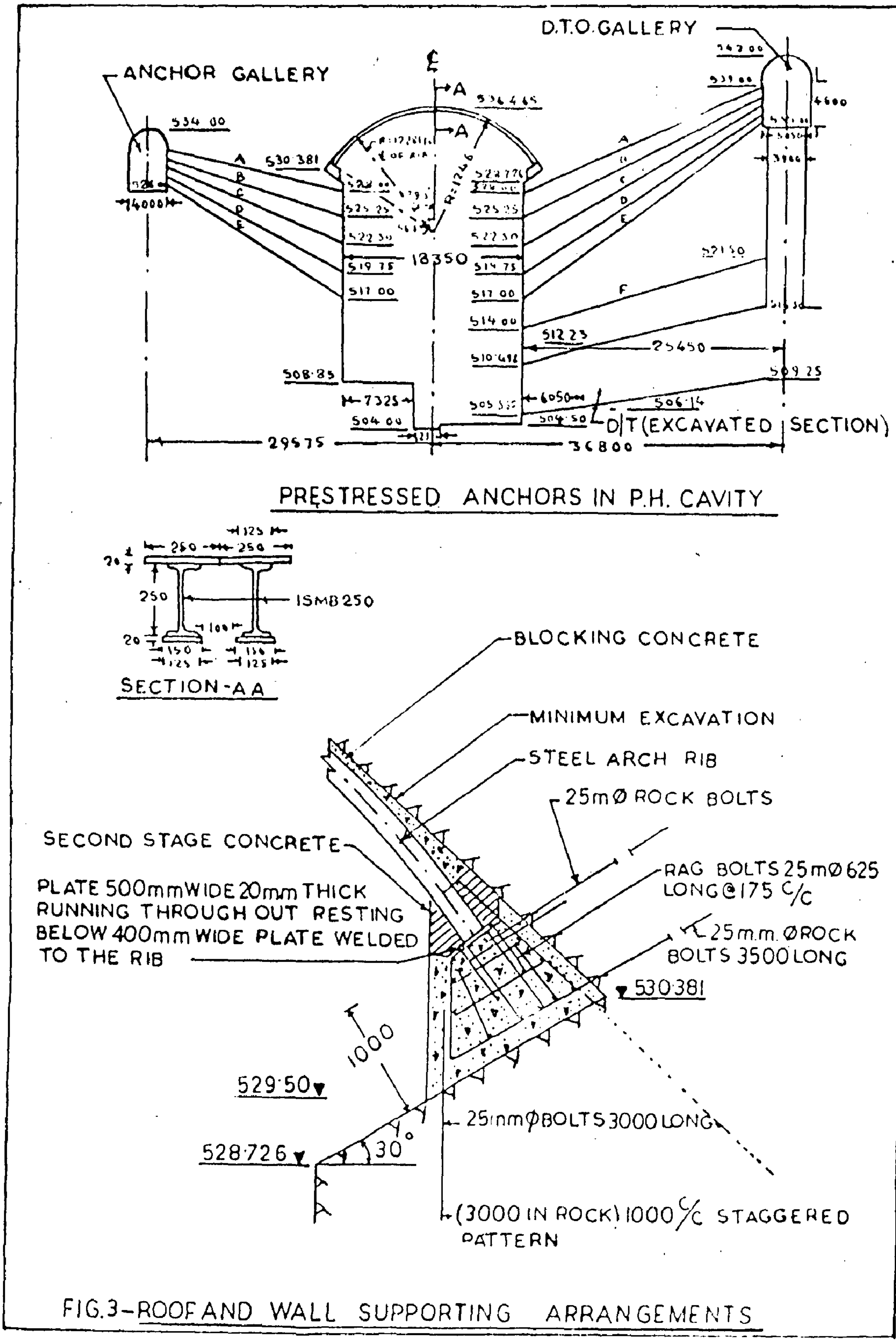


FIG. 3.7 : ROOF AND WALL SUPPORTS IN CHHIBRO POWER HOUSE

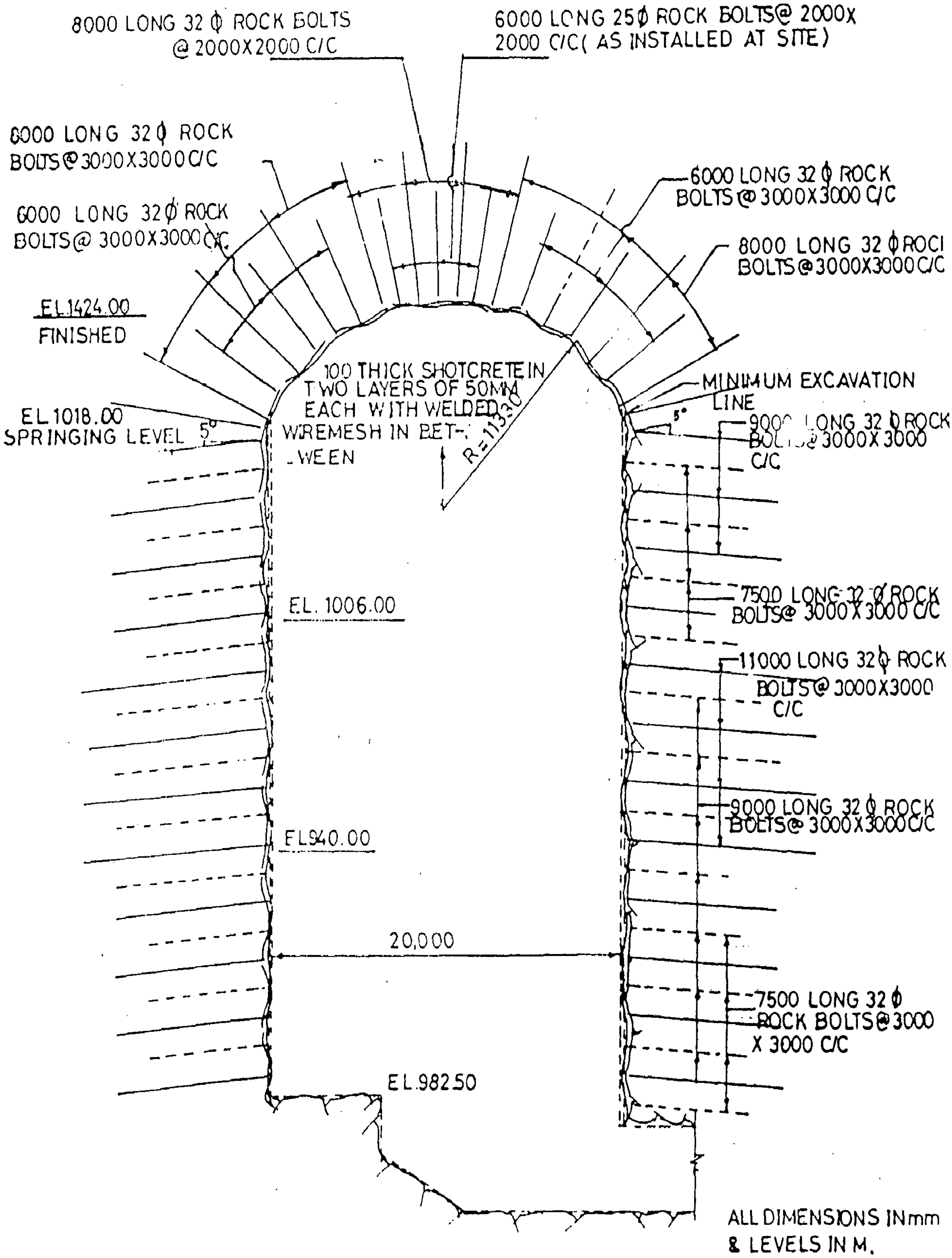


FIG. 3.2 : ROCK SUPPORTS ADOPTED IN NATHPA JHAKRI POWER HOUSE

Joint striking ENE have been found to contain thin fillings of calcite and chlorite. In general the joints can be described as having a rough surface, very narrow aperture and having medium persistence. These characteristics have been found to be favourable for constructing a cavern of such dimension.

A shear zone of 1 to 2 m wide dipping 60° to 65° south rows across the cavern along the contact of a dolerite dyke at the southern end. It consists of rock fragments of dolerite with little clay and is calcified. The porphyritic basalt which covers 85% of the cavern roof, is traversed by two shear zones 0.1 to 0.8 m thick running across the cavern roof.

3.2.2 Rock Mass Classification and Rock Pressures

The rock pressures have been presented in Table 3.2.

TABLE 3.2 : ROCK CLASS AND SUPPORT PRESSURES AT SARDAR SAROVAR POWER HOUSE, GUJARAT

Method of rock classification	Rock Type		
	Basalt	Dolerite	Sheared rock mass
Rock Class By Terzaghi	Very Blocky & Seamy,Class 4	Very Blocky & Seamy,Class 4	Completely crushed but chemically inert,Class 5
Supp.Pr., kg/cm^2	1.6 to 4.4	1.6 to 4.4	2.2 to 6.8
Bieniawski's Rock Mass Rating	63 (Good)	72 (Good)	40-21 say 30 (Poor)
Supp.Pr.,kg/cm^2	2.26	1.71	4.57
Barton's Q Values (Bhasin et.al., 1996)	9.2 to 14.4 (Av. 11.8)	14.4 to 18.5 (Av. 16.45)	0.33
Supp.Pr., kg/cm^2 P_{roof}	0.63 - 0.73	0.58 - 0.63	1.1

3.2.3 Support Recommended

The recommended supports have been presented in Table no 3.3.

3.2.4 Supports Actually Provided

The roof of the underground power house, located within the basaltic rocks, have been stabilised with Willium's hollow core, mechanically anchored grouted bolts, 6-7 m long bolts at 1.75 m staggered spacing, with two layers of 7.5 cm thick shotcrete wire mesh. Across shear zone, three rows of 8-10 m long inclined bolts were installed with minimum 1.5 m length grouted in sound rock.

TABLE 3.3 : RECOMMENDED SUPPORT FOR SARDAR SAROVAR POWER HOUSE

Rock Type		
Basalt	Dolerite	Shear Zone
As per Bieniawski's RMR Method		
Locally 20 mm dia rock bolts 3 m long at 2.5m + Shotcrete 5 cm in crown where applicable		20 mm dia Systematic bolts at 1-1.5m + Shotcrete 10-15cm thick in crown and 10cm on sides
As per Barton's Q System		
Systematic rock bolts, 6m long in roof and 11 m in walls at 2.3m + Shotcrete of 5 cm thickness		Rock bolts at 1.5 m spacing + 15 cm thick Fibre reinforced shotcrete.

The side walls of P.H. were initially reinforced using 6 m long bolts at 2 x 2 m pattern, pre tensioned to 14 T and grouted. The distressed area have been reinforced with 40 tendons, pre-tensioned to 50 T. Numerical studies showed stress concentration around the junction which were required to be strengthened using 32 mm diameter, 10 m long pre tensioned grouted bolts.

3.3 SANJAY VIDYUT PARIYOJNA, H.P.

Sanjay Vidyut Pariyojna is located underground in a hill which runs nearly east-west in Sungra in Kinnaur district of Himachal Pradesh. This hill is flanked by the river Sutlej in the south and Bhaba in the north. The size of the power house is 71m (L) x 20.5 m (W) x 12.25 m (H). Power house capacity is 3x40 MW (Pelton Turbines) operating under a head of 988 m.

3.3.1 Geology

The site of power house lies on the crystalline rocks belonging to the Jutog Group. The Jutog formation comprises phylites, carbonaceous schists, sericite mica schists with tiny garnets, quartz-biotite schists and amphibolites. The site has been explored by a drift 200 m long with its axis running between N35^{o}E and N40^{o}E cuts across the rocks belonging to the Jutogs.

3.3.2 Rock Mass Classification and Rock Pressures

The rock pressures have been worked out using various methods and are presented in Table 3.4.

TABLE 3.4 : ROCK CLASS AND SUPPORT PRESSURES AT SANJAY VIDYUT PARIYOJNA, H.P.

Method of Classification	Augen Gneiss Rock
Rock Class by Terzaghi	Very blocky and seamy (Class 5)
Supp. Pr. (kg/cm^2)	5.34 to 16.8
Bieniawski's RMR Value (Agarwal et. al. 1985)	44 (Poor)
Supp. Pr. (kg/cm^2)	3.03
Stand-up time	Immediate Support is required
Barton's Q Value (Agarwal et. al. 1985)	13 (Good Rock)
Supp. Pr.,(kg/cm^2), P_{roof}	0.16
P_{wall}	0.09

3.3.3 Supports Recommended

Bieniawski's Method suggests the systematic bolts at 1.5 to 2.0 m spacing in crown and walls with wire mesh in crown and shotcrete 10-15 cm in crown and 10 cm on walls. Whereas as per Q system, 5m long bolts at 1.1 m (untensioned) & 5 cm thick mesh reinforced shotcrete are the adequate support.

3.4 BASPA H.E. PROJECT, H.P.

Underground power house complex for Baspa Hydro-electric Project is located on the left bank of river Sutlej about 800 m u/s of confluence of river Sutlej and Baspa. The power house cavity of Baspa is 92 m (L) x 18 m (W) x 39.75 m (H). It will house 3 pelton turbines and generating units of 100 MW each. spherical inlet valves of 1.5 m diameter, service bay at one end and control block on the other end. Transformer Hall cavity 75 m (L) x 13 m (W) x 20.4-m

(H) is aligned parallel to the power house cavity at a distance of 31 m in the downstream direction.

3.4.1 Geology

One central adit has been excavated along the full length (92 m) of the power house cavity in N82°W - S82°E direction. In this adit the rock show a general strike of N10°E - S10°W to N20°E - S20°W and dip of 45° in S70°E - S80°E direction, whereas in the exposed cliff face the strike ranges from N-S to N10°E to S10°W and dip varies from 45° to 50° in easterly direction (Fig. 3.3). In the adit quartzite has been met from chainage 0-37m and 51-53m (42.4% length) while quartzite mica schist from 37-51 and 53 to 92 m (57.6% length). Average Q values are above 4 (Singh et. al. 1995b) except in reaches 17-50 m where the average Q value is 3.

3.4.2 Rock Mass Classification and Rock Pressures

The rock pressures and the rock quality has been presented in Table 3.5.

TABLE 3.5 : ROCK CLASS AND SUPPORT PRESSURES AT BASPA POWER HOUSE, H.P.

Method of rock Classification	Description
Rock Class by Terzaghi	Massive Rock (Class 3)
Supp. Pr. (kg/cm^2)	0 to 2.39
Wickham's RSR (Appendix 3C)	78
Supp.Pr., kg/cm^2	0.2
Bieniawski's RMR	54-62 (Fair Rock)
	1.74 - 2.1
Stand-up time	Immediate Support is required
Barton's Q Value (Singh et. al., 1995b)	3 to 8 (Fair Rock)
Supp. Pr.(kg/cm^2), P_{roof}	0.66 to 0.92
P_{wall}	0.49 to 0.68

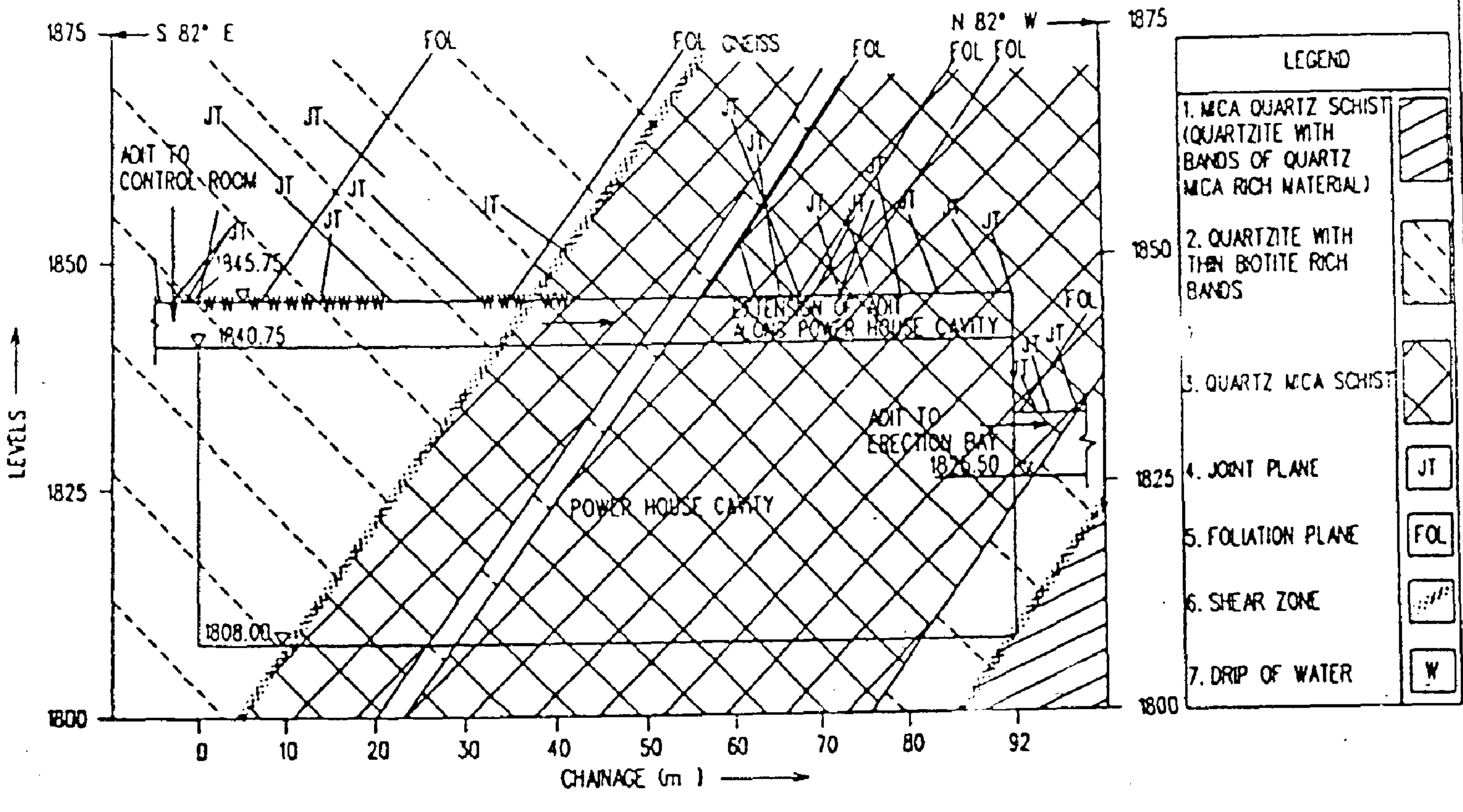


FIG. 3.3 : GEOLOGICAL SECTION ALONG CENTRE LINE OF BASPA POWER HOUSE CAVITY

3.4.3 Support Recommended

As per RMR method 20mm diameter, 4 m long systematic bolts at a spacing of 1.5 to 2.0 m in crown and walls with wire mesh in crown should be provided. Shotcreting 5 to 10 cm on the crown and 3 cm on sides is also required. Q system recommends the use of 5 m long rock bolts in roof and 8 m in walls spaced at 2.1 m and shotcrete of 5 cm thickness.

3.4.4 Support Actually Provided

Chainage 0-24 and 51-92 m.

1. 10 cm thick shotcrete with weld mesh (100 mm x 100 x4.2 mm)
2. 25 mm diameter bolts with grid spacing of 1.5 m. Depth of rock bolts will be 5m and 6 m staggered.

Chainage 0-24 and 51-92 m.

1. 15 cm thick shotcrete with weld mesh (100 mm x 100 x4.2 mm)
2. 25 mm diameter bolts with grid spacing of 1.25 m. depth of rock bolts will be 5 m and 6 m staggered.

   Rock bolts are tensioned and grouted to load of 15 Tonnes.

3.5 LAKHWAR H.E. PROJECT, U.P.

Lakhwar Vyasi Project envisages the construction of 204 m high concrete gravity dam across river Yamuna and an underground Power House located inside right abutment of the dam near village Lakhwar about 80 km from Dehradun in Uttar Pradesh. The power house will have an installed capacity of 3 x 100 MW and shall utilise a drop of 166 m. The power house cavity of 130 m (L) x 20 m (W) x 43.5 m (H) size comes almost in the line of the dam axis. Layout of the underground power house is shown in Fig. 3.4.

3.5.1 Geology

At the power house location there exists a narrow strip of Jaunsar traps (Basic rock) having maximum width of about 300 m along the flow of river which is just sufficient to accommodate the base width of solid gravity dam. The basic rock ranges in composition from

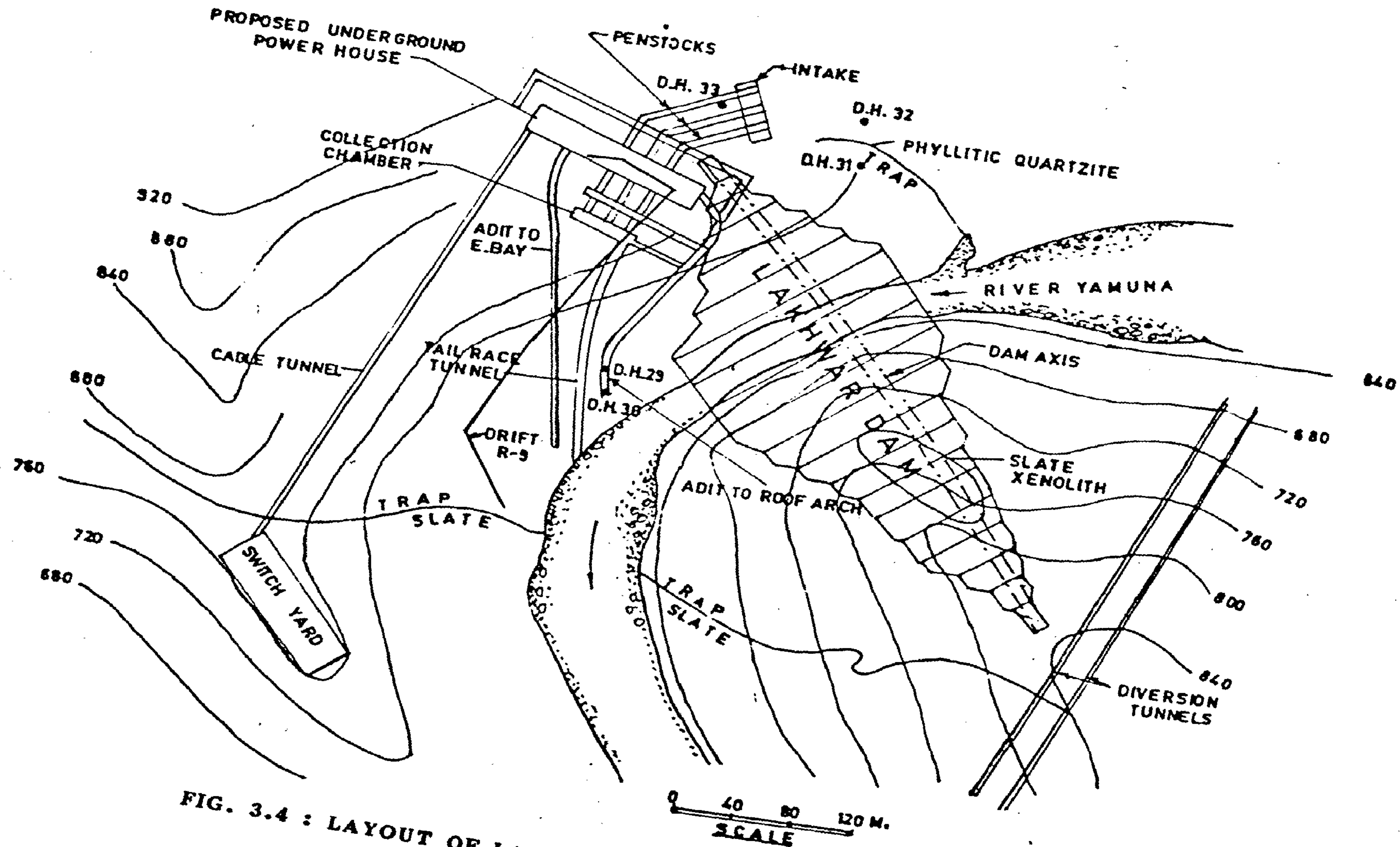


FIG. 3.4 : LAYOUT OF LAKHWAR UNDERGROUND POWER HOUSE

dolerite to hornblende rhyolite. The trap is generally coarse grained and jointed. The rock in the power house cavity at right abutment is comparatively massive and less jointed.

3.5.2 Rock Mass Classification and Rock Pressures

The rock pressures and are presented in Table 3.6.

TABLE 3.6 : ROCK CLASS AND SUPPORT PRESSURES AT LAKHWAR POWER HOUSE, U.P.

Method of rock Classification	Description
Rock Class by Terzaghi	Massive and Moderately Jointed Rock (Class 3)
Supp. Pr. (kg/cm^2)	0 to 2.65
Bieniawski's RMR Value	63 (Good Rock)
Supp. Pr. (kg/cm^2)	2.09
Barton's Q Value (Singh et. al. 1992)	8.5 (Fair Rock)
Supp. Pr,(kg/cm^2), P_{roof}	0.35

3.5.3 Support Recommended

As per RMR method the support works out to be 3 m long bolts at 2.5 m spacing with occasional wire mesh and 5 cm shotcrete whereas Barton's Q system recommends the use of 5m long bolts in roof and 9 m in walls at 2.1 m centres with 5 cm thick shotcrete.

2.5.4 Support Actually Provided

The power house cavity has been supported on ISMB 250 x 125 mm steel sets with 250 x 20 mm top and 150 x 20 mm bottom plate at 75 cm centres and 3 m long rock bolts at 1.5 m spacing The rib assembly used to support the rock is shown in Fig. 3.5.

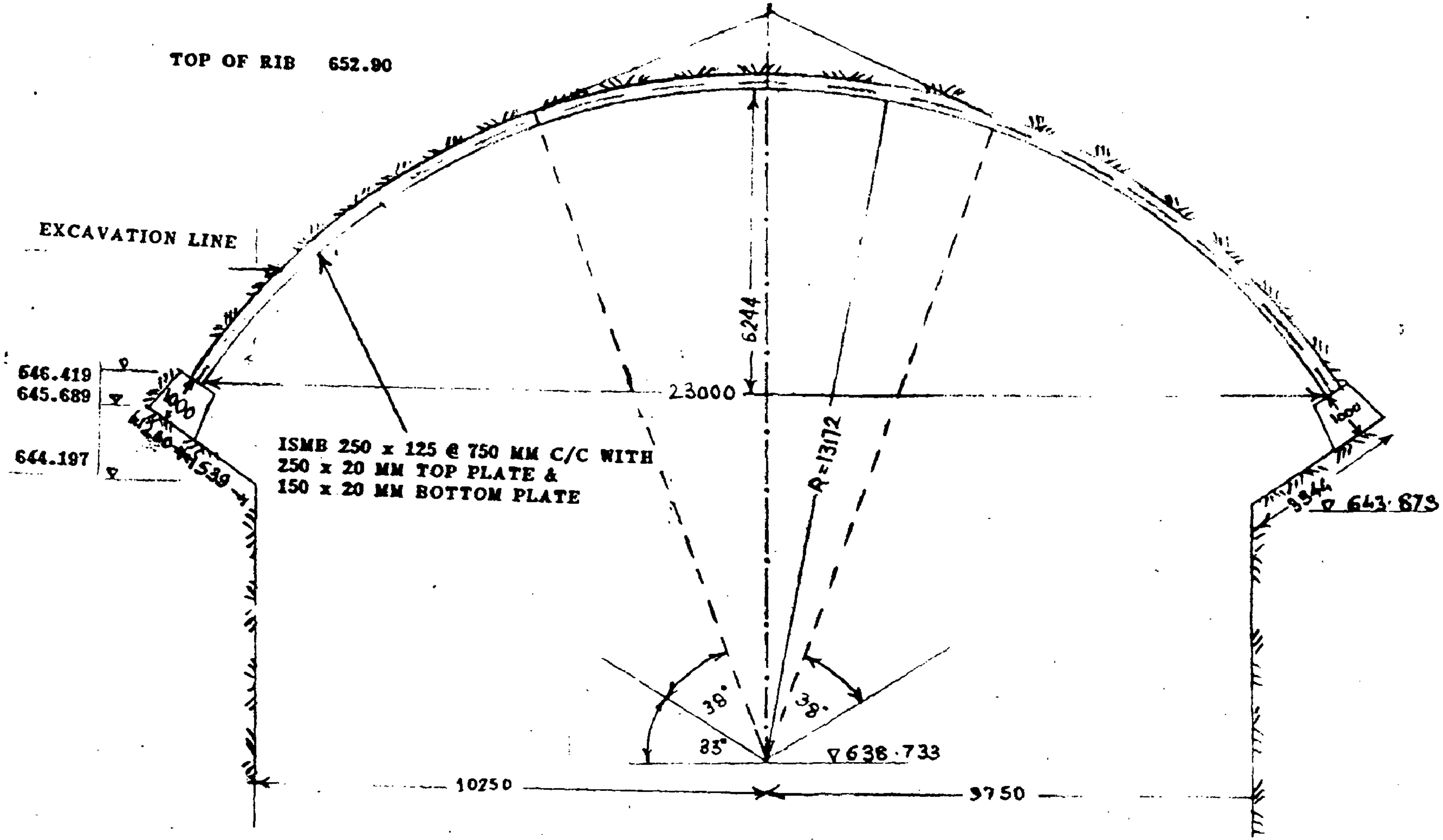


FIG. 3.5 : SECTION OF STEEL RIB SUPPORTED ARCH OF LAKHWAR POWER HOUSE

3.6 YAMUNA HYDRO-ELECTRIC SCHEME STAGE II (CHHIBRO POWER HOUSE) U.P.

Yamuna Hydro-electric Scheme Stage II in Dehradun district of U.P., envisages development of the power potential of river Tons,
a tributory of river Yamuna at Ichhari and its outfal at Dakpathar. The total available drop of 186 m is utilised for power generation in two stages. Part I utilises a drop of 124 m by the construction of dam at Ichhari, for diverting water through a 6.3 Km long tunnel to an underground power house at Chhibro (first underground power house) with installed capacity of 4 x 60 MW.

The underground power house at Chhibro comprises a network of cavities for housing the machines, transformers, turbine inlet valves, control room and to serve as various operating galleries and water conductor system to feed the Part II of the project. The main cavity is 113.2 M long x 18.35 M wide x 32.5 M high and has a circular roof and vertical sides.

3.6.1 Geology

The power house cavity is located in a stratified limestone band 25 m thick and 200 m horizontal thickness with minor or thinly bedded slate bands. The rock is closely jointed with numerous shear zones ranging from 2 to 50 cm thick and nearly parallel to the bedding. A major shear zone lies at a minimum depth of 10 m below the lowest draft tube level in the power house cavity. The formations dip at about 45° towards N150°W to N29°E. The cavity is aligned parallel to the strike of rock formations with cover ranging from 208 m over the transformer hall. Fig. 3.6 shows the geological section of the power house.

3.6.2 Rock Mass Classification and Rock Pressures

The rock at the site may be classified as stratified limestone closely jointed. The support pressure as per Terzaghi's classification works out to 4.8-5.1 kg/cm^2 (very blocky and seamy rocks, class 5) and by Singh et. al. 1995, 1-2.0 kg/cm^2.

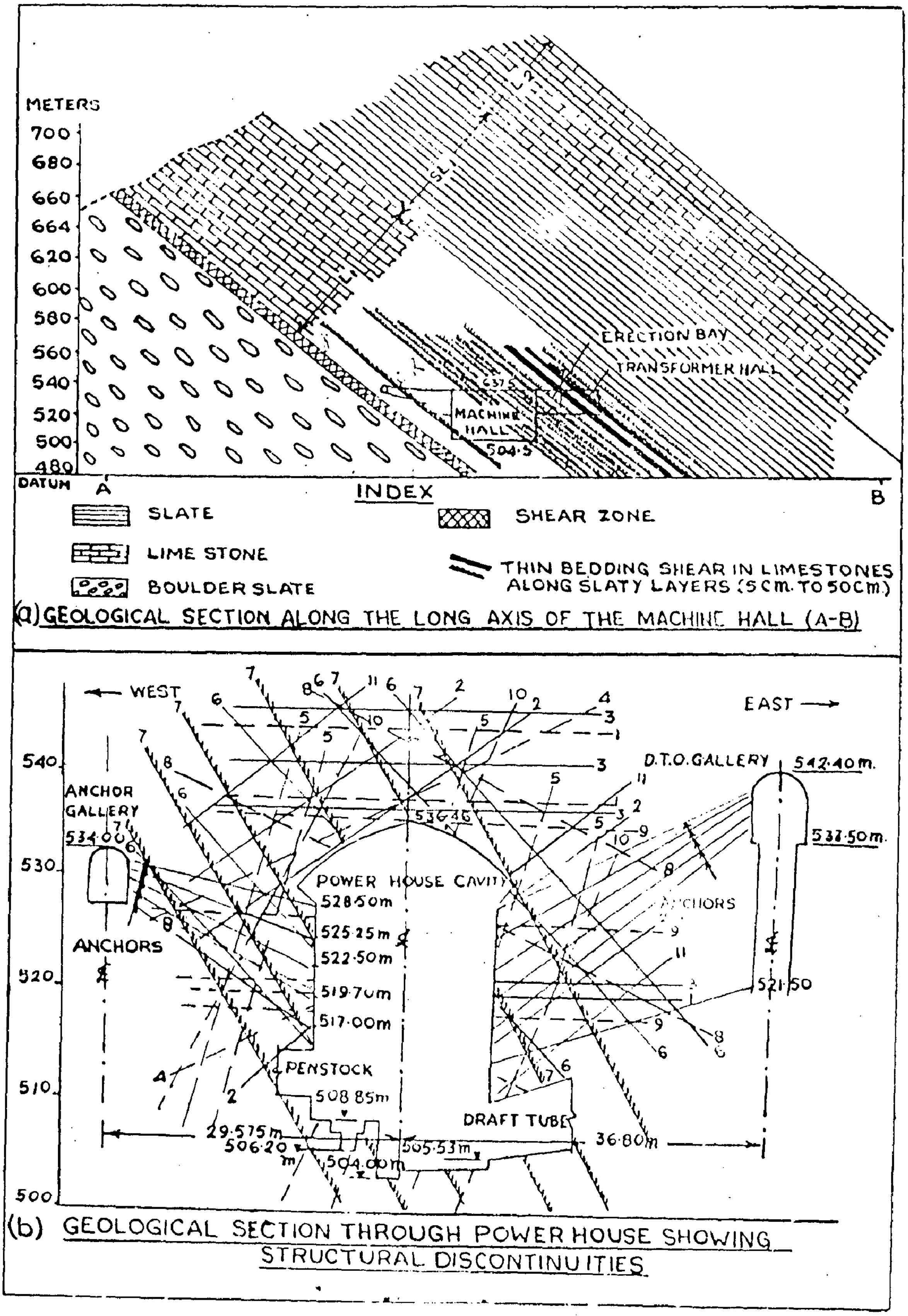


FIG. 3.6 : GEOLOGICAL SECTION OF CHHIBRO POWER HOUSE

3.6.3 Support Recommended

The support requirements have been worked out as 25 mm diameter rock bolts of 5 m length in the roof and 8 m in the walls at a spacing of 1.0 m with 10 cm thick shotcrete.

3.6.4 Support Actually Provided

The roof of the power house has been supported by steel arches and the walls have been supported by 350 prestressed anchors of average length 23.5 M of 600 KN capacity at 2-5 m spacing and reinforced shotcrete 7.5 Cm thick has been used where found necessary. In the roof R.S. Joists of 250 x 125 mm with cover plates of 250 x 20 mm at top and 150 x 20 mm at bottom spaced at 25 cm centres have been used as rock support. Backfill concrete of M-150 strength has been used. Fig. 3.7 shows the roof and wall supporting arrangements in the power house.

3.7 CHAMERA H.E. PROJECT, H.P.

The project is located near Chaurah village in Chamba district of Himachal Pradesh on approximate latitude N32° 36′ and longitude E75° 56′. The project is linked to the nearest rail head Pathankot by a 97 km road. The underground power house of 112 m (L) x 24 m (W) x 37 m (H) size is located at Khairi. The total installed capacity of the project is 540 MW (3 nos of francis turbines of 180 MW each). The project comprise of an 9.5 M finished diameter head race tunnel of 6414 m long.

3.7.1 Geology

The power house complex comprising of two caverns viz. The machine hall and transformer hall and other ancillary components have been excavated in metamorphosed andesite basalt. Rock mass in the power house may be categorised as blocky to foliated and intersected by five sets of discontinuities of different orientations, predominant being foliation joints and shears. Foliation joints are continuous and undulating and generally moderate to closely spaced. Most of the joint sets have low persistence. Water seepage in caverns during excavation was negligible. Geological section of the project is shown in Fig. 3.8.

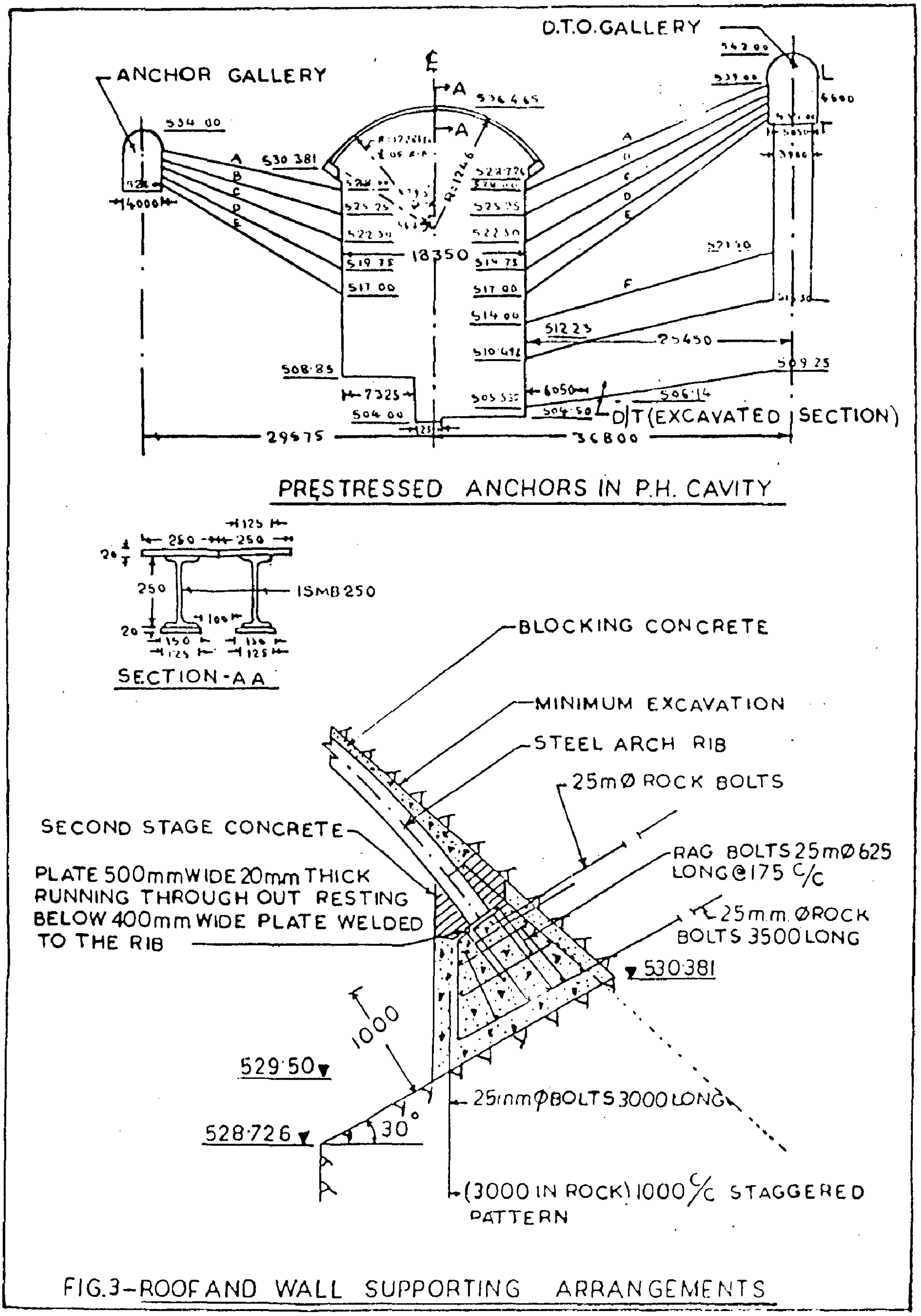


FIG. 3.7 : ROOF AND WALL SUPPORTS IN CHHIBRO POWER HOUSE

3.7.2 Rock Mass Classification and Rock Pressures

The rock mass classification is as per Table 3.7.

TABLE 3.7 : ROCK CLASS AND SUPPORT PRESSURES AT CHAMERA POWER HOUSE, H.P.

Method of classification	Rock type metamorphosed andesite basalt
Terzaghi's classification	Blocky to foliated, class 4
Supp. Pr., kg/cm^2	1.65 to 5.87
Bieniawski's RMR(Sharma et. al.1994)	50, Fair rock mass
Supp. Pr., kg/cm^2	3.3
Barton's Q value	1.95
Supp. Pr.,kg/cm^2, P_{roof}	1.07
P_{wall}	0.62

3.7.3 Supports Recommended

As per RMR method the required support system conform to systematic 20 mm diameter rock bolts of 4m length at a spacing of 1.5 to 2.0 m in the crown and walls with wire mesh in the crown alongwith 5-10 cm thick shotcrete layer in the crown and 3 cm in sides. However, Q system recommends rock bolts of 6 m length in the roof and 8 m in the walls at a grid spacing of 1.9 m with 10 cm thick fibre reinforced shotcrete layer.

3.7.4 Support Actually Provided

To support the rock mass, flexible support system consisting of combination of rock bolts, anchors and shotcrete was considered prudent with regular monitoring of excavated section by instrumentation. Based on this design approach, the following two support systems were adopted.

- 7.5 M long 25 mm diameter rock bolts (yield strength 267 kn) on 1.5 M square grid.

3.8.2 Rock Mass Classification and Rock Pressures

The rocks have been classified by Terzaghi's method as moderately jointed (Class 3) with rock pressure varying from 0 - 2.84 kg/cm^2. By Singh et. al. 1995, the support pressures should be between 0.7-1.0 kg/cm^2.

3.8.3 Support Recommended

The support system recommended correspond to rock bolts 5-6 m long in the roof and 8 m long in the walls at a spacing of 1.0 m alongwith 10 cm thick shotcrete.

3.8.4 Supports Actually Provided

The arch portion of the roof has been reinforced with 20 mm diameter, 5 m long expansion shell bolts at a spacing of 2 x 2 m, subsequently grouted to full length using thick cement grout. Additionally 7.5 cm thick guniting has been carried out over chain link fabric. In view of the occurrence of minor cracks both in the upstream and downstream walls, the side walls were also reinforced with similar bolts of 5 - 7 m length supplemented with 7.5 cm thick guniting and chain link mesh. Fig. 3.10 shows the support system adopted in the Kadamparai power house.

3.9 RAMGANGA PROJECT TUNNELS, U.P.

Ramganga dam, 126 m high earth and boulder fill dam has been constructed across river Ramganga, a tributory of Ganga. Based upon economical studies and feasibility of construction of the Ist stage dam for diversion of floods, two tunnels of 9.45 m internal diameter are constructed in the right abutment of the dam. In order to make maximum utilisation of the tunnels as permanent works after construction of dam is over, eastern tunnel (no. 1) is converted into power tunnel and western tunnel (no. 2) is utilised as outlet works for releasing water for irrigation requirements when power house is closed or for emergency dewatering of the reservoir in case of any damage to the power house.

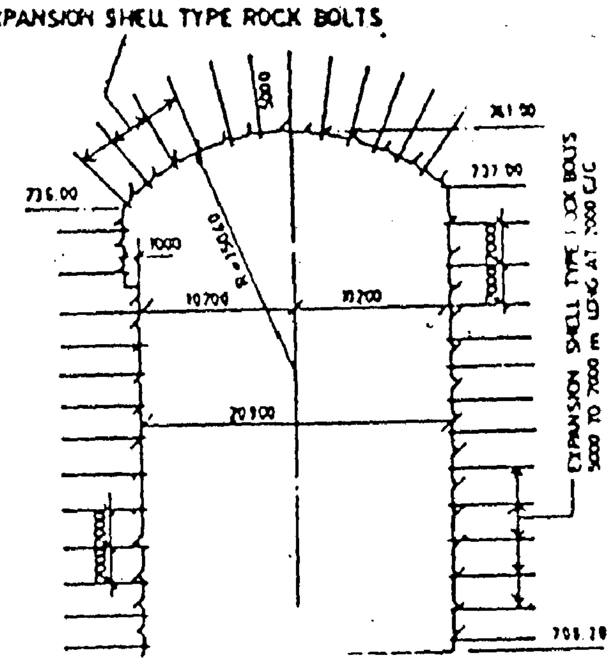


FIG. 3.10 : ROCK SUPPORTS IN KADAMPARAI POWER HOUSE

3.9.1 Geology

Tunnels pass through alternate bands of sand rock and clayshales, the latter covering about one fourth of entire length. Tunnels are excavated in favourable geological set up. Rocks are mostly massive and closely jointed. The bands are highly micaceous and included thin layers and lenses of hard calcified sandstones. Clayshales are green or chocolate coloured with thickness varying from 1.5 m to 2.0 m. Rocks are soft and concretionary in nature. The geological section along the tunnels is shown in Fig. 3.11.

3.9.2 Rock Mass Classification and Rock Pressures

The rocks have been classified under category 5 of Terzaghi's classification with rock pressure varying between 1.75 and 6.74 kg/cm^2 (Gupta et. al., 1968). As per modified classification (Singh et. al. 1995), the support pressure comes out to be 1.0-2.0 kg/cm^2.

3.9.3 Supports Recommended

The rock supports have been worked out to be rock bolts of 3.25 m length spaced at 1.0 m centres (minimum yield strength 150 KN) and 5 cm thick fibre reinforced shotcrete.

3.9.4 Supports Actually Provided

Full circle ribs of 10.993 m outer diameter made from R.S. joists 300 mm x 140 mm were used to support the rock mass. The spacing of ribs vary from 0.61 to 1.2 m depending upon rock conditions. In tunnel no. 1 which was to be converted to power tunnel, spacing of ribs were kept as 0.61 m in all reaches except portals. In tunnel no. 2, rib spacing is 0.61 to 1.2 m except in reaches near portal having inadequate cover and the plug and valve chamber reaches where it has been reduced to 0.305 m. The support measures adopted in the tunnels are shown in Fig. 3.12.

3.10 NARMADA SAGAR PROJECT, M.P.

Narmada Sagar Project on river Narmada near Punasa comprise of 92 m high concrete gravity dam across river Narmada to divert 2040 cusec of water through 40 to 55 m deep, 450 m

3.6 YAMUNA HYDRO-ELECTRIC SCHEME STAGE II (CHHIBRO POWER HOUSE) U.P.

Yamuna Hydro-electric Scheme Stage II in Dehradun district of U.P., envisages development of the power potential of river Tons,

a tributory of river Yamuna at Ichhari and its outfal at Dakpathar. The total available drop of 186 m is utilised for power generation in two stages. Part I utilises a drop of 124 m by the construction of dam at Ichhari, for diverting water through a 6.3 Km long tunnel to an underground power house at Chhibro (first underground power house) with installed capacity of 4 x 60 MW.

The underground power house at Chhibro comprises a network of cavities for housing the machines, transformers, turbine inlet valves, control room and to serve as various operating galleries and water conductor system to feed the Part II of the project. The main cavity is 113.2 M long x 18.35 M wide x 32.5 M high and has a circular roof and vertical sides.

3.6.1 Geology

The power house cavity is located in a stratified limestone band 25 m thick and 200 m horizontal thickness with minor or thinly bedded slate bands. The rock is closely jointed with numerous shear zones ranging from 2 to 50 cm thick and nearly parallel to the bedding. A major shear zone lies at a minimum depth of 10 m below the lowest draft tube level in the power house cavity. The formations dip at about 45° towards N150°W to N29°E. The cavity is aligned parallel to the strike of rock formations with cover ranging from 208 m over the transformer hall. Fig. 3.6 shows the geological section of the power house.

3.6.2 Rock Mass Classification and Rock Pressures

The rock at the site may be classified as stratified limestone closely jointed. The support pressure as per Terzaghi's classification works out to 4.8-5.1 kg/cm^2 (very blocky and seamy rocks, class 5) and by Singh et. al. 1995, 1-2.0 kg/cm^2.

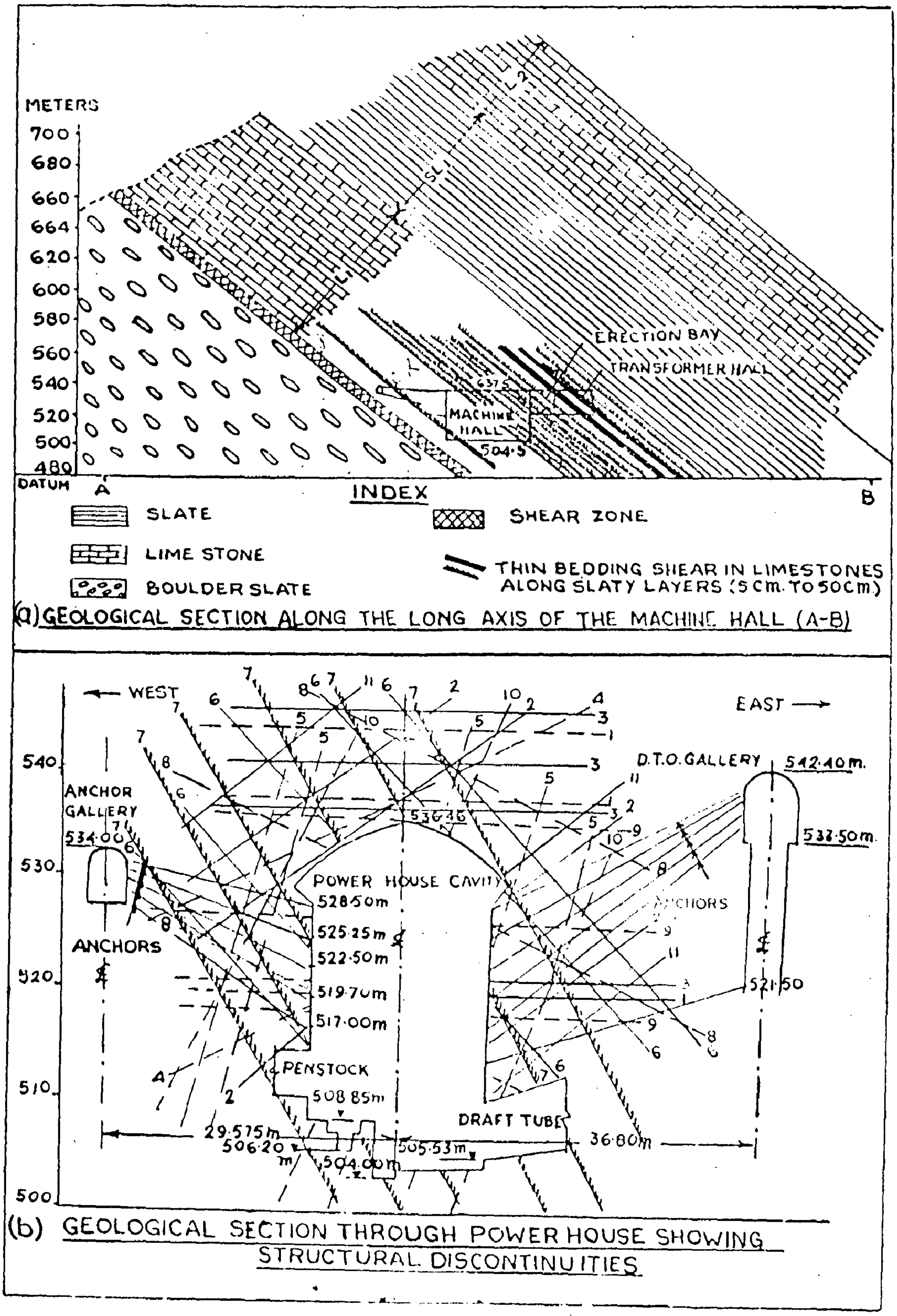


FIG. 3.6 : GEOLOGICAL SECTION OF CHHIBRO POWER HOUSE

3.6.3 Support Recommended

The support requirements have been worked out as 25 mm diameter rock bolts of 5 m length in the roof and 8 m in the walls at a spacing of 1.0 m with 10 cm thick shotcrete.

3.6.4 Support Actually Provided

The roof of the power house has been supported by steel arches and the walls have been supported by 350 prestressed anchors of average length 23.5 M of 600 KN capacity at 2-5 m spacing and reinforced shotcrete 7.5 Cm thick has been used where found necessary. In the roof R.S. Joists of 250 x 125 mm with cover plates of 250 x 20 mm at top and 150 x 20 mm at bottom spaced at 25 cm centres have been used as rock support. Backfill concrete of M-150 strength has been used. Fig. 3.7 shows the roof and wall supporting arrangements in the power house.

3.7 CHAMERA H.E. PROJECT, H.P.

The project is located near Chaurah village in Chamba district of Himachal Pradesh on approximate latitude N32° 36' and longitude E75° 56'. The project is linked to the nearest rail head Pathankot by a 97 km road. The underground power house of 112 m (L) x 24 m (W) x 37 m (H) size is located at Khairi. The total installed capacity of the project is 540 MW (3 nos of francis turbines of 180 MW each). The project comprise of an 9.5 M finished diameter head race tunnel of 6414 m long.

3.7.1 Geology

The power house complex comprising of two caverns viz. The machine hall and transformer hall and other ancillary components have been excavated in metamorphosed andesite basalt. Rock mass in the power house may be categorised as blocky to foliated and intersected by five sets of discontinuities of different orientations, predominant being foliation joints and shears. Foliation joints are continuous and undulating and generally moderate to closely spaced. Most of the joint sets have low persistence. Water seepage in caverns during excavation was negligible. Geological section of the project is shown in Fig. 3.8.

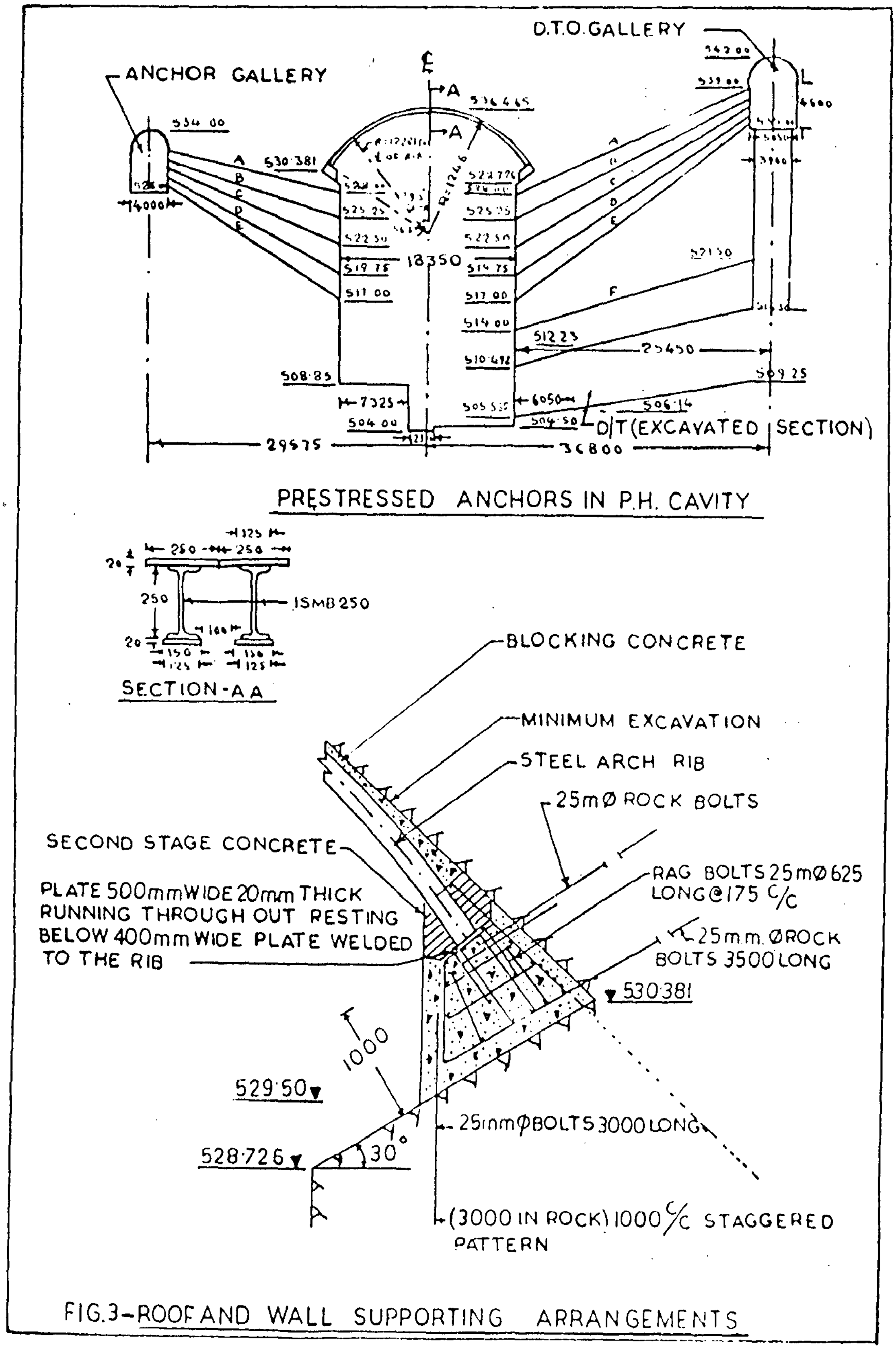


FIG. 3.7 : ROOF AND WALL SUPPORTS IN CHHIBRO POWER HOUSE

3.7.2 Rock Mass Classification and Rock Pressures

The rock mass classification is as per Table 3.7.

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Method of classification	Rock type metamorphosed andesite basalt
Terzaghi's classification	Blocky to foliated, class 4
Supp. Pr., kg/cm^2	1.65 to 5.87
Bieniawski's RMR(Sharma et. al.1994)	50, Fair rock mass
Supp. Pr., kg/cm^2	3.3
Barton's Q value	1.95
Supp. Pr.,kg/cm^2, P_{roof}	1.07
P_{wall}	0.62

3.7.3 Supports Recommended

As per RMR method the required support system conform to systematic 20 mm diameter rock bolts of 4m length at a spacing of 1.5 to 2.0 m in the crown and walls with wire mesh in the crown alongwith 5-10 cm thick shotcrete layer in the crown and 3 cm in sides. However, Q system recommends rock bolts of 6 m length in the roof and 8 m in the walls at a grid spacing of 1.9 m with 10 cm thick fibre reinforced shotcrete layer.

3.7.4 Support Actually Provided

To support the rock mass, flexible support system consisting of combination of rock bolts, anchors and shotcrete was considered prudent with regular monitoring of excavated section by instrumentation. Based on this design approach, the following two support systems were adopted.

- 7.5 M long 25 mm diameter rock bolts (yield strength 267 kn) on 1.5 M square grid.

- 6.0 M long, 25 m diameter rock bolts (yield strength 204 kn), on a 1.5 M square grid as primary support and longer, 51 mm diameter hollow core anchors, 10.5 M long of 843 kn on a 4.5 M square grid as the secondary support.

The depth of anchors was sufficient to ensure formation of the required zone of compression. Fig. 3.9 shows supports adopted.

3.8 KADAMPARAI PUMPED STORAGE H.E. PROJECT, TAMIL NADU

This project undertaken by Tamil Nadu Electricity Board in South India, has been designed to meet the peaking requirements of Tamil Nadu Grid. The project envisages construction of a dam across the Kadamparai river for forming the upper reservoir and utilisation of the existing Upper Aliyar Reservoir as a tail pool. The project has an underground power house of 128.5 m (L) x 20.9 m (W) x 38.0 m (H) size of 400 MW installed capacity.

3.8.1 Geology

The area encompassing the Kadamparai Scheme is occupied mainly by biotite gneiss intruded by pegmatites. The gneisses are folded but the folds are generally tight along the water conductor system. From the Kadamparai dam to the tail race tunnel outlet, granite gneiss with veins of pegmatite are met with. The general foliation of the gneisses varies from NNE - SSW direction, with dips ranging from 60° to 80° in the easterly direction. There are three sets of joints in the gneisses.

Set No.	Strike of Joint Set	Dip
Set 1	N25°W - S25°E	10°-20° in S65°
Set 2	N75°E - S75°W	Vertical
Set 3	NE - SW	70° towards SE

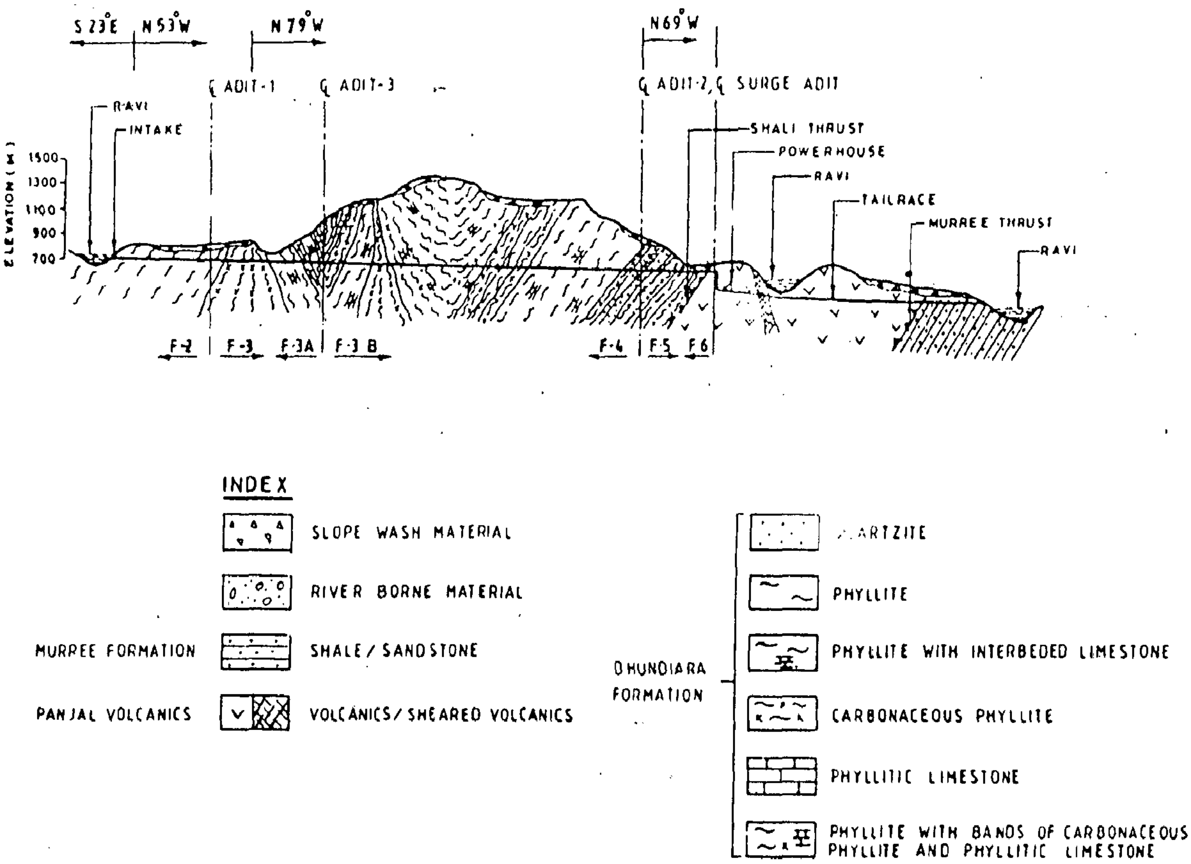


FIG. 3.8 : GEOLOGICAL SECTION OF CHAMERA PROJECT

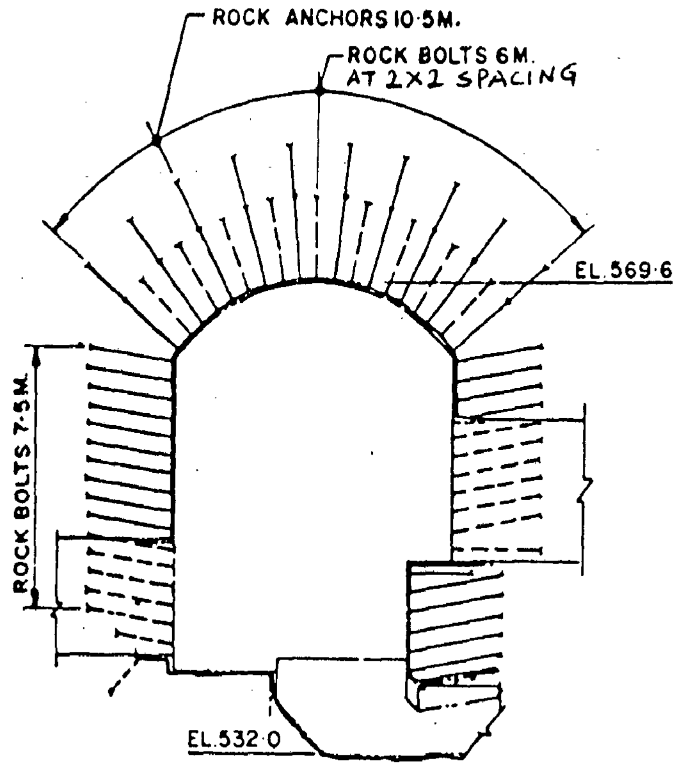


FIG. 3.9 : ROCK REINFORCEMENT IN CHAMERA POWER HOUSE

3.8.2 Rock Mass Classification and Rock Pressures

The rocks have been classified by Terzaghi's method as moderately jointed (Class 3) with rock pressure varying from 0 - 2.84 kg/cm^2. By Singh et. al. 1995, the support pressures should be between 0.7-1.0 kg/cm^2.

3.8.3 Support Recommended

The support system recommended correspond to rock bolts 5-6 m long in the roof and 8 m long in the walls at a spacing of 1.0 m alongwith 10 cm thick shotcrete.

3.8.4 Supports Actually Provided

The arch portion of the roof has been reinforced with 20 mm diameter, 5 m long expansion shell bolts at a spacing of 2 x 2 m, subsequently grouted to full length using thick cement grout. Additionally 7.5 cm thick guniting has been carried out over chain link fabric. In view of the occurrence of minor cracks both in the upstream and downstream walls, the side walls were also reinforced with similar bolts of 5 - 7 m length supplemented with 7.5 cm thick guniting and chain link mesh. Fig. 3.10 shows the support system adopted in the Kadamparai power house.

3.9 RAMGANGA PROJECT TUNNELS, U.P.

Ramganga dam, 126 m high earth and boulder fill dam has been constructed across river Ramganga, a tributory of Ganga. Based upon economical studies and feasibility of construction of the Ist stage dam for diversion of floods, two tunnels of 9.45 m internal diameter are constructed in the right abutment of the dam. In order to make maximum utilisation of the tunnels as permanent works after construction of dam is over, eastern tunnel (no. 1) is converted into power tunnel and western tunnel (no. 2) is utilised as outlet works for releasing water for irrigation requirements when power house is closed or for emergency dewatering of the reservoir in case of any damage to the power house.

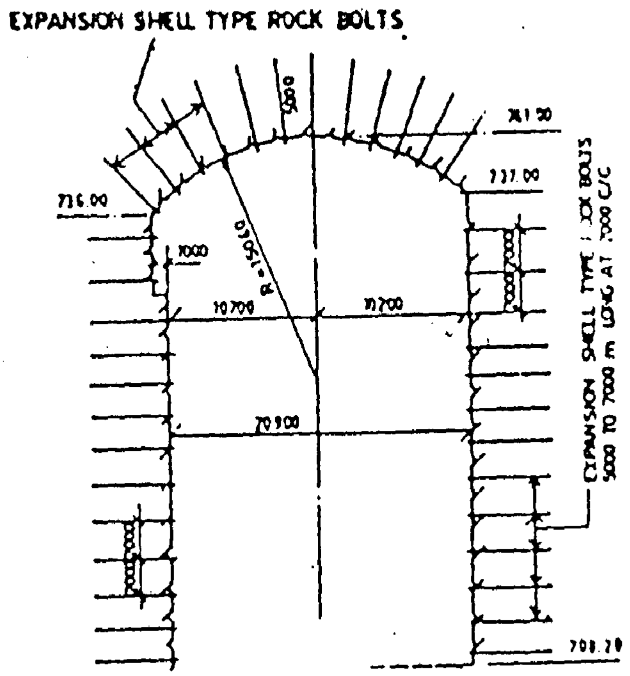


FIG. 3.10 : ROCK SUPPORTS IN KADAMPARAI POWER HOUSE

3.9.1 Geology

Tunnels pass through alternate bands of sand rock and clayshales, the latter covering about one fourth of entire length. Tunnels are excavated in favourable geological set up. Rocks are mostly massive and closely jointed. The bands are highly micaceous and included thin layers and lenses of hard calcified sandstones. Clayshales are green or chocolate coloured with thickness varying from 1.5 m to 2.0 m. Rocks are soft and concretionary in nature. The geological section along the tunnels is shown in Fig. 3.11.

3.9.2 Rock Mass Classification and Rock Pressures

The rocks have been classified under category 5 of Terzaghi's classification with rock pressure varying between 1.75 and 6.74 kg/cm^2 (Gupta et. al., 1968). As per modified classification (Singh et. al. 1995), the support pressure comes out to be 1.0-2.0 kg/cm^2.

3.9.3 Supports Recommended

The rock supports have been worked out to be rock bolts of 3.25 m length spaced at 1.0 m centres (minimum yield strength 150 KN) and 5 cm thick fibre reinforced shotcrete.

3.9.4 Supports Actually Provided

Full circle ribs of 10.993 m outer diameter made from R.S. joists 300 mm x 140 mm were used to support the rock mass. The spacing of ribs vary from 0.61 to 1.2 m depending upon rock conditions. In tunnel no. 1 which was to be converted to power tunnel, spacing of ribs were kept as 0.61 m in all reaches except portals. In tunnel no. 2, rib spacing is 0.61 to 1.2 m except in reaches near portal having inadequate cover and the plug and valve chamber reaches where it has been reduced to 0.305 m. The support measures adopted in the tunnels are shown in Fig. 3.12.

3.10 NARMADA SAGAR PROJECT, M.P.

Narmada Sagar Project on river Narmada near Punasa comprise of 92 m high concrete gravity dam across river Narmada to divert 2040 cusec of water through 40 to 55 m deep, 450 m

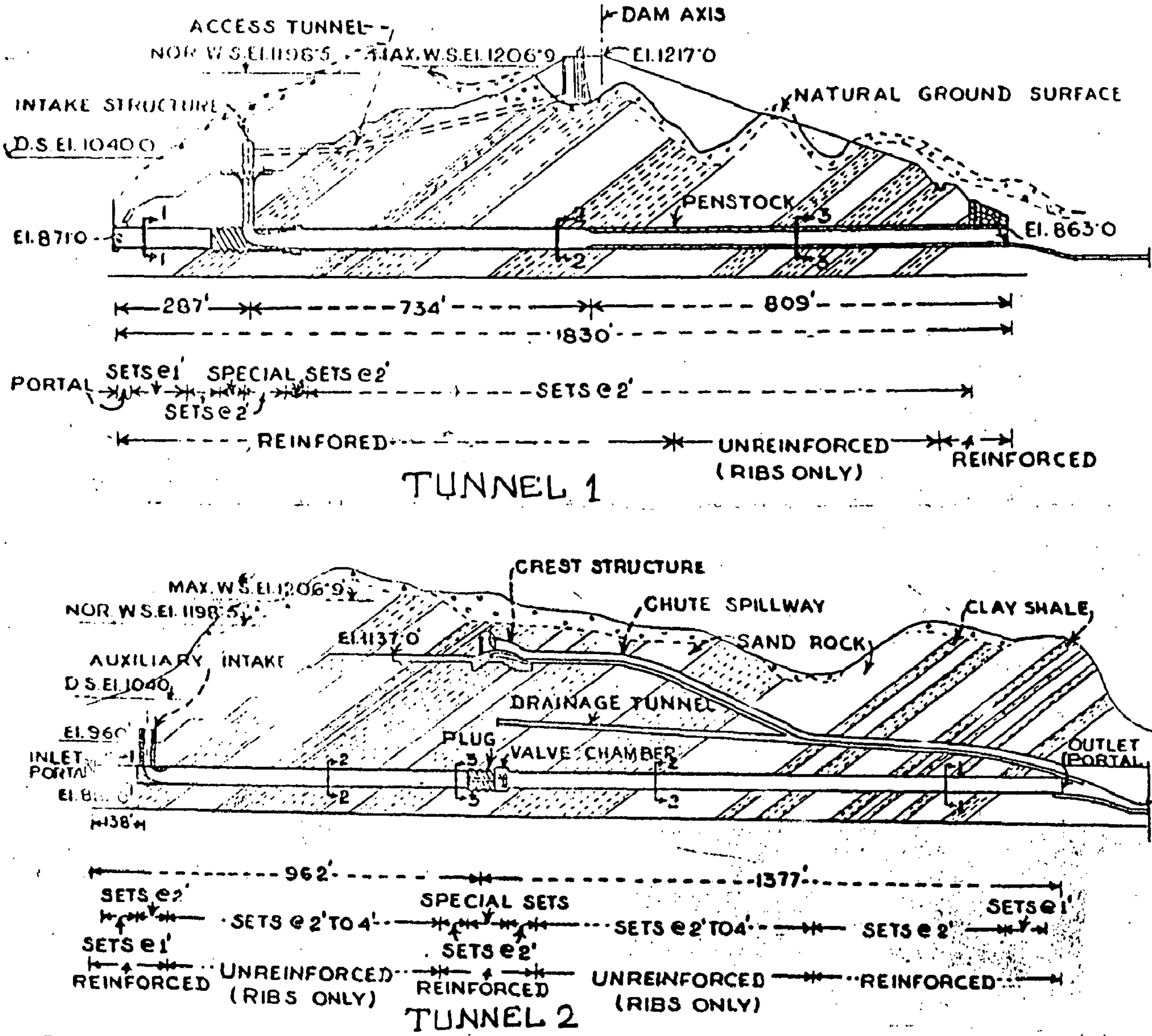


FIG. 3.11 : GEOLOGICAL L-SECTION OF RAMGANGA PROJECT TUNNELS

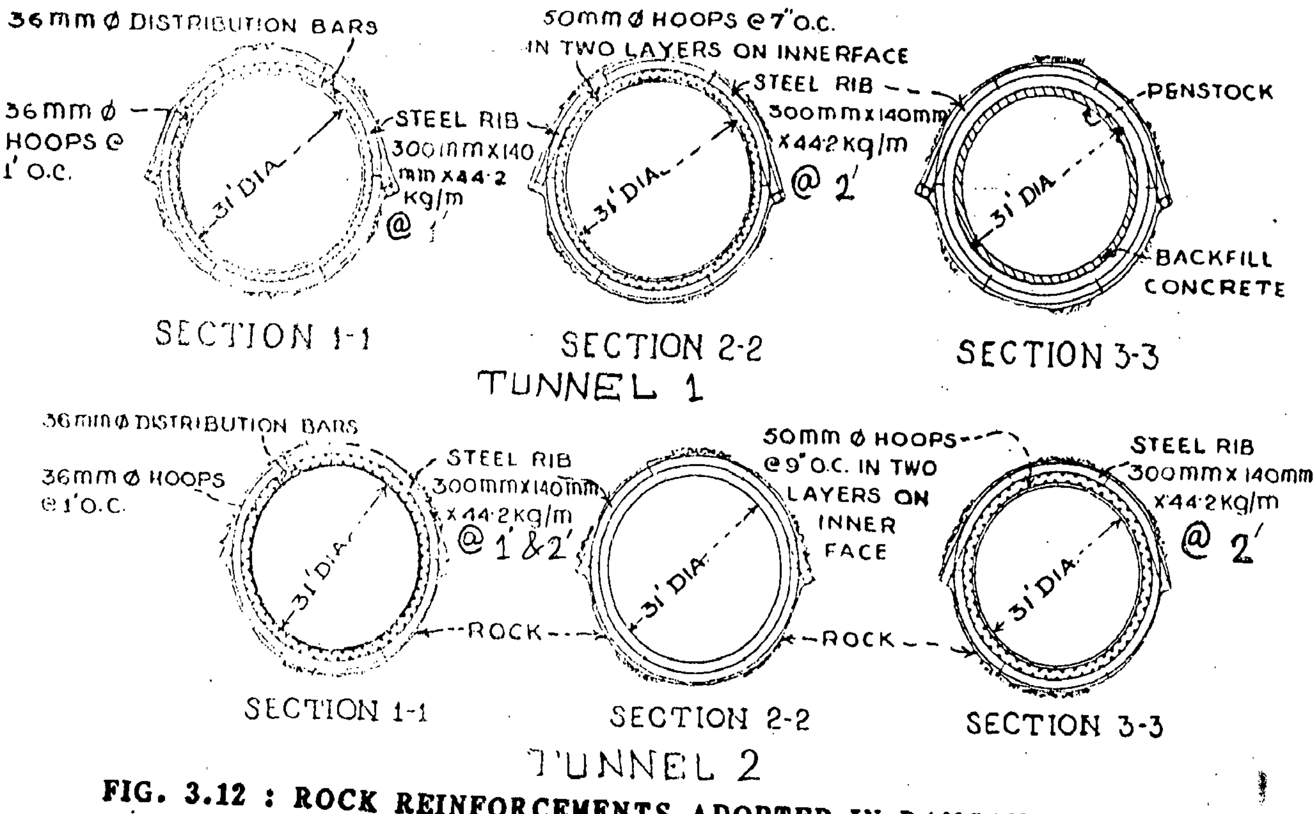


FIG. 3.12 : ROCK REINFORCEMENTS ADOPTED IN RAMGANGA TUNNELS

long head race channel to feed eight pressure shafts of 8 m finished (9 m excavated) diameter to generate 8x125 = 1000 MW of power in a 55 m deep pit power house and release the tail water back into the Narmada river through a 25-48 m deep 865 m long tail channel.

The rock cover over the pressure shafts varies between 40 and 50 m in about 130 m length and about 4 m on the power house end for a length of about 12 m. The 142 m long shafts were explored by drillholes and an adit.

3.10.1 Geology

The tunnelling media is quartz arenites (quartzites) and ferruginous fine grained sandstones with the intercalated layers of silt/clay stones (Fig. 3.13). The pressure shafts are aligned parallel to the general trend of the rocks in N50°E to S50°W direction. The beds dip by 20° to 35° towards NNW i.e. towards right abutment with occasional dips of 40° due to local warping between Pressure shafts No 5 and 8.

The rock mass has been characterised in the three categories as follows:

Cat 1. Quartz Arenites (Quartzites)

Cat 2. Ferruginous Sand stones

Cat 3. Ferruginous Silt/Clay stones

3.10.2 Rock Mass Classification and Rock Pressures

The rocks have been classified as per the available methods and the support pressures are as per Table 3.8.

3.10.3 Support Recommended

The supports recommended for various categories of rock are presented in Table 3.9.

3.10.4 Support Actually Provided

Rock bolts of 20 mm diameter expansion shell type 4-5 m long at 2 m spacing of variable depths restricting the bottom level to about 0.5 m above crown level have been used to support the rock. Bolts are grouted and tensioned to 60% of yield strength of pull out test i.e. to about 8-10 tonnes. Permanent steel half supports (ISMB- 300) at 1 m spacing in the crown portion by

TABLE 3.8 : ROCK CLASS AND SUPPORT PRESSURES AT PRESSURE SHAFTS OF NARMADA SAGAR PROJECT, M.P. (MOHD., J. AHMAD, 1996)

Method of rock classification	Rock Type		
	Quartz Arenites (Quartzites) Cat. I	Ferrugineous Sandstone, Cat. II	Ferrugineous Silt/Clay stone Cat. III
Joint Spacing	0.1 to 0.7	0.1 to 0.3	0.1 to 0.2
Joint Vol. Count	Between 2 & 8	Between 15 & 20	>40
Comp.Strength, kg/cm^2	622 - 143	496 - 762	126 - 203
Tensile Strength, kg/cm^2	99 - 148	30 - 73	43 - 64
Rock Class By Terzaghi	Widely to Moderately Jointed (Class 3)	Closely to moderately jointed (Class 4)	Closely to very closely jointed (Class 5)
Support Pr. (kg/cm^2)	0-1.2	0.6 to 1.7	1.7 to 5.2
Deere's Method, RQD[a]	95 (Excellent)	57 (Fair)	<25 (Very Poor)
Supp. Pr. (kg/cm^2)	0-0.7	1.43-3.1	4.77-6.68
Bieniawski's Rock Mass Rating	75 (Av.) Good to very good rock	47 (Av.) Fair rock	20-26 (23), Poor rock
Supp. Pr. (kg/cm^2)	0.6	1.25	1.83
Barton's Q Values	21.1	9.5	0.83
Supp.Pr.,kg/cm^2,P_{roof}	0.42	0.54	1.64
P_{wall}	0.25	0.40	1.21

[a] RQD has been determined using the Palmstrom's relation (1975).

TABLE 3.9 : SUPPORTS RECOMMENDED BY DIFFERENT METHODS AT NARMADA PROJECT SAGAR PRESSURE TUNNELS

Method of rock classification	Support Provided		
	Quartzite arenite	Ferrugineous sandstone	Ferrugineous silt/clay stone
Deere's Method	Local application of shotcrete 5-7.5 cm	Rock bolts at 0.9-1.8 m + shotcrete 10 cm or more.	Rock bolts at 0.9 m spacing + shotcrete 15 cm or more. Combine with medium to heavy sets.
Bieniawski's RMR method	20 mm dia, 3m long, 2.5 spaced locally bolts with occasional wire mesh.	20 mm dia, Systematic bolts 4m long, 1.5-2.0 m spaced with wire mesh in crown + shotcrete 5-10 cm.	20 mm dia, systematic bolts, 4-5 m long, 1-1.5 m spaced with wire mesh + shotcrete 10-15 cm thick.
Barton's Q system	20mm dia,3m long untensioned bolts & 1.75 m spacing + shotcrete 2-3 cm thick	20 mm dia,3 m long bolts (untensioned grouted) at 1.3 m spacing + shotcrete 2-3 cm thick	20 mm dia,3m long bolts (tensioned grouted) at 1.0 m spacing + fibre reinforced shotcrete 9 cm thick

cutting haunches at the spring level with backfill concrete. Fig. 3.14 shows the class of rock and support system.

3.11 GIRI PROJECT HEAD RACE TUNNEL, H.P.

Giri Hydel Project is situated in Himachal Pradesh across river Giri having an installed capacity of 60 MW (2 X 30 MW each). Besides this there is 160 m long barrage and an intake regulator. The water conductor system of the project comprise of a concrete lined tunnel 7.12 km long with a circular finished diameter of 3.6 m and passes under the ridge separating the Giri and Bata valleys.

3.11.1 Geology

The tunnel passes through various types of rocks namely slate/phylites interbedded with quartzites, shales of various shades, limestone conglomerates and sandstones of various grades. The most important feature from the engineering geology view point was the occurrence of three thrusts lying in the close proximity to one another. The tunnel crosses two major regional thrusts (viz. Krol and Nahan) which were considered to be the most problematic zones for tunnelling operations.

Along the tunnel alignment, the strata changes to claystones and siltstones which are highly jointed and deteriorate on saturation with water. The material in the vicinity of the faults is highly saturated, soft and plastic. However near the outlet of the tunnel the strata generally comprise of sandstones and siltstones. The rock is jointed but generally compact except when saturated and claystones bands are present. The geological section along the tunnel alignment is shown in Fig. 3.15.

3.11.2 Rock Mass Classification and Rock Pressures

The support pressures are presented in Table 3.10.

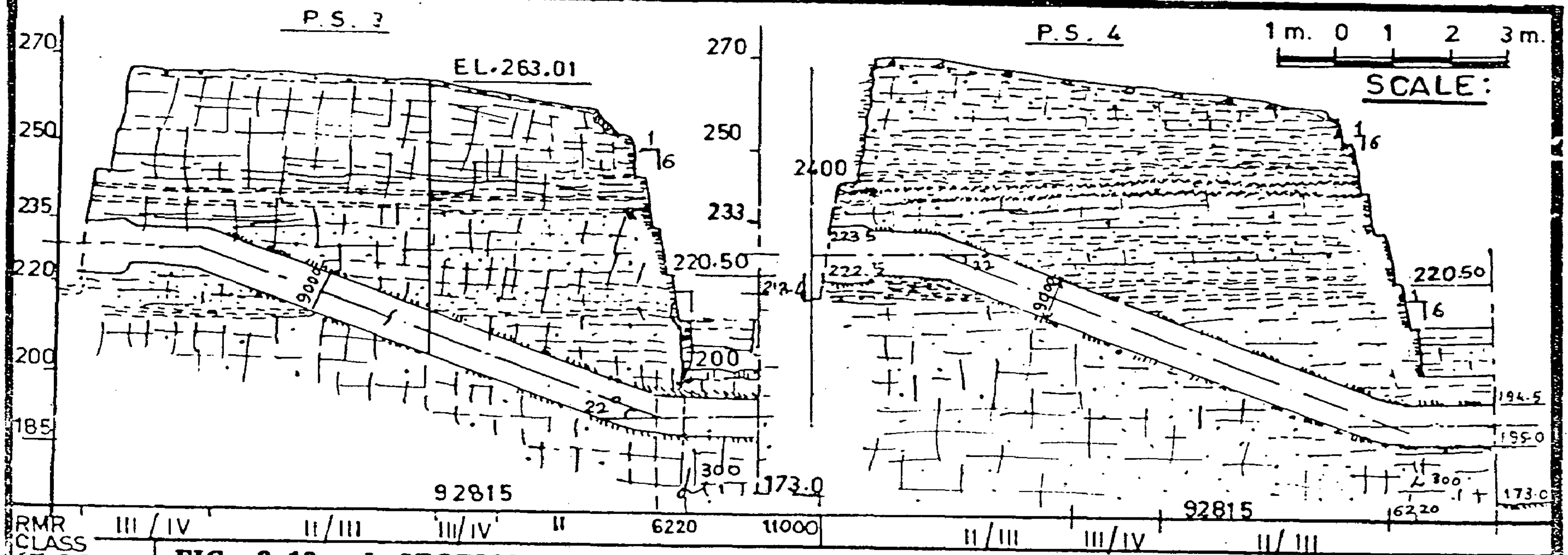


FIG. 3.13 : L-SECTION OF PRESSURE TUNNELS OF NARMADA SAGAR PROJECT

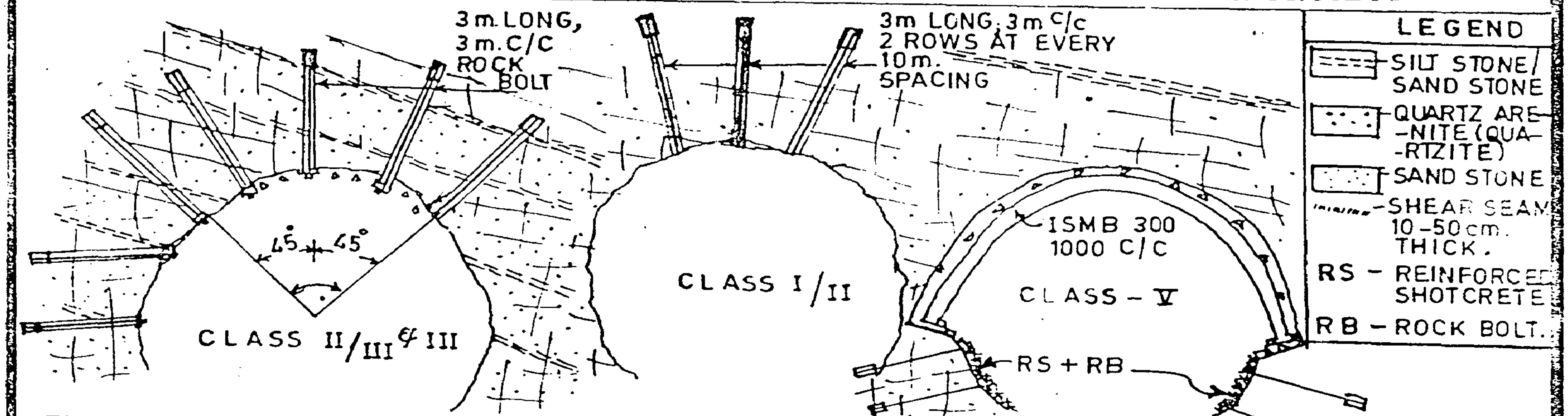


FIG. 3.14 : SECTIONS SHOWING ACTUAL CLASS OF ROCK AND SUPPORT SYSTEM

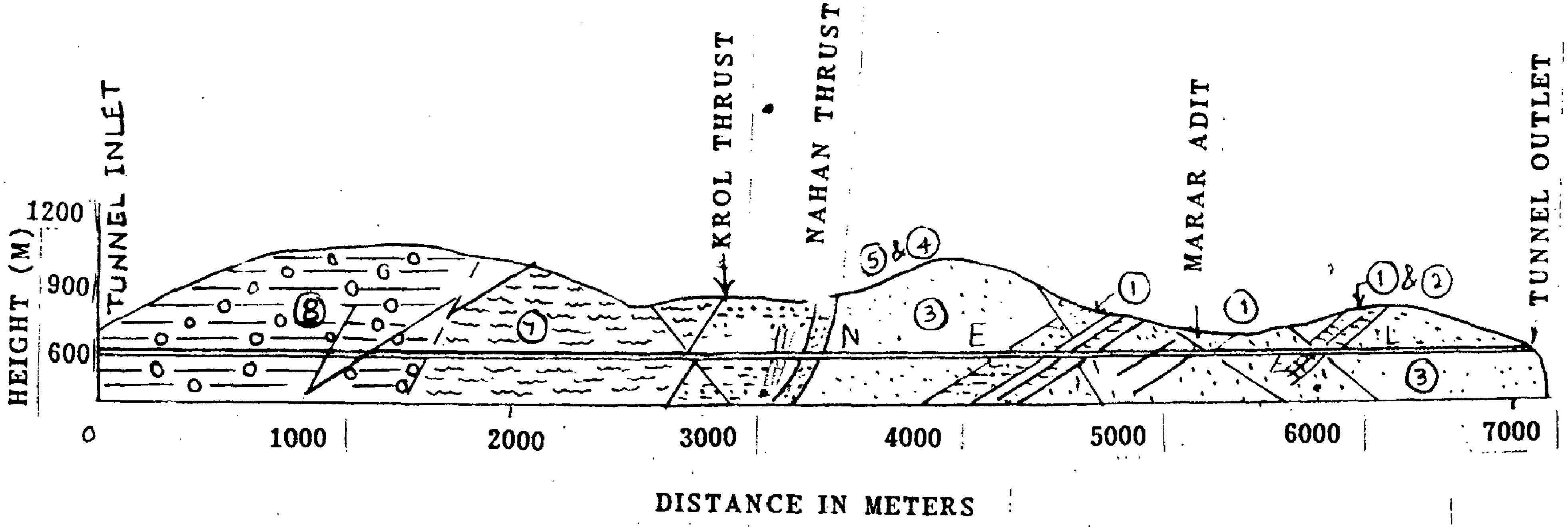


FIG. 3.15 : GEOLOGICAL L-SECTION OF GIRI PROJECT HEAD RACE TUNNEL

TABLE 3.10 : ROCK CLASS AND SUPPORT PRESSURES AT GIRI PROJECT HEAD RACE TUNNEL, H.P.

Method of Rock Classification	Rock Type	
	Slates	Phylites
Rock Class By Terzaghi	Very Blocky and Seamy (Mild Squeezing) Class 7	Crushed Phylites, (Highly Squeezing) Class 8
Supp.Pr., kg/cm^2	2.4 - 4.6	6.1 - 11.6
Deere's Method, RQD	5 - 25	5 - 25
Supp.Pr.,kg/cm^2	1.88 to 3.12	60.75 (upto 75 m rock)
Bieniawski's Rock Mass Rating[a]	38 (Poor rock)	25 (Very poor rock)
Supp.Pr.,kg/cm^2	0.7	0.84
Barton's Q Values (Jethwa et. al 1982)	0.51	0.12
Supp. Pr., kg/cm^2	2.4	3.4

[a] : RMR has been assessed from Q values using the relation

3.11.3 Support Recommended

As per Deere's RQD method, either very heavy circular steel sets at 0.6 m centers or the rock bolts at 0.6 to 0.9 m spacing with heavy steel sets, are required. RMR method recommends the use of either medium to heavy steel sets spaced at 0.75 m with steel lagging and fore poling if required and invert to be closed, or the rock bolts 20 mm diameter, 5-6 m long spaced at 1-1.5 m in crown and walls with wire mesh, bolt invert and 15-20 cm thick shotcrete. Q system recommends 3 m long bolts at 1.5 m spacing for slates and 1.3 m for phylites with 10 cm thick fibre reinforced shotcrete.

3.11.4 Supports Actually Provided

Horse shoe shaped steel sets with bottom struts have been used to support the rock. Two steel sections viz. 150 x 80 mm and 150 x 150 mm have been used with varying spacing as shown in Table 3.11.

TABLE 3.11 : SUPPORT PROVIDED AT GIRI PROJECT TUNNEL

Rib Section (mm)	Spacing (m)	Capacity in Tonnes (fibre stress = 2500 kg/cm^2)
150 x 80	1.0	65
150 x 80	0.5	65
150 x 80	0.33	65
150 x 150	1.0	100
150 x 150	0.5	100

3.12 URI PROJECT, J. & K.

The Uri Hydroelectric project is situated 75 km west of Srinagar in Baramulla District (Fig. 3.16a). The project is run of the river scheme with 20 m high barrage across Jhelum river near Village Bunyar. The project comprise of 8.4 m finished dia horseshoe shaped hear race tunnel, 10.5 km long, 22 m dia & 75 m high underground surge tank, 5m dia twin vertical pressure shafts, an underground power house of 4x120 MW capacity operating under a gross head of 260m and finally a 2 km long tail race which will carry the water back to the river Jhelum near Uri town.

3.12.1 Geology

The overall rock mass is fairly hard but intensely folded, faulted/sheared leading to various degree of fracturisation. The general foliation trend vary from N60°E - S60°W to EW with 70 to 90° dips mostly in northerly direction. The quartzitic schist estimated to be in about 50% of the tunnel length were found to be hard, compact with some softer and very closely foliated phylitic zones in between. On the other hand the Panjal Volcanoes found in the rest part of the tunnel are greenish grey and well foliated with more frequency of schistose zones. Bieniawski's rock mass classification has been slightly modified in categorising the quality of prevailing rocks (Sharma et. al 1995). Geological section of the tunnel is shown in Fig. 3.16b.

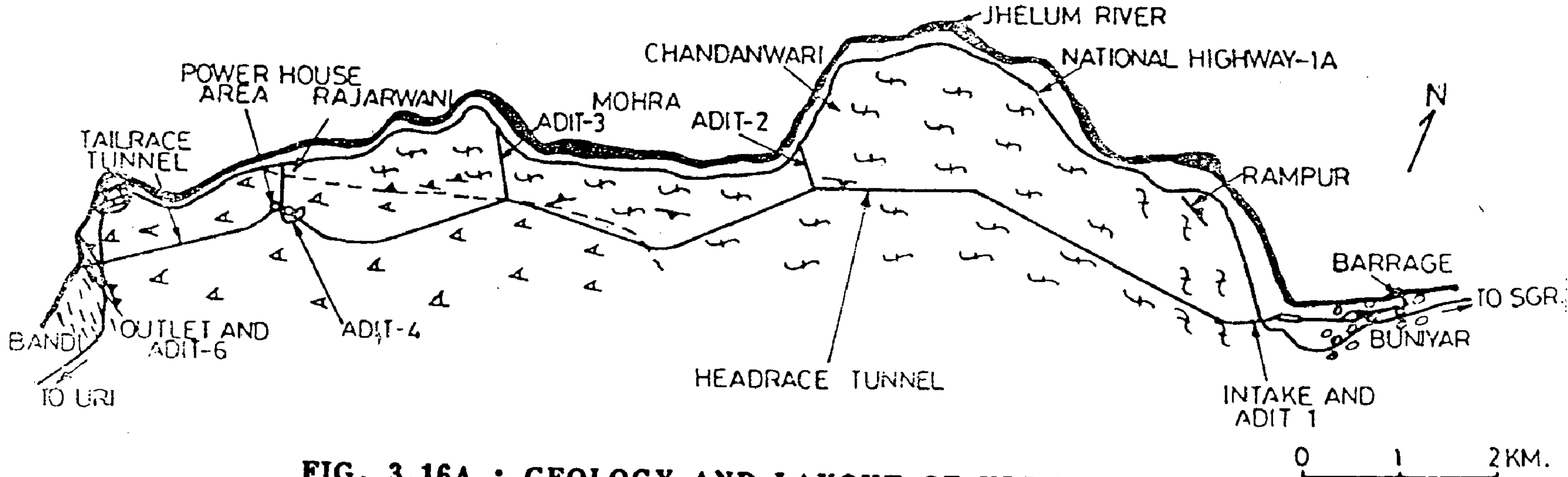


FIG. 3.16A : GEOLOGY AND LAYOUT OF URI PROJECT

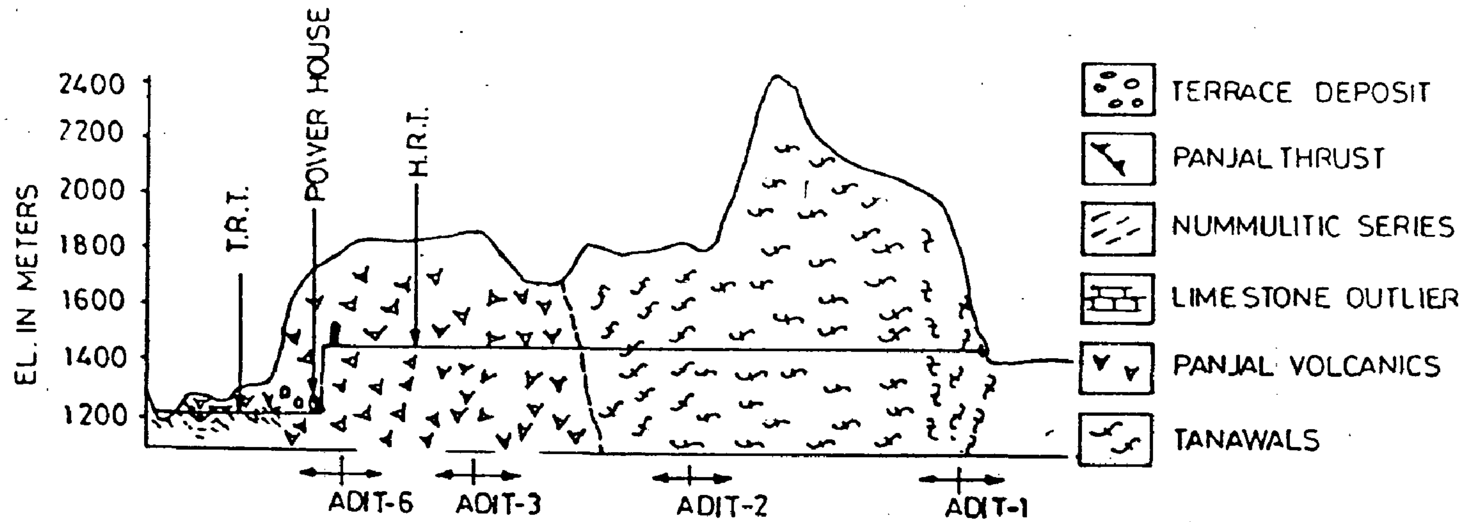


FIG. 3.16B : GEOLOGICAL SECTION ALONG H.R.T. AND T.R.T.

3.12.2 Rock Mass Classification and Rock Pressures

The rock loads and the classification of rocks are presented in the Tables 3.12 and 3.13.

3.12.3 Supports Recommended

RMR methods recommends 4 m long systematic bolts, at 1.5 to 2.0 m spacing with wire mesh and 5-10 cm thick shotcrete for category I rock whereas for category II, same rock bolts of 4-5 m length at 1-1.5 m spacing with 10 to 15 cm thick shotcrete.

As per Q system the use of 3.5 m long systematic bolts at 2.0 m and 1.5 m spacing for categories I and II, respectively, are suggested. In addition to the bolts the shotcrete of 5 cm thickness for category I and fibre reinforced shotcrete of 12 cm thickness for the other category rock is also required.

3.12.4 Support Actually Provided

The support system provided in the head race tunnel correspond to both the RMR and Q systems. The supports provided in the head race tunnel are as follows.

Cat. I 3-4 m long bolts (grouted) at 2m spacing + fibre reinforced shotcrete 6 cm thick.

Cat. II 4 m long, bolts at 1.5 m spacing and 10 cm thick fibre reinforced shotcrete.

The special support system provided in the head race tunnel is shown in Fig. 3.17.

3.13 BODHGHAT HYDEL PROJECT, MADHYA PRADESH

The water conductor system of Bodhghat Hydel Project in Madhya Pradesh, consists of a 13 m dia and 2.8 km long head race tunnel (HRT), 450 m long penstocks and a 40 m deep power house pit. The head race tunnel cuts the transverse hills within the loop of Indravati River and it intersects high ridges and saddles trending along NW-SE direction. The penstocks are located in south western slopes of the hill ranges with the power house pit away from the toe of the hill in a gently undulating terrace.

TABLE 3.12 : ROCK CLASSIFICATION AT URI PROJECT HEAD RACE TUNNEL, J. & K.(AFTER SHARMA ET. AL. (1995)

Rock Type	Tunnel Length, %	Rock Mass	RMR Value	Description
I	1.8	Good Rock	>61	Massive Blocky, Partly Foliated Competent Hard Rock
IIA	17.8	Fair Rock	51-60	Jointed,Fractured,Thinly Foliated, Competent and Hard. Foliation Perpendicular to Tunnel
IIB	60	Fair Rock	41-50	Rock Mass as That of IIA but Foliation Parallel
IIB	6.6	Fair Rock (High Stress)	41-50	As That of IIB with High Stress
III	13	Poor Rock	21-40	Fractured or Thinly Foliated of Low to Medium Strength
IV	0.8	Very Poor Rock	<20	Crushed or Shattered with Clay & Gauge Material or Weathered Rock

Hence as per Bieniawski we can safely divide the rock into two categories viz. III and IV.

TABLE 3.13 : ROCK CLASS AND SUPPORT PRESSURES AT URI PROJECT HEAD RACE TUNNEL, J. & K.

Method of rock classification	Rock Type	
	Cat. I	Cat. II
Rock Class By Terzaghi	Moderately blocky and seamy, Class 4	Completely crushed but chemically inert, Class 6
Supp.Pr., kg/cm^2	0.6 - 1.7	5.2
Bieniawski's Rock Mass Rating	41-60 (Av. 50) Class III, fair Rock	21-40 (Av. 30) Class IV, Poor rock
Supp.Pr.,kg/cm^2	0.95-1.4 (Av. 1.2)	1.43-1.90 (Av. 1.67)
Barton's Q Values*	2.0	0.21
Supp. Pr., kg/cm^2, P_{roof}	1.06	2.24

* Q values have been assessed from the RMR.

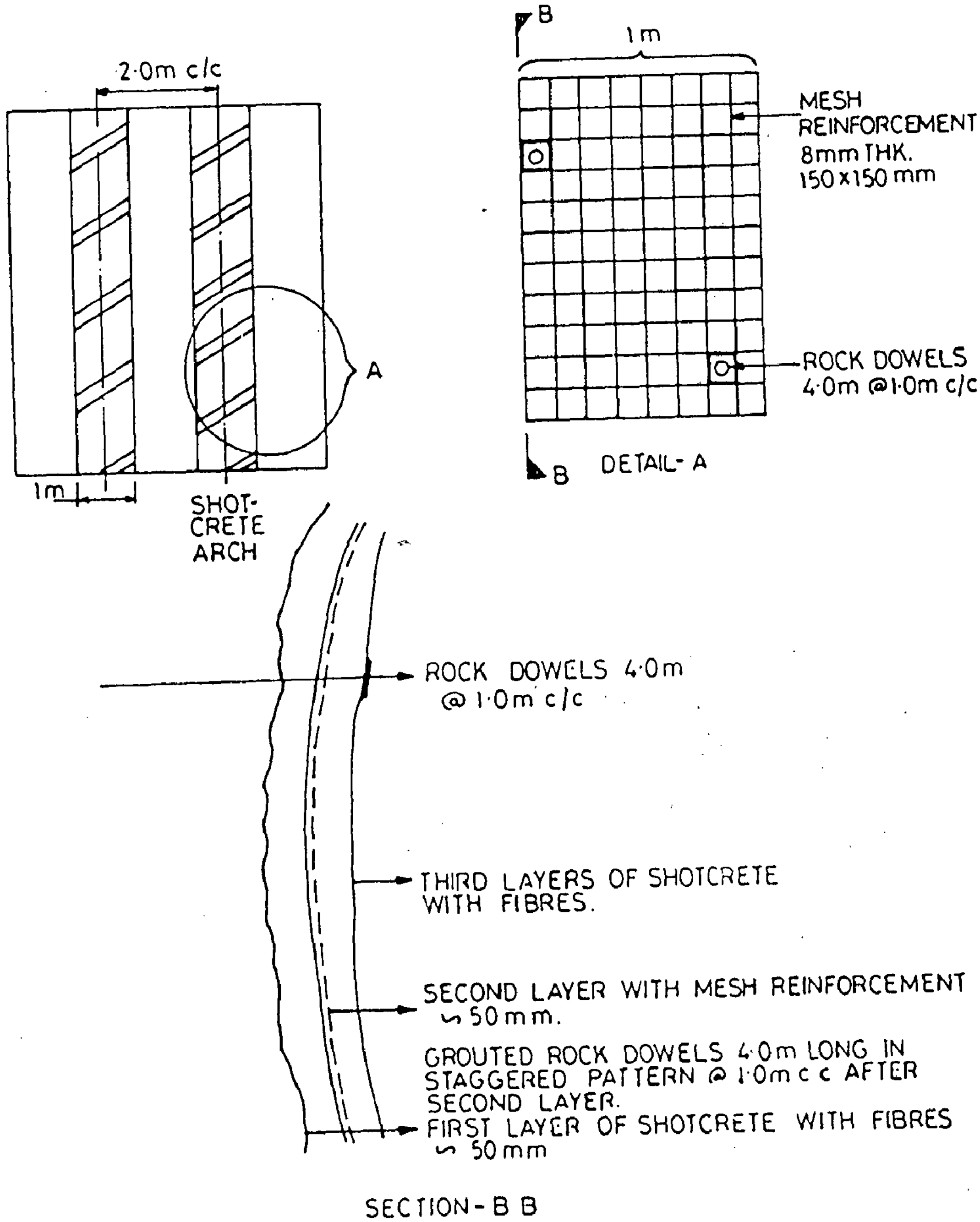


FIG. 3.17 : SPECIAL SUPPORT MEASURES BY SHOTCRETE ARCHES IN CRUSHED OR GRAPHIC ZONES IN URI PROJECT HEAD RACE TUNNEL

3.13.1 Geology

The area is occupied by tightly folded sequence of metamorphic rocks - phylites, schist and quartzites. The joints are spaced at 15-30 cm apart and their surfaces are plane, smooth and coated. Rough surfaces are rare. Some incipient planes of weaknesses along which movement of the rock masses have taken place, occur in the form of axial shear stresses and faults. The tunnelling media have been classified into four categories (Fig. 3.18) for purposes of support system as follows:

- Blocky structure in quartzite, metabasics (40%) which include schistose quartzite and massive variety of approximately 10%.
- Layered structure in phylite, schist (35%) which includes their variants as quartzitic phylite, quartz sericite schist.
- Fractured structure in weathered zone and closed jointed reaches (10%)
- Loosened structure along shear zones (15%).

3.13.2 Rock Mass Classification and Rock Pressures

The rocks have been classified as per RMR and Q systems (Ghosh et. al. , 1985) and support pressures have shown in Table 3.14.

3.13.3 Support Recommended

As per Deere's RQD method, rock support works out to be rock bolts at 0.9 to 1.8 m with 10 cm thick shotcrete.

According to RMR method, for metabasic rocks, generally no supports is required except for occasional spot bolting but for the quartzite phylites systematic bolts 4-5 m long and spaced at 1.5 to 2.0 m may be provided with 5 to 10 cm thick shotcrete.

As per Q system, spot bolting for metabasic rocks and systematic bolts for quartz phylite rocks are adequate supports.

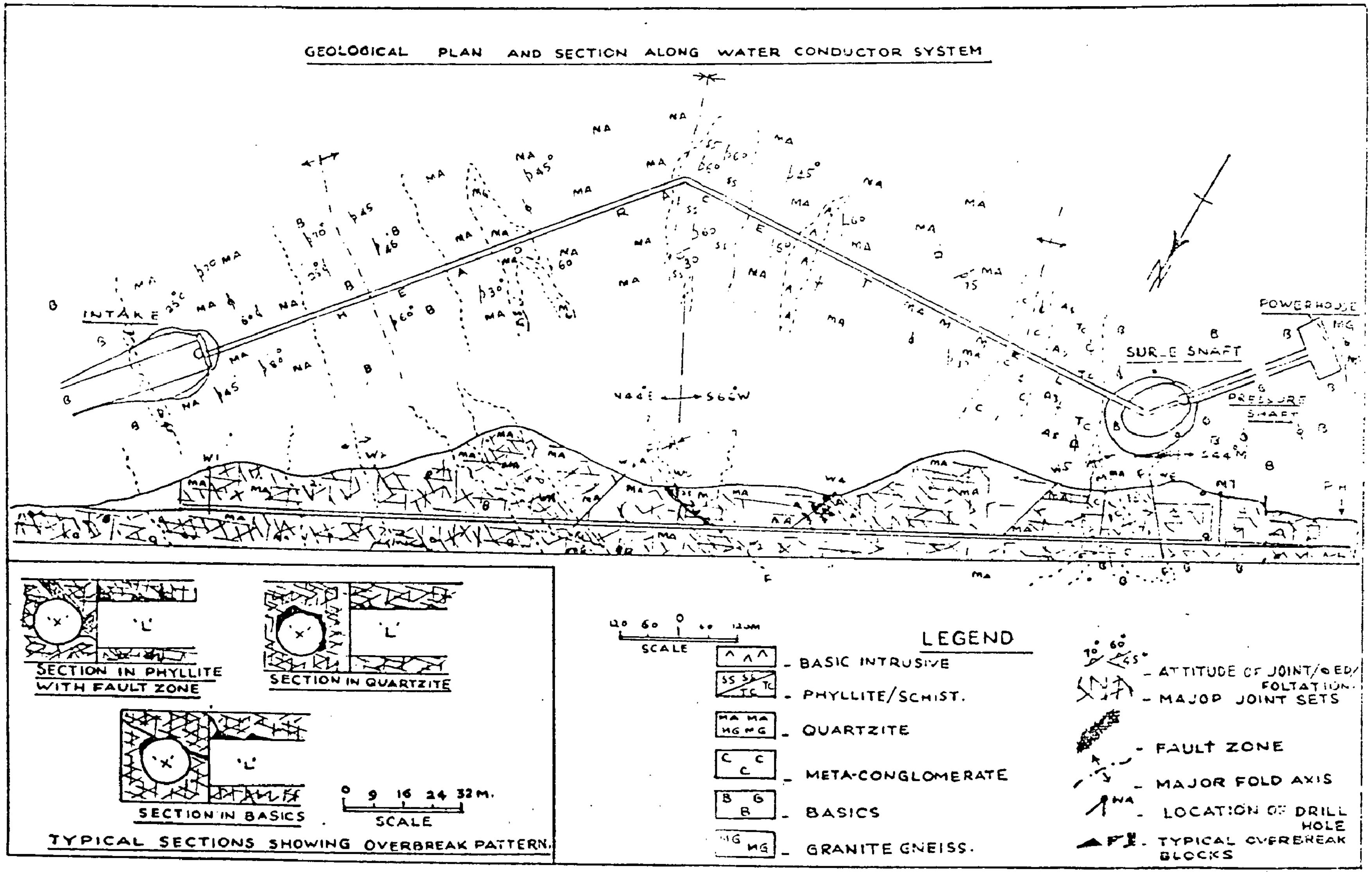


FIG. 3.18 : GEOLOGICAL PLAN AND SECTION OF WATER CONDUCTOR SYSTEM OF BODHGHAT PROJECT

TABLE 3.14 : ROCK CLASS AND SUPPORT PRESSURES AT BODHGHAT HYDEL TUNNEL, M.P.

Method of rock classification	Rock Type	
	Metabasics	Quartzite phylites
Rock Class By Terzaghi	Blocky and seamy zones, Class 4	Layered structure Class 5
Supp.Pr., kg/cm^2	0.8 - 2.4	2.4 - 7.4
Deere's RQD	Av. RQD > 50, Fair rock	
Supp.Pr.,kg/cm^2	2.1 - 4.56	
Bieniawski's Rock Mass Rating	96, Class I, Very Good Rock	69 Class II, Good Rock
Supp.Pr.,kg/cm^2	0.14	1.1
Barton's Q Values	19.8 (Good)	8.8 (Fair)
Supp. Pr., kg/cm^2	0.25	0.65
P_{wall}	0.15	0.48

3.14 LOKTAK H.E. PROJECT, MANIPUR

The Loktak Hydro-electric Project in the Eastern Himalaya is situated 39 km south of Imphal, the capital city of Manipur State. It envisages diversion of 42 cumecs of water from Loktak lake formed due to construction of a barrage across Manipur river with a gross head of 312 m for generation of 105 MW (3 x 35 MW) of power. The water conductor system is 10.27 km long and consists of 2.27 km long open channel, a 1.22 km long and 5.0 m dia horse shoe shaped cut and cover section, a 6.5 km long and 3.81 m dia horse shoe shaped head race tunnel.

3.14.1 Geology

The head race tunnel passes through lake sediments, terrace deposits and rock units of Disang group. The lake sediment is constituted by silt, sand and pebbles of variable proportions (Fig. 3.19). The terrace material contains broken rock fragments and large size boulders in addition to silt and sand fractions. The rocks are mainly sandstones, shale, and siltstones. The sandstone is predominant rock and more abundantly exposed.

Along the tunnel the rock mass shows three generation folding. The ground water in the hilly area has been observed to circulate within the weathered mantle and open fractures in rock and emerges out as springs. The majority of these springs emerge much above the tunnel grade and are the principal source of water for streams draining the hill slopes. .

3.14.2 Rock Mass Classification and Rock Pressures

The support pressures and rock mass classes are presented in Table 3.15.

TABLE 3.15 :ROCK CLASS AND SUPPORT PRESSURES AT LOKTAK HYDEL TUNNEL, MANIPUR

Method of Classification	Description
Rock Class by Terzaghi	Highly Squeezing (Class 8)
Supp. Pr. (kg/cm^2)	6.1 to 11.6
Deere's Method, RQD	5-25 (highly squeezing)
Supp. Pressure	Very Very High, Upto 75 m of rock
Bieniawski's RMR Value	10 (Very Poor)
Supp. Pr. (kg/cm^2)	1.17
Stand-up time	Immediate Collapse
Barton's Q Value (Jethwa et. al., 1982)	0.023 (Very Poor Rock)
Supp. Pr.,(kg/cm^2), P_{roof}	4.7
P_{wall}	4.7

3.14.3 Support Recommended

As per Deere's method either very heavy circular steel sets at 0.6 m spacing are required to supports the rocks or rock bolts at 0.6 to 0.9 m spacing alongwith shotcrete of 15 cm thickness combined with heavy steel sets.

By RMR method the support system works out to be either systematic bolts 5-6 m long at 1-1.5 m spacing with wire mesh and shotcrete of 15-20 cm thickness, or medium to heavy steel sets at 0.75 m spacing with steel lagging and fore poling if required.

Q system, recommends 3 m long rock bolts at 1.1.m spacing with 10 cm thick fibre reinforced shotcrete.

3.14.4 Support Actually Provided

The following supports have been provided in the head race tunnel.

- 3 m long bolts with a flexible shotcrete lining with wire mesh is provided as temporary or immediate support.
- Finally steel sets of 150 mm x 150 mm size embedded in 30 cm thick M-250 cement concrete lining was adopted as permanent support.

In this case no gap was left between shotcrete lining and steel supports. The steel supports were designed to take entire squeezing rock pressure. Details of the adopted supports are as shown in Fig. 3.20.

3.15 SALAL HYDRO-ELECTRIC PROJECT, J. & K.

The Salal Hydro-electric Project is situated around 120 km north of Jammu in J. & K. State. The tail race tunnel of the project consists of 12 m diameter horse shoe shaped, 2.6 km long and passes through various grades of dolomites of Lower Himalayas.

While tunnelling, no frequent tunnelling problems were encountered except a major collapse with water inrush and gaugy
material. The tunnel was monitored by installing load cells and closure studs for evaluating the steel rib supports.

3.15.1 Geology

The tunnel is aligned through single litho-unit of dolomitic rocks. Since the site is located in the close proximity of the `Main Boundary Fault (MBT)', the dolomites are highly jointed. The geological cross section shows the anticlinal fold with its axis trending NNW-SSE (Fig. 3.21). At inlet side, the dolomites generally strikes N80^{o}E - S80^{o}W to E-W with dip 50^{o}-60^{o} towards NNW-North and at outlet strike NE-SW with dip of 45-60^{o}. The orientation axis of the tunnel is N20^{o}. The dolomites exposed in the area have been divided in various categories based on their physical behaviour, extent of crushing, shearing, number of joints and their spacing (Goel et. al., 1996).

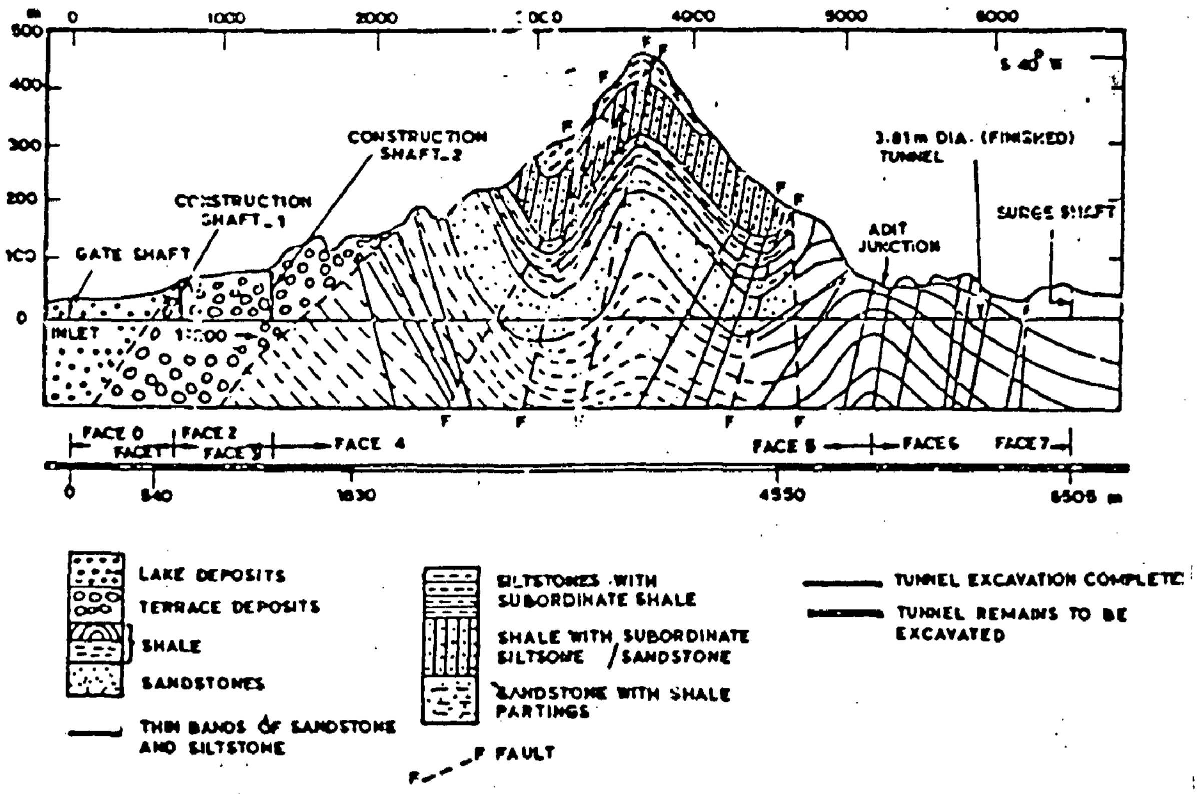


FIG. 3.19 : GEOLOGICAL SECTION OF LOKTAK HYDEL TUNNEL

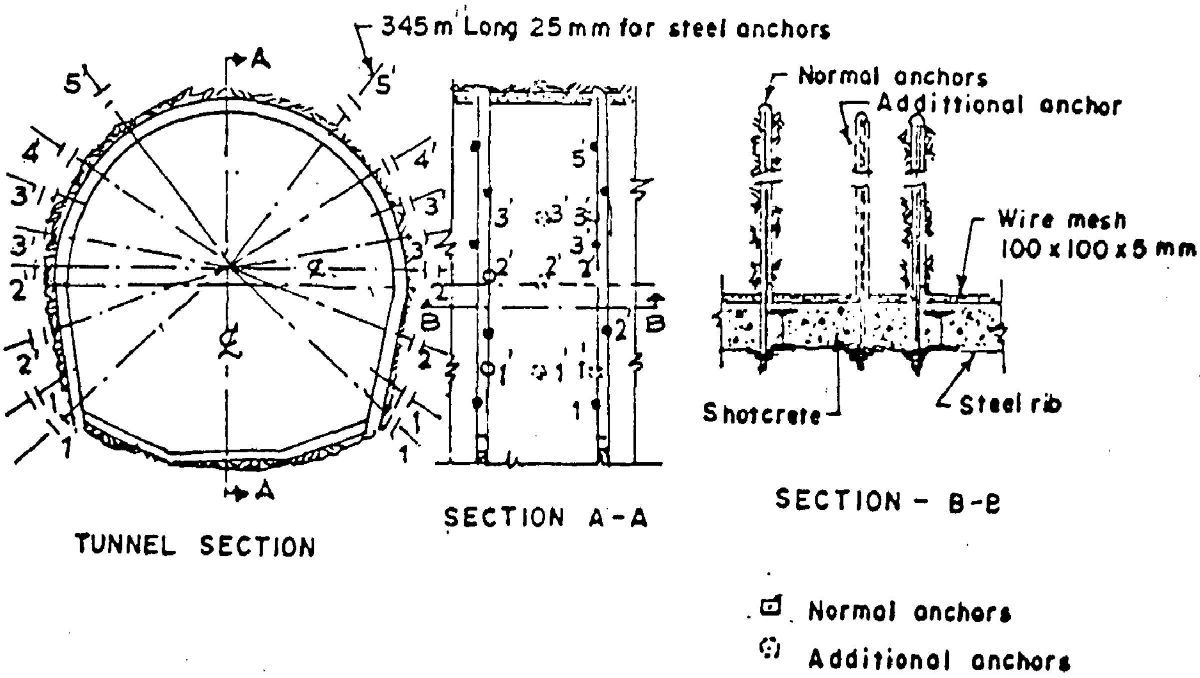


FIG. 3.20 : DETAILS OF ROCK SUPPORT IN LOKTAK HYDEL TUNNEL

3.15.2 Rock Mass Classification and Rock Pressures

Support pressures and rock mass classes are shown in Table 3.16 (Goel et. al., 1996).

TABLE 3.16 : ROCK CLASS AND SUPPORT PRESSURES AT TAIL-RACE TUNNEL OF SALAL H.E. PROJECT, J. & K.

Method of rock classification	Rock Type		
	Blocky & cherty dolomites	Highly jointed Dolomites	Crumbly and sheared dolomites
Rock Class By Terzaghi	Massive moderately jointed (Class 3)	Very Blocky and Seamy (Class 5)	Squeezing at moderate depth (Class 7)
Supp.Pr., kg/cm^2	0 to 1.59	2.23 to 7.0	7-13.4
Bieniawski's Rock Mass Rating	47 (Fair)	32 (Poor)	15 (Very Poor)
Supp.Pr., kg/cm^2	1.7	2.2	2.7
Barton's Q Values	1-2.3 (Av. 1.7)	0.17-0.22(Av. 0.2)	0.02
Supp.Pr., kg/cm^2 P_{roof}	1.1	2.3	4.4

3.15.3 Supports Recommended

The support systems have been worked out using Bieniawski's RMR and Barton's Q systems and are shown in the Table 3.17.

3.15.4 Support Actually Provided

For Grades II & III rock masses, steel supports with concrete backfill has been used as the primary support, whereas in Grade I rocks no support or spot bolting as primary support has been used. Mainly four sections of steel has been used in the tunnel. The capacities of these sections in case of TRT II with varying spacing of steel ribs are given below in Table 3.18.

It is evident from the table that the capacities of steel rib support can be increased or decreased by changing the spacing of steel ribs. ISMB 300 x 140 mm has been used in grade III rocks with their spacing as 0.5 m. Fig 3.22 shows the final profile of the tail race tunnel after the installation of supports.

TABLE 3.17 : RECOMMENDED SUPPORT FOR TAIL RACE TUNNEL OF SALAL H.E. PROJECT, J.K.

Method of rock classification	Support Recommended for Different Grades of Rock		
	Blocky & cherty dolomites	Highly jointed Dolomites	Crumbly and sheared dolomites
Bieniawski's RMR system	Systematic bolts of 4m long, 1.5-2.0m centers with wire mesh + shotcrete 5-10cm thick.	Systematic bolts of 4-5m long, 1-1.5m centers with wire mesh + shotcrete 10-15 cm thick.	Rock bolts of 5-6m long, 1-1.5m centers with wire mesh + shotcrete 15-20cm thick.
Q System	Untensioned grouted rock bolts of 3.5m long at 1.8 m spacing + 7.5 m thick fibre reinforced shotcrete	Tensioned grouted rock bolts of 3.5 m long at 1.4 m spacing + fibre reinforced shotcrete of 12 cm thick	Reinforced ribs and shorcrete more than 15 cm thickness & rock bolts at 1.2 m spacing.

TABLE 3.18 : PROVIDED SUPPORT AT TAIL RACE TUNNEL OF SALAL HYDEL PROJECT, J & K.

Steel rib section	Cross-sectional area, cm^2	Support capacity for spacing, MPa			
		0.5 m	0.7 m	1.0 m	1.3 m
ISMB 200 x 200	47.54	0.39	0.28	0.19	0.15
ISMB 300 x 140	56.26	0.47	0.33	0.23	0.18
ISMB 250 x 125	42.02	0.35	0.25	0.17	0.13
ISMB 300 x 150	48.08	0.399	0.285	0.199	0.15

3.16 YAMUNA HYDRO-ELECTRIC SCHEME, STAGE II, PART I

Yamuna Hydro-electric Stage II, Part I comprise of a diversion dam at Ichhari, a head race tunnel and an underground power house at Chhibro. The head race tunnel of circular section comprise of 7.0 m finished diameter and 6.1 km long.

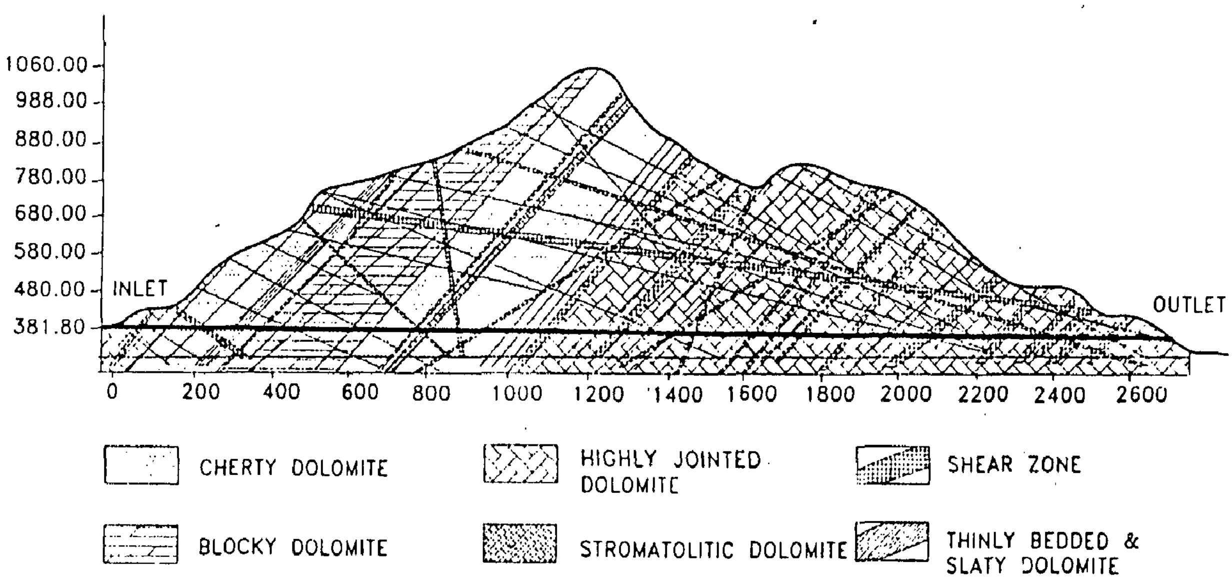


FIG. 3.21 : GEOLOGICAL L-SECTION ALONG TRT II OF SALAL HYDEL PROJECT

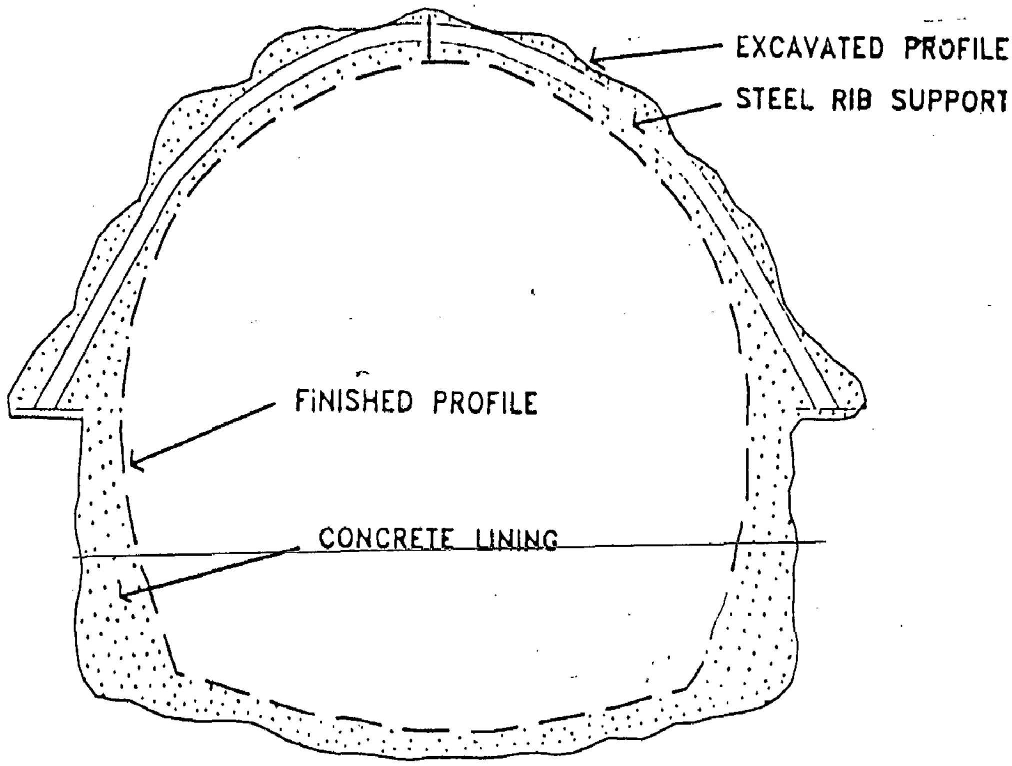


FIG. 3.22 : SECTION OF SALAL HYDEL PROJECT TAIL RACE TUNNEL SHOWING RIB SUPPORT

3.16.1 Geology

The rock type comprise of slates interbanded with quartzites and limestones. The limestones belongings to the Bansa stage of the Chandpur are hard and tough, whereas the limestones of Dhaira stage of the Mandhalis are relatively soft and interbanded with slates. The alignment of the tunnel is N60°W to S60°E direction, which is almost at right angle to the regional strike of the local variations. For geological section of the tunnel refer Fig. 3.23.

3.16.2 Rock Mass Classification and Rock Pressures

The rock load varies between 0.25 B to 0.5 B as per Terzaghi's rock classification. A rock load of 0.375 B (0.7 kg/cm^2) has been taken for the design of supports. The rock pressures by Singh et. al. comes out to be between 0.4 to 0.7 kg/cm^2.

3.16.3 Support Recommended

The recommended support for the tunnel correspond to 3m long rock bolts at a spacing of 1.5 m and 5 cm thick shotcrete layer.

3.16.4 Support Actually Provided

For supporting the rock load steel supports of 250 x 125 mm size at 1.5 m spacing have been provided (Fig. 3.24). In greater part of the tunnel the support was not required. Rock bolts have been provided for jointed rocks.

3.17 YAMUNA HYDRO-ELECTRIC SCHEME, STAGE II, PART II

Yamuna Hydro-electric Scheme Stage II, Part II in Outer Himalayas envisages utilisation of 64 m drop available between tail race of Chhibro underground power house and power house at Khodri in Uttar Pradesh State. The Chhibro-Khodri tunnel is 5.6 km long having a finished diameter of 7.5 m constructed to carry water from Chhibro power house for generation of 120 MW of power.

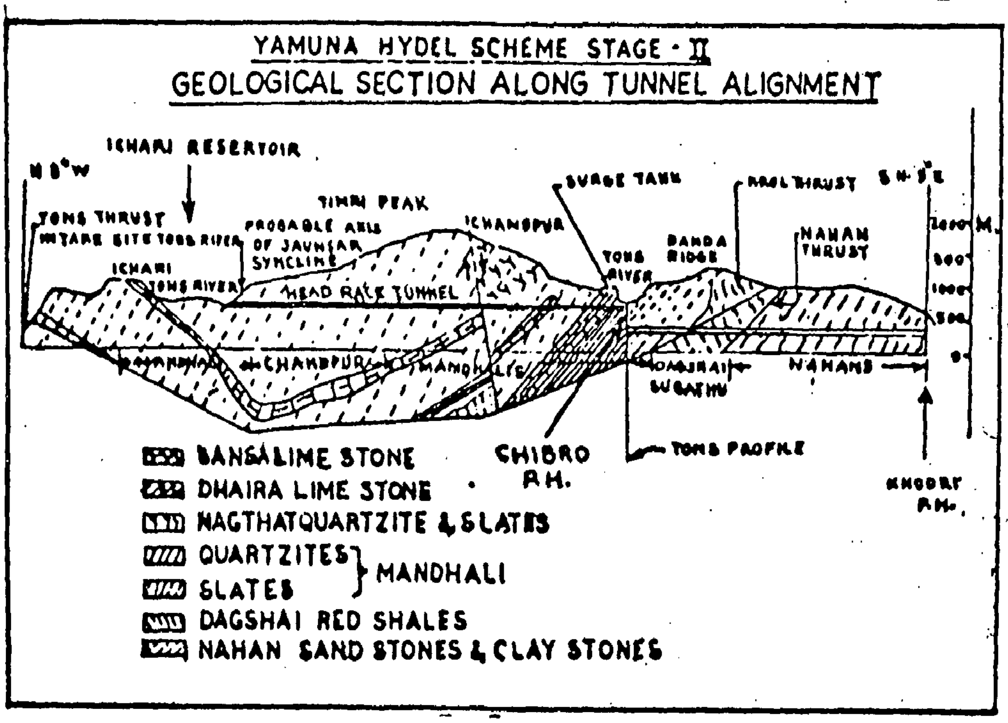


FIG. 3.23 : GEOLOGICAL L-SECTION OF ICHHARI - CHHIBRO HRT

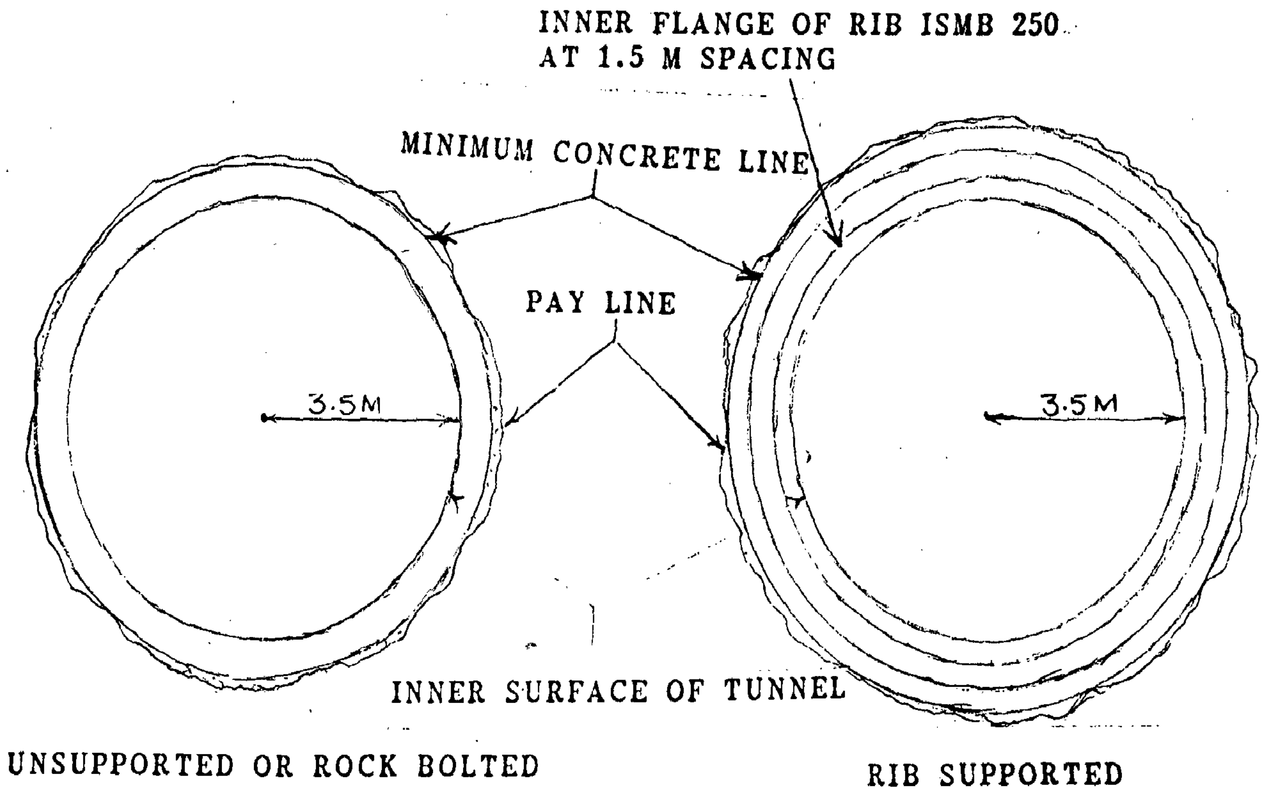


FIG. 3.24 : ADOPTED ROCK SUPPORT FOR ICHARI - CHHIBRO HRT

3.17.1 Geology

The Chhibro-Khodri head race tunnel passes through Nahans constituted of bands of sandstone, siltstone and claystones from Khodri end in about 3.0 km length. From Chhibro end the tunnel passes through Mandhalies consisting of quartzites and slates in a length of about 2.3 km. In between these two formations about 300 m length thrust zone bounded by Krol and Nahan thrusts and comprising of crushed, sheared and highly brecciated red shales and subathu clay has been met along the tunnel alignment (Fig. 3.25).

3.17.2 Rock Mass Classification and Rock Pressures

The rock mass classification and the support pressures are as shown in Table 3.19.

TABLE 3.19 : ROCK CLASS AND SUPPORT PRESSURES AT CHHIBRO KHODRI TUNNEL, U.P.

Method of rock classification	Description of rock	
	Red Shales	Black clays
Rock Class By Terzaghi	Moderately squeezing (Class 7)	Moderately Squeezing (Class 7)
Support Pr. (kg/cm^2)	4.08-7.79	4.08-7.79
Deere's Method, RQD[a]	<25 (Very Poor)	<25 (Very Poor)
Supp. Pr. (kg/cm^2)	3.71-5.19	3.71-5.19
Bieniawski's RMR	17 (Very Poor)	7 (Very Poor)
Supp. Pr. (kg/cm^2)	1.54	1.73
Barton's Q Values (Jethwa et. al.,1982)	0.05 (Extremely Poor)	0.022 (Extremely Poor)
Supp.Pr.,kg/cm^2,P_{roof}	3.5	7.0
P_{wall}	3.5	7.0

3.17.3 Support Recommended

As per Deere's method rock bolts at 0.6 to 0.9 m spacing with 15 cm thick shotcrete on whole section combined with heavy steel sets is the adequate support.

According to RMR method 20 mm dia, 5-6 m long rock bolts at 1-1.5 m spacing with 15-20 cm thick wire mesh shotcrete will be the required support. Or, medium to heavy steel sets spaced 0.75 m with steel lagging can be used to support the rock.

However, Q system recommends 2.3 m long rock bolts at 1 m spacing alongwith fibre reinforced shotcrete 15 cm thick.

3.17.4 Support Actually Provided

Heavy supports of size 300 mm x 140 mm RS joists with cover plates of size 250 mm x 20 mm welded in the outer and inner flange of RS Joist placed at 0.35 m centres and rigid backfill has been used in this case (Fig. 3.26).

3.18 MANERI BHALI HYDEL PROJECT STAGE I, U.P.

Maneri Bhali Hydro-electric Project, Stage I has been constructed across the river Bhagirathi in the State of Uttar Pradesh. The Maneri Bhali Stage I project in the Middle Himalayas has 8.36 km long tunnel with 4.75 m finished diameter circular tunnel. In case of Maneri Bhali Stage II the tunnel is 16 km long and 6.0 m finished diameter horse shoe shaped.

3.18.1 Geology

The tunnel passes through heterogeneous rock formations represented by the metavolcanics, basic intrusives (epidiorites), quartzites, slates, phylites, limestone, sandstones, shales and even consolidated sand, soil clay siltstones and bed material deposit (Fig. 3.27). The gneisses and granites exhibit sheared and weathered phylites at thrust contacts. Apart from this squeezing ground was encountered for a length of about 350 m.

3.18.2 Rock Mass Classification and Rock Pressures

The rock mass classification and the support pressures are as shown in Table 3.20.

3.18.3 Support Recommended

The recommended rock supports have been shown in Table 3.21.

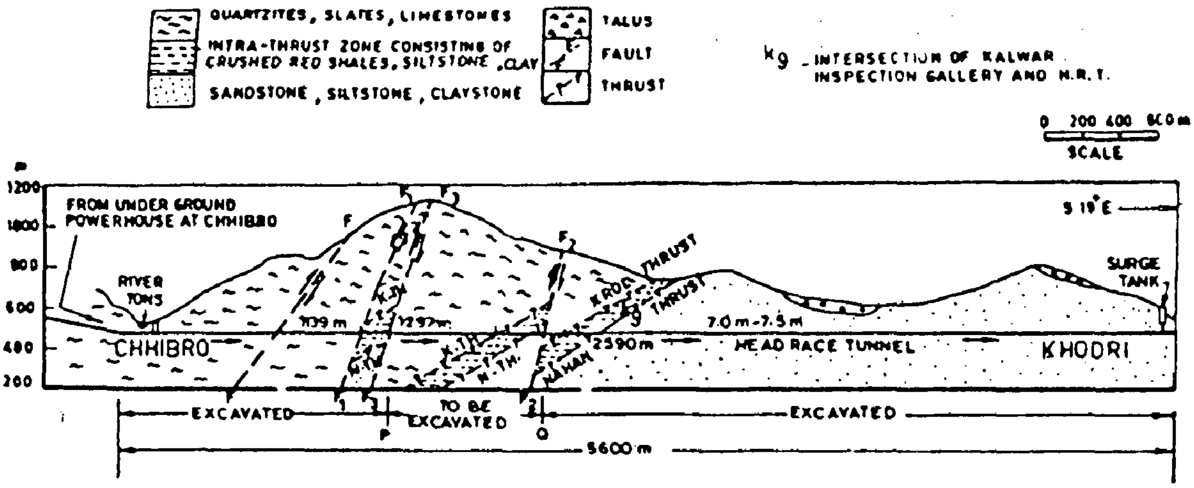


FIG. 3.25 : GEOLOGICAL L-SECTION OF CHHIBRO - KHODRI HRT

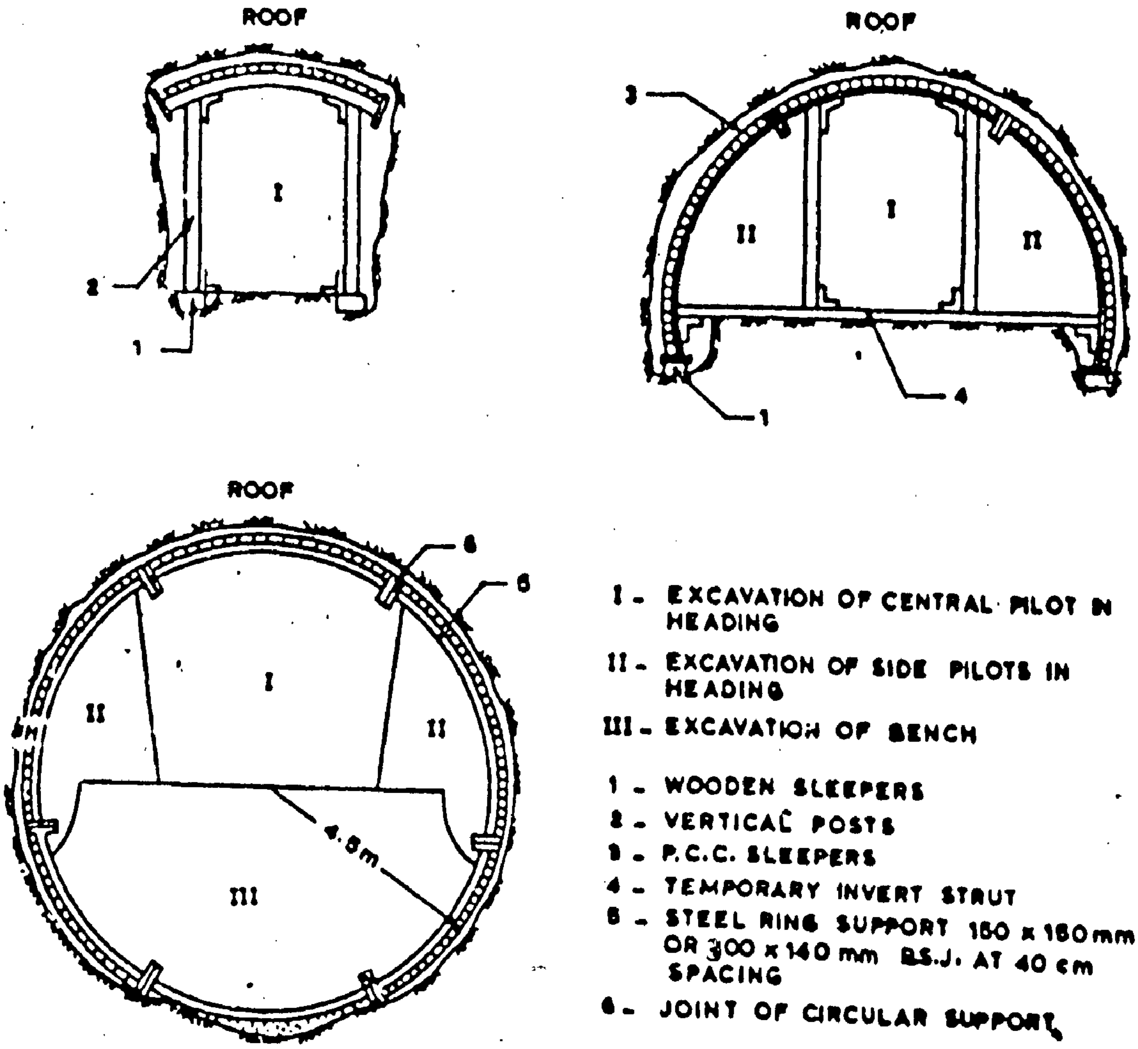


FIG. 3.26 : SEQUENCE OF EXCAVATION AND SUPPORT FOR CHHIBRO - KHODRI TUNNEL THROUGH INTRA-THRUST ZONE AT KALAWAR

TABLE 3.20 : ROCK CLASS AND SUPPORT PRESSURES AT MANERI BHALI STAGE I HEAD RACE TUNNEL, U.P.

Method of rock classification	Rock Type			
	Moderately fractured quartzites	Foliated Metabasics	Sheared Metabasics	Highly fractured quartzites
Terzaghi's Method	Rock Class 4	Rock Class 4	Rock Class 6	Rock Class 7
Supp. Pr., kg/cm^2	0.3 to 0.80	0.3-0.80	2.77	2.77-5.29
Deere's method	75 (fair to Good Rock)	82 (Good Rock)	60 (Fair Rock)	60 (fair Rock)
Supp. Pr., kg/cm^2	0.48	0.36-0.72	0.72-1.56	0.72-1.56
Bieniawski's RMR	58 (Fair Rock)	59 (Fair Rock)	49 (Fair rock)	38 (Poor rock)
Supp. Pr., kg/cm^2	0.61	0.60	0.74	0.90
Barton's Q Values	3-6	3.4-6.8	0.3-3.3	0.5
Jethwa et.al.,(1982) Supp. Pr.,kg/cm^2 P_{roof}	0.5-0.7	0.5-0.7	0.7-1.8	1.6
P_{wall}	0.1-0.2	0.1-0.2	0.2-1.2	1.1

TABLE 3.21 : RECOMMENDED SUPPORT FOR MANERI BHALI STAGE I HEAD RACE TUNNEL, U.P.

Method of rock classification	Recommended Rock Support			
	Moderately fractured quartzites	Foliated Metabasics	Sheared Metabasics	Highly fractured quartzites
Deere's method	Rock bolts at 1.5-1.8 m spacing with occasional local application of 5 -7.5 cm thick shotcrete	Rock bolts at 1.5-1.8 m spacing with occasional local application of 5 -7.5 cm thick shotcrete	Rock bolts at 1.5-1.8 m spacing with occasional local application of 15 cm thick shotcrete	Rock bolts at 1.5-1.8 m spacing with occasional local application of 15 cm thick shotcrete
Bieniawski's	Systematic bolts 4-5 m long at 1.5 - 2.0 m spacing with shotcrete 5-10 cm thick.	Systematic bolts 4-5 m long at 1.5 -2.0 m spacing with shotcrete 5-10 cm thick.	Systematic bolts 4-5 m long at 1.5 -2.0 m spacing with shotcrete 5-10 cm thick.	Systematic bolts 4-5 m at 1-1.5 m spacing with 10-15 cm thick shotcrete with wire mesh
Barton's Q System	Rock bolts at 1.6 m spacing with nominal shotcrete	Rock bolts at 1.6 m spacing	Rock bolts at 1.7 m spacing with 5 cm thick shotcrete	Rock bolts at 1.5 m spacing with 9 cm thick fibre reinforced shotcrete

3.18.4 Support Actually Provided

In the Maneri Bhali head race tunnel ISMB 250 x 125 mm steel ribs have been used to support the rock (Fig. 3.28). Depending upon the type of rock quality, the rib spacing has been varied from 50 cm to 120 cm. Steel rib supports of 150 x 150 mm has also been used at a spacing of 120 cm for rock load of 0.375 B and 80 cm for rock load of 1.0 B respectively.

3.19 KHARA HYDEL PROJECT, U.P.

The project lies within Shivalik formation of tertiary ages. However, the tunnelling is confined to Upper Shivaliks. Two twin tunnels of 6 m diameter and 1.2 km long each are located on the left bank of Yamuna river near Paonta Sahib in Western part of Uttar Pradesh.

3.19.1 Geology

The tunnels passes through weakly compacted and erratically distributed calcareous and argillaceous boulder conglomerates of shivalik formation. The conglomerates in the area are represented by boulder to granular size fragments of various shapes of quartzite, sandstone, schists and gneisses. Two types of conglomerates have been identified within the tunnels site namely calcareous and argillaceous (Figs. 3.29).

3.19.2 Rock Mass Classification and Rock Pressures

The support pressures using the classifications are shown in the Table 3.22 (after Saini et. al, 1985).

3.19.3 Support Recommended

As per Deere's RQD method the support for the phylites rock mass is rock bolts at a spacing of 1.5-1.8 m and shotcrete 7.5 cm thick. Bieniawski's RMR method recommends the use of rock bolts of 3 m long at 2.5 m spacing and 5 cm thick shotcrete whereas Barton's Q system suggests rock bolts 3 m long at 1.6 m spacing alongwith 4 cm thick shotcrete. For argillaceous

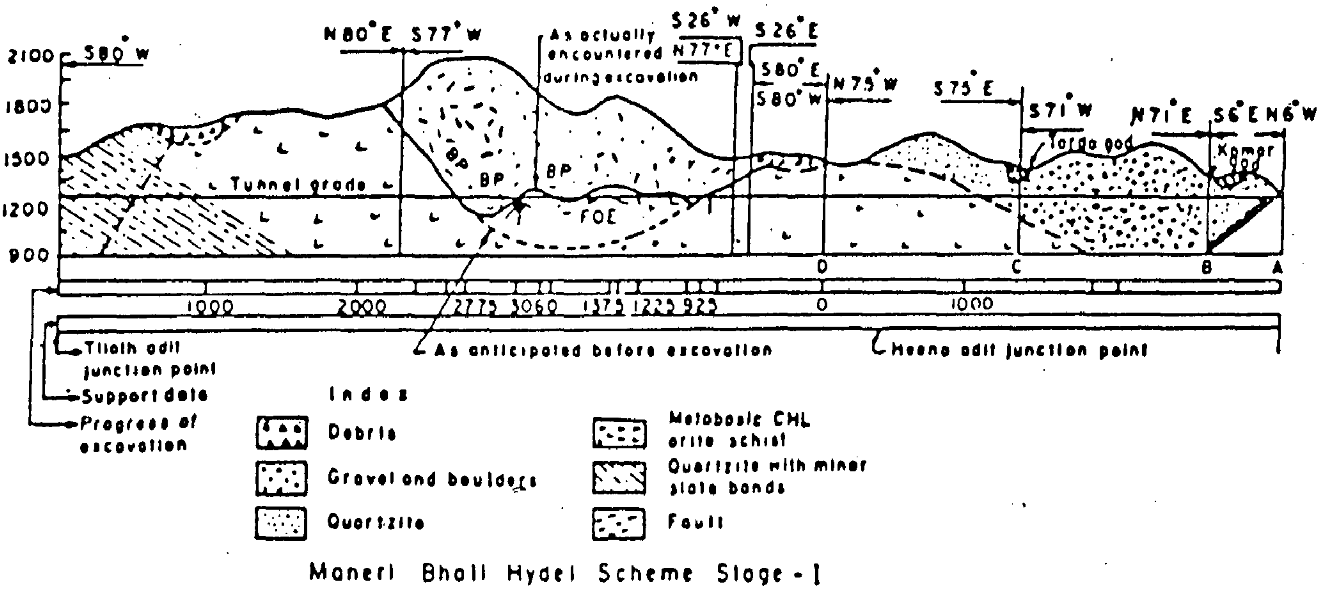


FIG. 3.27 : GEOLOGICAL L-SECTION ALONG HRT OF MANERI BHALI HYDEL SCHEME STAGE I

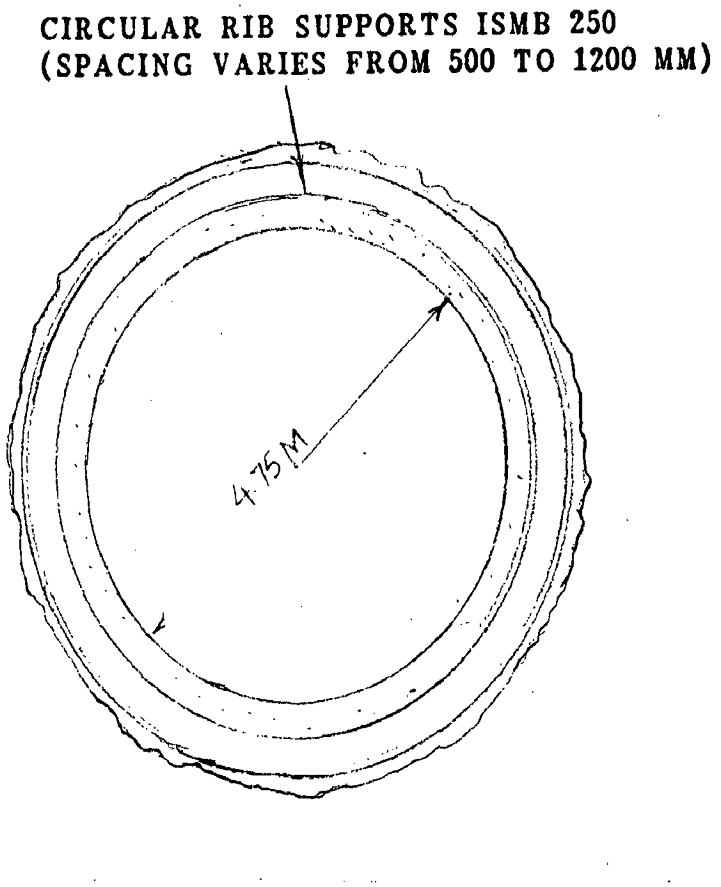


FIG. 3.28 : ROCK SUPPORT SYSTEM ADOPTED IN HRT OF MANERI BHALI HYDEL SCHEME STAGE I

and calcareous agglomerates the recommended support is rock bolts spaced at 1 m and 15 cm thick fibre reinforced shotcrete.

TABLE 3.22 : ROCK CLASS AND SUPPORT PRESSURES AT KHARA PROJECT TUNNELS, U. P.

Method of rock classification	Description of rock		
	Phylites	Argillaceous Conglomerates	Calcareous Conglomerates
Rock Class By Terzaghi	Massive and Distinctly jointed (Class 2)	Moderately squeezing (Class 7)	Moderately blocky (Class 4)
Support Pr. (kg/cm^2)	0.92	3.5-6.8	0.5-1.3 (after Saini et. al. 1985)
Deere's RQD	75 (Good rock)		
Supp. Pr., kg/cm^2	1.48		
Bieniawski' RMR	67 (Good rock)		
Supp. Pr.,kg/cm^2	1.22		
Barton's (Q Values)	5 (Fair rock)	0.022,Extremely Poor	0.05 (Extremely Poor)
Supp.Pr.,kg/cm^2,P_{roof}	0.4	3-3.5	0.7-1.7

3.19.4 Support Actually Provided

In Khara head race tunnels ISMB 250 x 125 mm size steel ribs have been provided at varying spacing from 375 mm to 750 mm where the rock cover is less than 3D and in reaches where the rock cover is more than 3D, the same ribs have been provided at 500 mm centres (Fig. 3.30). Tie rods of 20 mm diameter have also been used.

3.20 TEHRI HYDRO-ELECTRIC PROJECT, U.P.

Tehri Dam Project, across river Bhagirathi, envisages the construction of a 260 m high rockfill dam and an underground power house to be built in two stages. The underground works mainly comprise of diversion tunnels, two on each bank, four head race tunnels and underground power house complex. The diversion tunnels of 11.0 m diameter horse shoe shaped are designed to pass a routed construction stage flow of nearly 7500 cumecs.

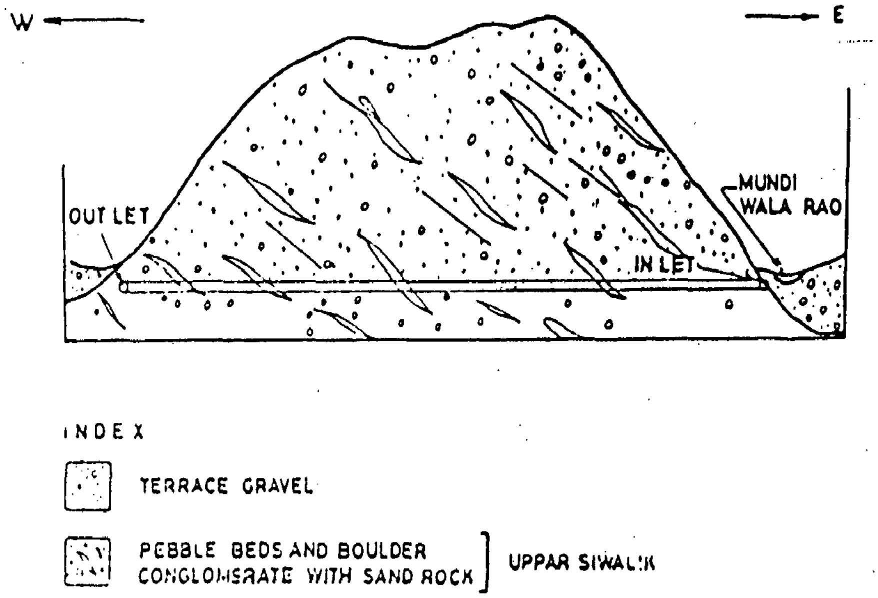


FIG. 3.29 : GEOLOGICAL SECTION ALONG KHARA PROJECT TUNNELS

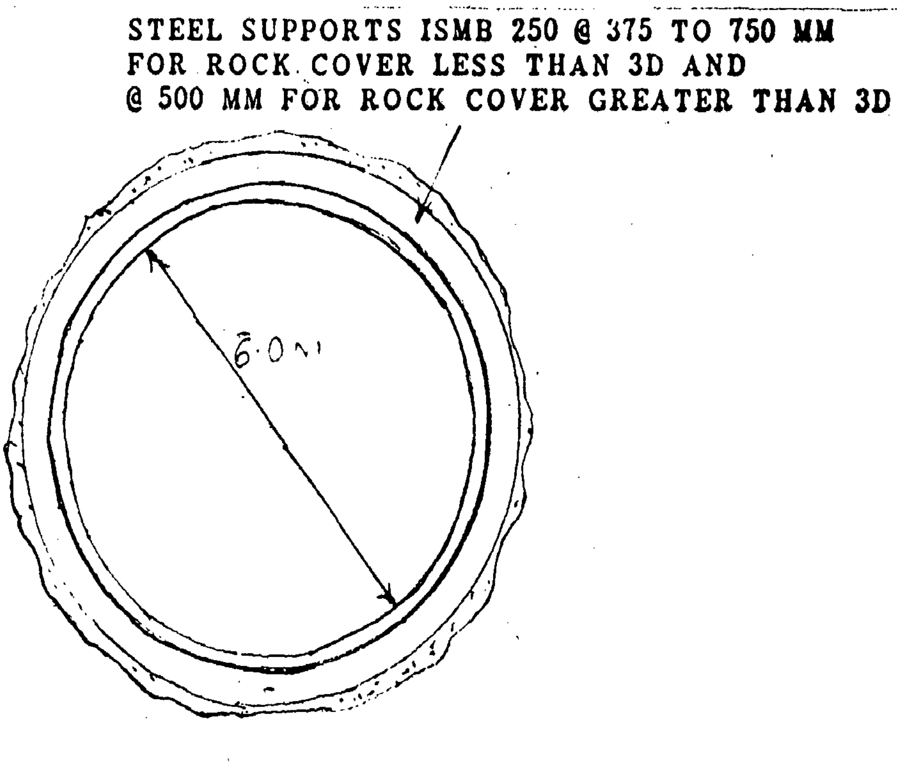


FIG. 3.30 : STEEL RIB SUPPORTS ADOPTED IN KHARA PROJECT TUNNELS

3.20.1 Geology

The rock type in tunnels T1 and T2 comprise of phylites of grade II and III whereas in tunnels T3 and T4 is phylites of Grade I, Grade II and Grade III about 30%, 60% and 10% respectively (Fig. 3.31a and 3.31b). The diversion tunnels are aligned in N6°W - S6°E direction. The maximum rock cover is 250 m. Rocks of grade I are most competent rock formations whereas rocks of grade III are weakest formations. The three grades of phylites are interbedded and show gradual change from one to the other grade.

3.20.2 Rock Mass Classification and Rock Pressures

The support pressures are shown in Table 3.23.

TABLE 3.23 : ROCK CLASS AND SUPPORT PRESSURES TEHRI DAM DIVERSION TUNNELS, U.P.

Method of rock classification	Description of rock		
	Phylites Grade I	Phylites grade II	Phylites Grade III
Rock Class By Terzaghi (after Singh et. al. 1995a)	Massive phylites jointed (Class 3)	Moderately blocky phylites (Class 4)	Phylites with band of argillaceous material (Class 5)
Support Pr. (kg/cm^2)	0-1.8	0.73-2.04	2.22-7.00
Deere's RQD	50-75 (Fair Rock)		50
Supp. Pr., kg/cm^2	0.95 - 1.90		4.13 - 6.36
Barton's (Q Values)	----- 0.25 - 2.00 (Av. 1.2)		0.8
Supp.Pr.,kg/cm^2,P_{roof}			1.2

3.20.3 Support Recommended

As per Deere's RQD method the rock support conform to light to medium steel sets 1.2 to 1.5 m centres or rock bolts at 0.9 to 1.8 m spacing for Grade I and II rocks whereas for grade III rocks medium to heavy sets at 0.6 to 1.2 m spacing or rock bolts at 0.6 to 1.2 m centres are specified. Q system recommends the use of rock bolts of 4.0 m length spaced at 1.3 m centres with fibre reinforced shotcrete of more than 15 cm thickness.

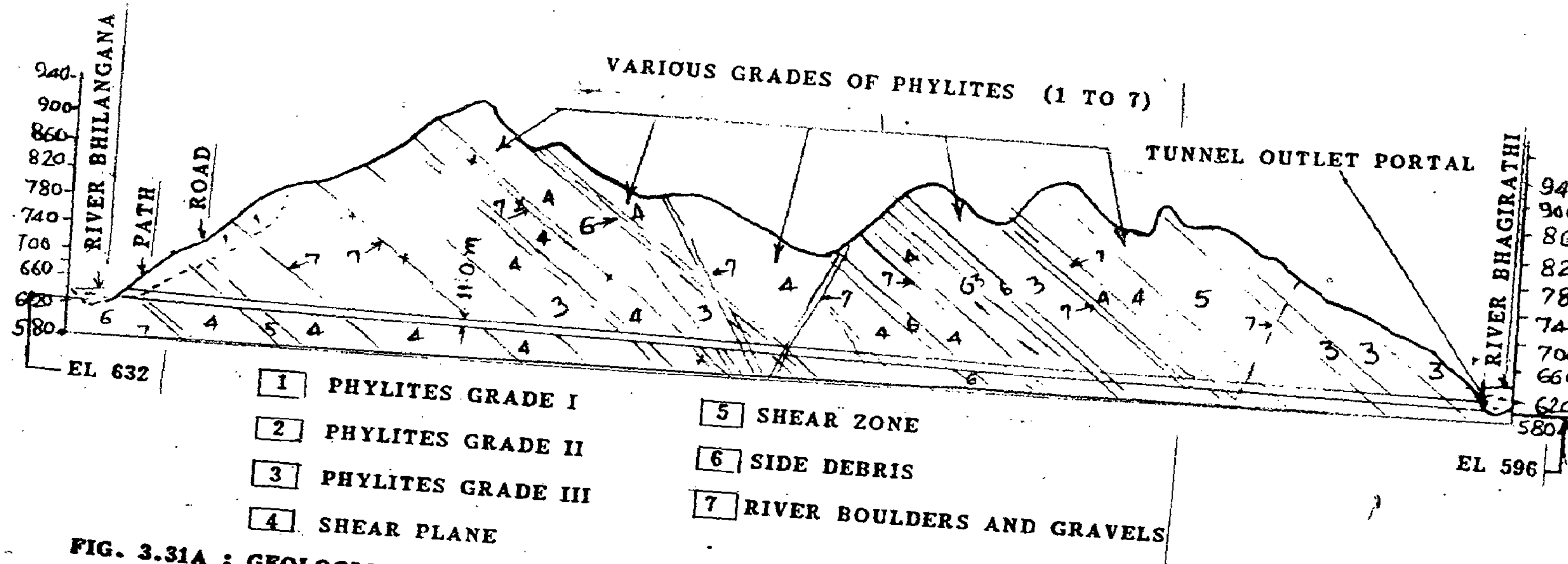


FIG. 3.31A : GEOLOGICAL SECTION ALONG LEFT BANK TEHRI DAM DIVERSION TUNNELS

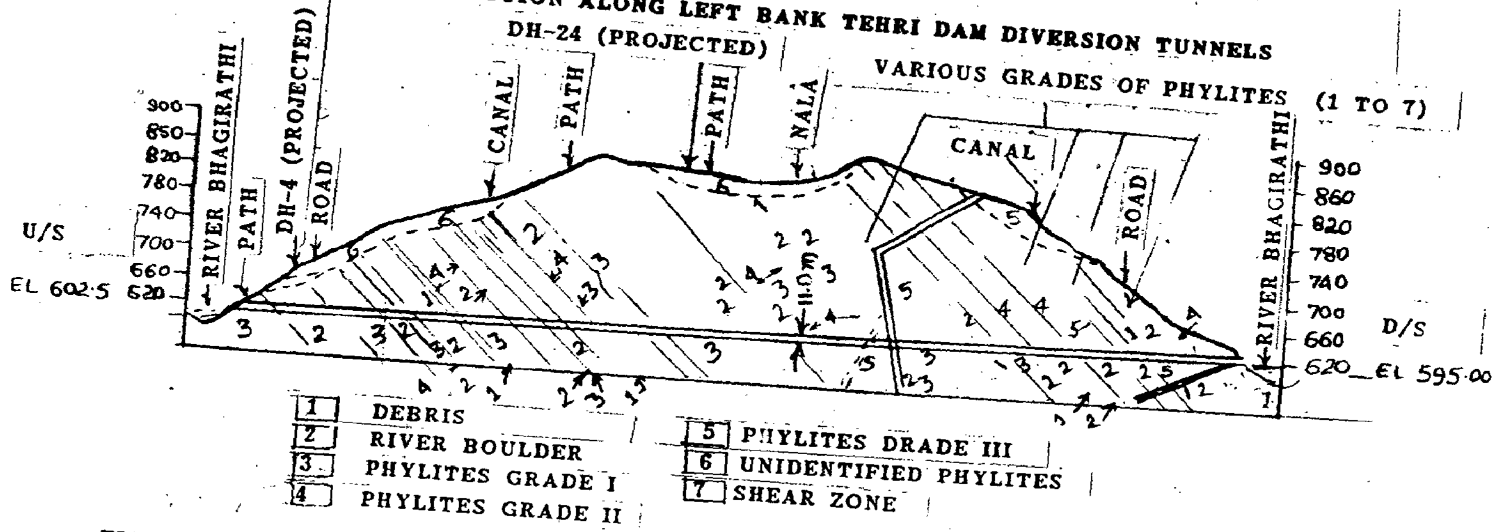


FIG. 3.31B : GEOLOGICAL SECTION ALONG RIGHT BANK TEHRI DAM DIVERSION TUNNELS

3.20.4 Supports Actually Provided

The supports actually provided for various classes of rock are as summarised in Table 3.24. The section of the tunnel showing the rib support is shown in Fig. 3.32.

TABLE 3.24 : ACTUAL SUPPORT PROVIDED IN TEHRI DAM DIVERSION TUNNELS, U.P.

Rock Type : Phylites			
Grade I	Grade II		Grade III
	Alternative I	Alternative II	
25 mm diameter bolts 3 m deep at 90 cm centres with 10 cm thick shotcrete	15 cm thick shotcrete without rock bolts	ISMB 150 x 1150 @ 34.6 kg/m at 50 - 70 cm centres	Steel supports of ISMB 300 x 140 @ 44.2 kg/m with plates of 250 x 10 mm on both flanges at a spacing of 95 cm centres

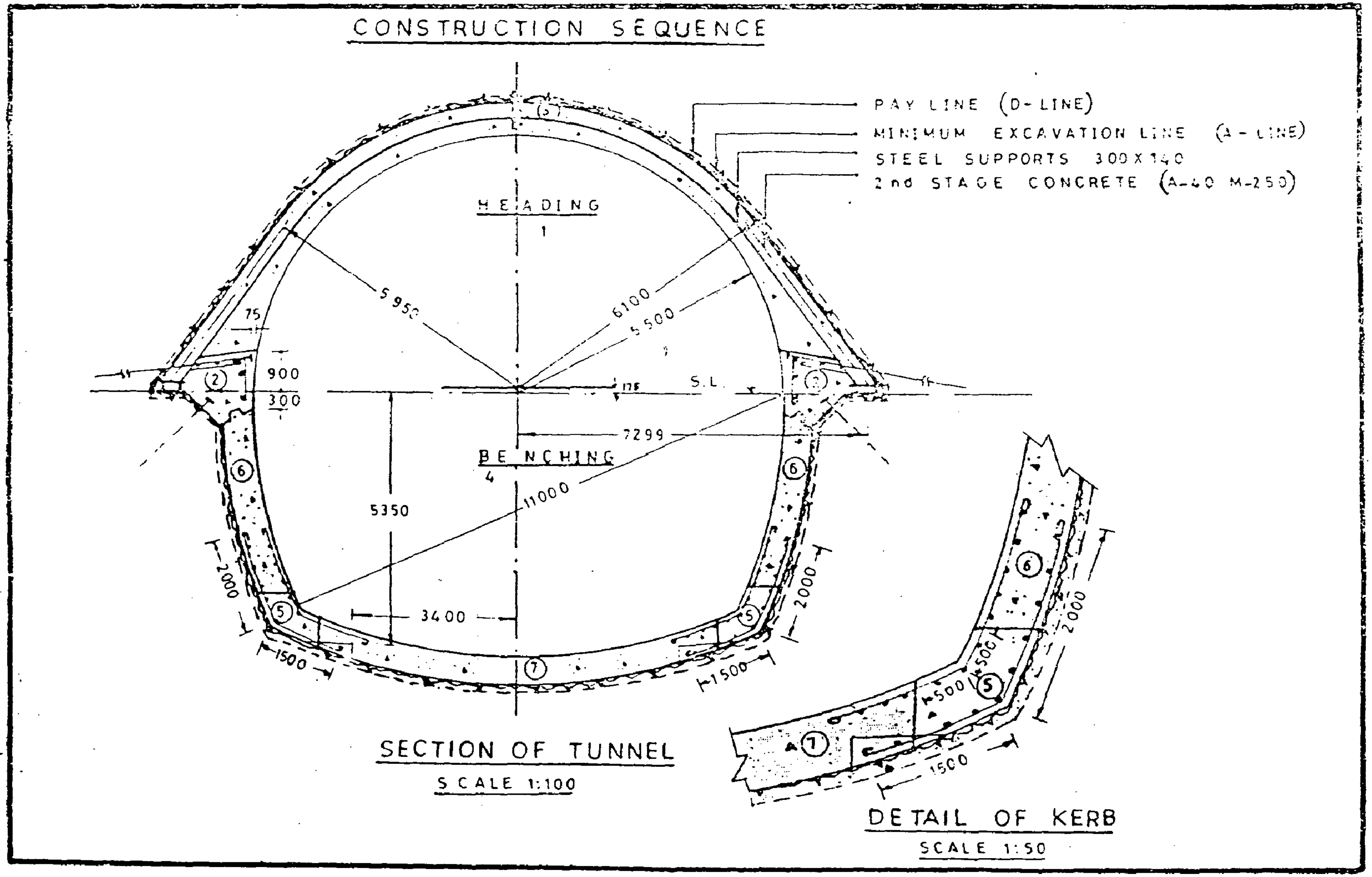


FIG. 3.32 : ROCK SUPPORT AND CONSTRUCTION SEQUENCE IN TEHRI DAM DIVERSION TUNNELS

CHAPTER 4
DESIGN OF CONCRETE LINING FOR WATER TUNNELS

4.1 PURPOSE OF LINING IN TUNNELS

Lining of water tunnel is required for the following reasons:

- To reduce head losses in the system in case of power tunnels
- to protect steel ribs from deteriorating
- to prevent erosion of rocks and entry of particles that may damage the turbines
- to increase the discharging capacity of diversion tunnels
- to take care of internal water pressures which is not taken by rock
- to prevent leakage of water

Lining costs almost 30 - 40 per cent of the tunnel. Power tunnels must be lined with cement concrete - plain or reinforced; or steel lined. However, in case where a tunnel is meant for operation for short periods and where it has to be abandoned after it has served the purpose as in case of diversion tunnels the lining could be avoided or economics can be worked out if lining is provided.

Concrete lining is normally provided for power tunnels directly connected to the pressure pipe lines and thence to the turbines in power houses. The concrete lining is required to ensure that no sand or rock particles are carried from the tunnel system into the machine.

When rock cover is inadequate to prevent the leakage and where high velocity erosion or cavitation is expected as in the case of silt flushing tunnels - a steel lining is required. The main function of the steel liner is to protect the concrete and to stop leakage of water from the tunnel. In general the steel lining should be strong enough to withstand the internal water pressure not taken by rock surrounding the lining and must be capable of taking the full external water pressure.

4.2 STRESS CONDITION IN A HOMOGENEOUS ELASTIC HALF SPACE

A general solution for the analytical determination of stress developing around a circular cavity in elastic media was given by Mindlin (1939). An approximate solution for the computation of tangential stresses was given by Kerisel as follows :

$$\sigma_t = \gamma h\left[1+\frac{a^2}{r^2}\right]$$

The radial stresses being given by :-

$$\sigma_t = \gamma h\left[1-\frac{a^2}{r^2}\right]$$

where h = Depth of rock cover

a = The radius of the circle

r = The distance of the point under consideration.

The influence of the shape of the cavity was investigated on the basis of pure elasticity theory by Terzaghi and Richart (1952) who dealt not only the actual determination of the magnitude of rock pressures but taking the magnitude of vertical and horizontal pressures, investigated the effect of their ratio, and the shape of the cavity upon the distortion resulting from pressures around it.

For the determination of stresses around a circular cavity the relationship of Kirch is applied.

$$\sigma_r = \frac{p}{2}\left[1-\frac{a^2}{r^2}\right]+\frac{p}{2}\left[1+\frac{3a^4}{r^4}-\frac{4a^2}{r^2}\right]\mathrm{Cos}\,2\theta$$

$$\sigma_\theta = \frac{p}{2}\left[1+\frac{a^2}{r^2}\right]-\frac{p}{2}\left[1+\frac{3a^4}{r^4}\right]\mathrm{Cos}\,2\theta$$

$$\sigma_{r\theta} = -\frac{p}{2}\left[1-\frac{3a^4}{r^4}+\frac{2a^2}{r^2}\right]\mathrm{Sin}\,2\theta$$

where r_1 and Q are polar coordinates and for vertical axis Q = 0

p = p_v, the uniformly distributed vertical rock pressure.

a = Radius of cavity.

p_x = k p, is uniformly distributed horizontal rock pressure.

s_r and s_Q = The radial and tangential normal stresses.

t_{rQ} = The shear stress acting in the r-Q plane.

4.3 THICKNESS OF CONCRETE LINING

The thickness of lining depends on the load consideration and tunnel shape and size. The customary thumb for thickness of concrete lining in use for a long time was 1" per foot of finished tunnel diameter or width. USBR practice (para 234, design of small dams) had been to adopt ordinarily a lining thickness of 3/4" per foot in reasonably stable ground.

It is recommended that the minimum thickness of unreinforced concrete lining be 15 cm for manual placement. Where mechanical placement is contemplated the thickness of lining shall be so designed that the slick line can be easily introduced on the top of the shutter without being constructed by steel supports. For 15 mm slick lining a clear space of 18 cm is recommended. For reinforced concrete lining, a minimum thickness of 30 cm is recommended.

4.4 DESIGN OF CONCRETE AND SHOTCRETE LINING

The design of tunnel lining is influenced by the employed excavating technique and the resulting ground disturbances, the elapsed time between excavation and support installation, the geological structural conditions and flexibility of design support system. Concrete and shotcrete (pneumatically applied concrete) are commonly used to provide support for both civil and mining structures. Concrete linings are of two types

- Cast in-situ concrete lining
- Segmental or precast concrete lining.

The inherent advantage of in-situ placed concrete is that it can be designed to accommodate any desired shape or cross section. The lining is designed to function in compression in order to minimise the need of reinforcement. Circular segmental rings also provide an immediate permanent lining of great strength. This is so provided that when erected they can be brought into close contact with the excavated ground by grouting, injection or otherwise. All the segmental type of concrete linings provide immediately on erection, the strong support and adequate flexibility. The timing of grouting operation is significant in developing the interaction between lining and ground. Szechy (1973) has collected a series of analytical solution for the structural design of concrete linings which account for uniform and non-uniform loadings of underground excavations of circular and other geometries.

Shotcrete differs from concrete cast in place by its higher compaction and its lower water cement ratio. Its application is often made on the recommendation of the different empirical systems and it is used exclusively or in combination with other reinforced methods. The use of shotcrete as a measure of support is an integral part of design philosophies such as New Austrian Method of Tunnelling (NATM). For a good design of shotcrete the following points must be considered

- its time dependent effects
- mix design
- layer thickness
- possible use of wire mesh.

4.5 STRUCTURAL DESIGN OF CONCRETE LINING

It requires the thorough study of geology of rock mass, the effective rock cover, in-situ modulus of elasticity, poisson's ratio, state of stress, crushing strength and other mechanical properties of the rock mass. The presence or absence of water in the rock being tunnelled through has a lot of influence on the design of lining. Drainage holes should be provided to drain off water. The following criteria should be adopted in the design of lining.

- P.C.C. lining should be provided until unless conditions warrant the reinforcement.
- Free flow tunnels should be provided with P.C.C. lining.
- Pressure tunnels should be reinforced.
- In competent rock where there is danger of blowout or landslides in adjacent areas due to saturation surcharge, the reinforcement is needed to be provided even when cover is between 1.0 to 0.7 H.

- Circular lining should be provided where the effective cover in good rock is less than the internal pressure head and in poor rock where effective cover is less than 1.25 times the internal head.

4.6 DESIGN LOADING

4.6.1 Water Load

The magnitude of design loads depends upon two operating conditions whether normal or emergency. Emergency conditions should not be considered as the basis for design because these are very infrequent or most unlikely to occur during the life span of the project. The design loadings are as follows.

4.6.1.1 External Design Loads

In normal design loading condition, the maximum loading is obtained either from maximum steady or steady state condition with loading equal to the normal maximum ground water pressure and no internal pressure (Applicable when no ground water drains are provided); or maximum difference in levels between the hydraulic gradient in the tunnel under steady state or static conditions and the maximum downsurge under normal transient operation.

In case of extreme design conditions, loadings are equal to maximum difference in levels between the hydraulic gradient in the tunnel under steady state and the maximum downsurge under extreme transient operation.

4.6.1.2 Internal Design Loads

Normal Design Loading Condition :

This condition is to be taken as the loading requiring maximum reinforcement in accordance with the design criteria shown in Fig 4. for either of the two cases.

Maximum static condition with maximum water level in the reservoir and no internal pressure, for condition of leakage being important, or loading equal to the difference in levels between the maximum upsurge occurring under normal transient operation and the tunnel invert, for condition of leakage being not important, as the loading is of a very short duration.

Extreme Design Loading Condition

Loading is equal to the difference between the highest level of hydraulic gradient in the tunnel under emergency transient operation and invert of the tunnel.

4.6.2 External Rock Pressure

Except in the immediate vicinity of portals, no load shall be taken due to the external rock pressure. Squeezing ground is to be considered as a special condition when encountered during excavation and is not covered by the above criteria.

4.6.3 Grout Pressure

Concrete lining should also be checked for an external pressure corresponding to 50 per cent of maximum grout pressure specified.

4.7 DESIGN CRITERIA AFTER LAUFFER AND SEEBER

The tunnel lining and the grouted rock mass are a composite construction which absorbs the internal water pressure or the internal water exceeding the external water pressure as the case may be. For an economical design, the supporting action of the rock must be utilised to the maximum extent. The following design procedure for lining is given (after Lauffer and Seeber, 1961).

4.7.1 Free Flow Tunnels

External water pressure on lining is approximately equal to the hydrostatic head in case of free flowing tunnels and the lining is designed as a thick wall lining. The following formula can be used for the design.

$$p = \frac{\sigma_c (b^2 - a^2)}{2b^2}$$

where, p = External pressure

s_c = Permissible compressive strength of concrete in direct compression test.

b = External excavated radius of the tunnel.

a = Internal finished radius of the tunnel.

t = Thickness of the tunnel = b-a

4.7.2 Pressure Tunnels

The distribution of internal pressure, p, on the rock mass and the lining is based on the secondary boundary condition that the lining and the rock mass must have the same radial deformation at their contact face. Assuming a homogeneous isotropic rock mass and a fully elastic behaviour of both the rock mass and the lining, relative deformation u/a is given by:

$$P_r = \frac{u}{a}\left[\frac{E_r m_r}{m_r + 1}\right]$$

where,

p_r and p_c = Portion of internal pressure taken by rock and lining respectively

E_r and E_c = Modulus of elasticity of rock and lining respectively

m_r and m_c = Poisson's number for rock and lining respectively

a = Internal radius of tunnel

The lining is subjected to pressure, p, from inside and rock reaction, p_r from outside. For a thin walled elastic lining u/a is dependent only on the circumferential stress s_t and is given by :

$$\frac{u}{a} = \frac{\sigma_t}{Ec\dfrac{m_c^2}{m_c^2 - 1}}$$

Since the relative deformation of the rock and the lining must be the same therefore :

$$p_r = \sigma_t \frac{E_r \dfrac{m_r}{m_r + 1}}{Ec\dfrac{m_c^2}{m_c^2 - 1}}$$

By taking s_t equal to the permissible tensile stress of the material of the lining, the above equation will give the maximum resistance offered by the rock mass.

For the case of thick walled concrete lining, it must be considered, that the actual circumferential stress along the inner edge is higher in the ratio :

$$\frac{2b^2}{(b^2+a^2)+\frac{(b^2-a^2)}{m_c}}$$

Hence the resistance offered by the rock mass becomes :

$$p_r = \sigma_t \frac{E_r \frac{m_r}{m_r+1}}{Ec\frac{m_c^2}{m_c^2-1}} x \frac{(b^2+a^2)+\frac{(b^2-a^2)}{m_c}}{2b^2}$$

Therefore, the internal pressure to be absorbed by the lining, p_c is given by :

$$p_c = p - p_r$$

The lining then can be designed by the formula :

$p_c = \sigma_t * t/a$ for a thin cylindrical lining

$p_c = \frac{\sigma_t(b^2-a^2)}{b^2+a^2}$ for a thick cylindrical lining

The permissible tensile stress in concrete taking reinforcement into account, is given by :

$$= \sigma_t \frac{t+(m-1)A_{st}}{t}$$

Where t is the thickness of concrete in cms, m the modular ratio = E_s/E_c and A_{st} is the area of steel in cm^2/cm, σ_t being in kg/cm^2.

and $A_{st} = p_c * a/\sigma_{st}$

where σ_{st} is the permissible tensile stress in steel.

4.8 DESIGN OF CONCRETE LINING USING I.S. CRITERIA

Indian Standard Institution (IS 4880, Part IV) has recommended the use of certain formulae and equations for design of concrete lining - both for external and internal pressures. The design for external loads may be done by considering the lining as independent structural member whereas the design for internal water pressure may be done by considering it as a part of composite thick cylinder consisting of peripheral concrete and surrounding rock mass subjected to specific boundary conditions.

4.8.1 Design For External Loads

A tunnel may be subjected to external loads due to rock, external water pressure, grout pressure, self weight, and weight of water contained in the tunnel as shown in the Fig. 4.1. Following formulae have been evaluated to obtain the values of bending moments, normal thrust, radial shear, horizontal and vertical deflections based on the assumption that it deflects under the active external loads and its deflection is restricted by the passive resistance developed in the surrounding rock mass. The IS procedure of concrete lining for external loads is enclosed as Appendix 4A.

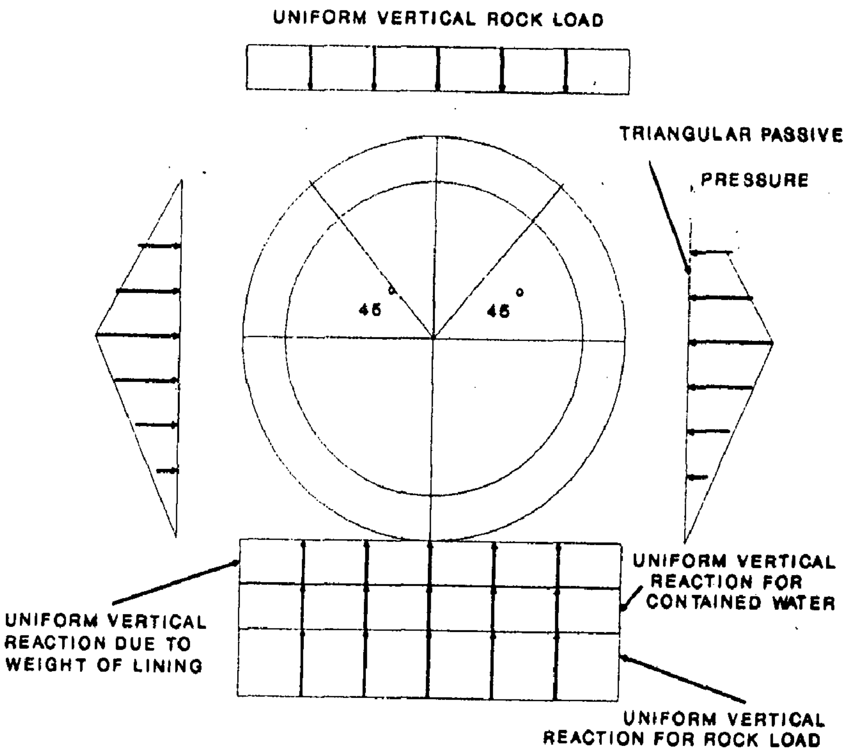


FIG. 4. : EXTERNAL LOADS ON LINING

4.8.2 Design for Internal Pressure

The basic assumption in the design is that the lining shall be considered as a part of the composite thick cylinder consisting of peripheral concrete and surrounding rock mass subjected to specific boundary conditions. To make the above assumption realistic, effective pressure grouting has to be done to fill up all the gaps and cracks in the surrounding rock mass.

Sometimes if the surrounding rock is good and cracking of lining does not involve much loss of water, the cracking of lining may be permitted to some extent. But in case of poor surrounding rock, reinforcement may be provided to reduce tensile stress in concrete thereby distributing the cracks in the whole periphery in the form of hair cracks which are not harmful.

The basic equations given below are for the design of circular section alone. For non-circular sections, it is recommended that the stress pattern may be obtained by carrying out photo-elastic studies. The equations given below may also be used for non-circular sections but the results obtained can not be regarded as true representatives of the inherent stress conditions. The procedure for the lining design for internal pressure is enclosed as Appendix 4B.

CHAPTER 5
CASE STUDIES OF CONCRETE LINING

Concrete lining for various project tunnels has been designed using Lauffer and Seeber method of concrete lining. In this case the rock participation has been taken into account. The actual lining provided has also been compared with the designed lining. Due to the limited information available, the linings have been checked by Lauffer and Seeber Criteria only.

5.1 CHHIBRO KHODRI HEAD RACE TUNNEL

The following data has been considered in the design of concrete lining :

Modulus of Elasticity of Rock Mass, E_r	= 10000 kg/cm^2
Modulus of Elasticity of Concrete	= 200000 kg/cm^2
Poisson Ratio of the rock	= 0.2
Poisson Number of the rock	= 5
Poisson Ratio of the Concrete	= 0.25
Poisson Number of the Concrete	= 4
Design head of water	= 62 m or 6.2 Kg/cm^2
Rock participation	= 1.48 kg/cm^2

As per IS 456, 1984, the permissible tensile strength of M-250 concrete is 32 Kg/cm^2. Hence allowing this tensile strength a concrete lining of 41.5 cm thick will be sufficient for the design loading. Hoop reinforcement of 40 mm diameter at 270 mm centres in two rows will be required. The longitudinal reinforcement may be provided as 0.3 % of the concrete area.

5.1.1 Actually Provided Lining

In the Chhibro Khodri head race tunnel 30 cm thick reinforced concrete lining has been provided except in the following reaches:

60 cm thick lining has been provided in

- Sheared,crushed and squeezing rocks in intra thrust zone and other portions to accommodate heavy reinforcement.

- From surge tank to junction of approach adit and head race tunnel in 75 m length at Khodri and in length of 10 m adjacent to construction shaft.

40 cm thick lining has been provided in the following reaches.

- In a length of 150 m u/s of HRT and approach adit junction at Khodri end.
- In a length of 10 m adjacent of construction shaft at Chhibro as a transition from 60 cm thickness to 30 cm thickness.

Permissible tensile strength in concrete has been adopted as 18 kg/cm^2.

5.2 RAMGANGA PROJECT TUNNELS

Ramganga tunnels have been designed for internal water pressure corresponding to 45 m.

Modulus of Elasticity of Rock Mass, E_r	= 33710 kg/cm^2
Modulus of Elasticity of Concrete	= 200000 kg/cm^2
Poisson Ratio of the rock	= 0.2
Poisson Number of the rock	= 5
Poisson Ratio of the Concrete	= 0.25
Poisson Number of the Concrete	= 4
Design head of water	= 45 m or 4.5 Kg/cm^2
Rock participation	= 3.1 kg/cm^2

As per IS 456, 1984, the permissible tensile strength of M-250 concrete is 32 Kg/cm^2. Hence allowing this tensile strength a concrete lining of 30 cm thick will be sufficient for the design loading. Hoop reinforcement of 36 mm diameter at 140 mm centres will be required. The longitudinal reinforcement may be provided as 0.3 % of the concrete area.

5.2.1 Actually Provided Lining

The concrete lining thickness provided for power tunnel varies from 300 mm to 400 mm in arch and 400 mm to 550 mm in the tunnel walls and invert depending upon the strength of the surrounding rock. The lining thickness is designed for external hydrostatic pressures equivalent to the ground water level in the area acting on the lining when the tunnel is empty. However four numbers of pressure relief holes in a ring of 4 m centres longitudinally have been assumed for the design. Hence the lining is actually designed for 10 m head of water. Reinforced in the lining is only provided at locations where the deformation modulus of the rock is less than 10000 kg/cm^2.

5.3 CHAMERA PROJECT HEAD RACE TUNNEL

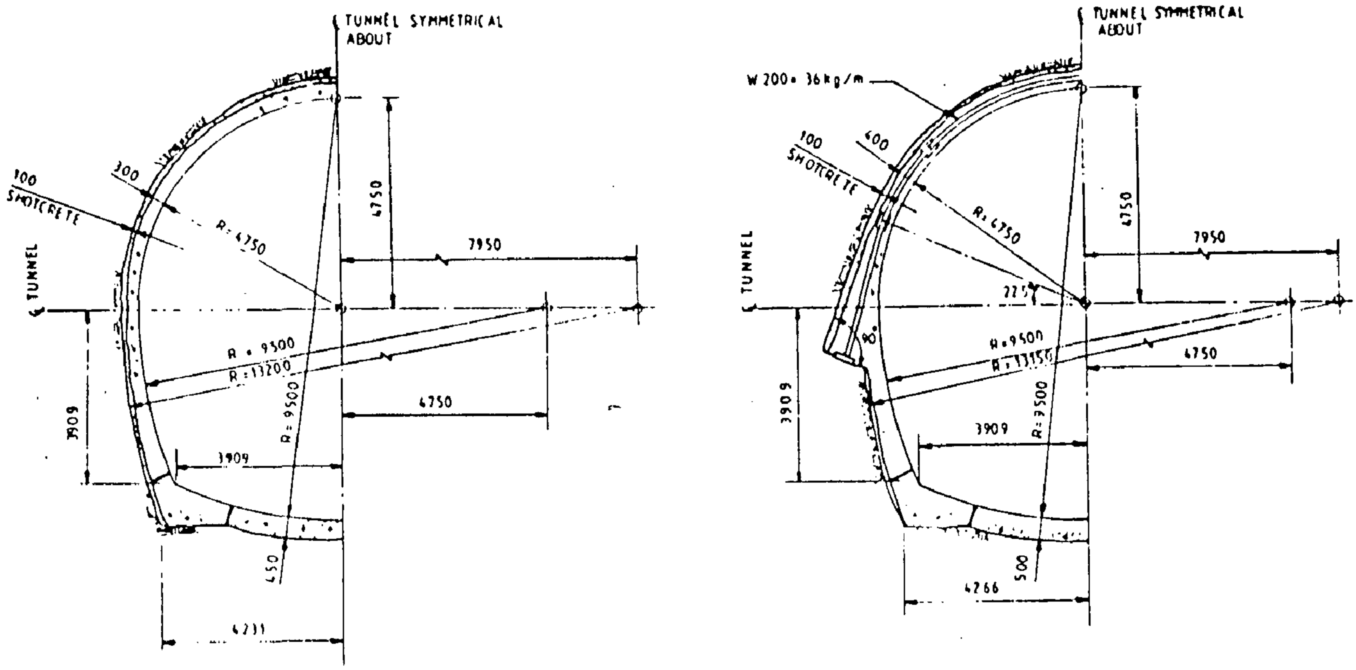
Assuming 4 nos of pressure relief holes in a ring at 4 m centre to centre longitudinally have been provided in the actual design, which has reduced the design head of water to 10 m.

Modulus of Elasticity of Rock Mass, E_r	= 30000 kg/cm^2
Modulus of Elasticity of Concrete	= 200000 kg/cm^2
Poisson Ratio of the rock	= 0.2
Poisson Number of the rock	= 5
Poisson Ratio of the Concrete	= 0.25
Poisson Number of the Concrete	= 4
Design head of water	= 10 m or 1.0 Kg/cm^2

Assuming a plain concrete lining of 25 cm thick, the tensile stress of 6.1 kg/cm^2 is developed which is within allowable limits as per IS 456, 1984.

5.3.1 Actually Provided Lining

In the case of Chamera head race tunnel plain concrete lining has been provided with thickness varying from 30 cm to 40 cm in the arch and 40 to 50 cm in the walls and invert depending upon strength of surrounding rock. The liner thickness is designed for external hydrostatic pressure equivalent to ground water level in the area acting on the liner when the tunnel is empty. Reinforced lining is provided only at locations where deformation modulus of rock is less than 10000 kg/cm^2 and also at all adit intersections. Contact grouting has been done throughout keeping grout pressure as 1.8 kg/cm^2. Five holes of 5 m long are provided in each ring at a longitudinal spacing of 4 m centre to centre for consolidation grouting and pressure adopted for this purpose varies from 3.5 to 7.0 kg/cm^2. Typical tunnel sections showing the lining details are shown in Fig. 5.1.



WITHOUT RIBS

WITH RIBS

Fig.5.1Chamera Hydroelectric Project — typical tunnel section

5.4 TEHRI DAM DIVERSION TUNNELS

The Tehri Diversion tunnels have been designed for 60 m head of water. The lining is designed as reinforced lining taking the rock participation into account.

Modulus of Elasticity of Rock Mass, E_r	= 30000 kg/cm^2
Modulus of Elasticity of Concrete	= 200000 kg/cm^2
Poisson Ratio of the rock	= 0.2
Poisson Number of the rock	= 5
Poisson Ratio of the Concrete	= 0.25
Poisson Number of the Concrete	= 4

Design head of water = 60 m or 6.0 Kg/cm^2

Rock Participation = 3.55 kg/cm^2

Assuming 60 cm thick plain concrete lining, the maximum tensile stress developed comes out to be 28.32 kg/cm^2 which is less than allowable tensile strength of M-250 concrete. However at the Kerbs some reinforcement is required.

5.4.1 Actually Provided Lining

Diversions tunnels have designed as unreinforced concrete lining of 60 cm thickness. The invert and sides have been reinforced with two layers of 25 mm diameter tor steel at 300 mm spacing.

CHAPTER 6
COMPARISON OF RECOMMENDED, ACTUALLY ACCOMMODATED AND OBSERVED SUPPORT PRESSURES

6.1 ANTICIPATED SUPPORT PRESSURES

The anticipated support pressures for twenty case studies including eight number of power houses have been worked out by different methods as in chapter 3. The support pressure which can be safely born by the supports actually provided in different case studies has also been worked out.

6.2 ACCOMMODATED SUPPORT PRESSURES BY ACTUAL SUPPORTS

The maximum actual support accommodated by the different rock supports have been worked out using the following equations.

$$P_{sbmx} = \frac{T_{bf}}{S_c S_l} \qquad \textit{for rock bolts}$$

$$P_{scmx} = \frac{\sigma_{c\text{-}conc}}{2}\left[1 - \frac{(r_i - t_c)^2}{r_i^2}\right] \qquad \textit{for shotcrete}$$

$$P_{ssmx} = \frac{3\, A_s I_s \sigma_{ys}}{2S\, r_i \theta \left[3\, I_s + x\, A_s \left[ri - (t_b + \frac{x}{2})\right](1 - \cos\,\theta)\right]} \qquad \textit{for steel supports}$$

Where

P_{sbmx} = Maximum support pressure accommodated by rock bolts.

T_{bf} = Ultimate failure load of bolts from pull out tests.

S_c = Circumferential rock bolt spacing.

S_l = Longitudinal rock bolt spacing.

P_{scmx} = Maximum support pressure accommodated by shotcrete.

$\sigma_{c\text{-}conc}$ = Uniaxial compressive strength of concrete or shotcrete

r_i = Internal radius of the opening.

t_c = Thickness of shotcrete.

P_{ssmx} = Maximum support pressure accommodated by steel supports.

x = Depth of section of steel set.

A_s = Cross sectional area of steel set.

I_s = Moment of inertia of steel section.

σ_{ys} = Yield strength of steel.

S = Steel set spacing along tunnel axis.

Θ = Half angle between blocking points.

t_b = Thickness of block.

Except two cases for which the actual supports provided at site are not available, support pressures accommodated by the actual supports have been calculated using the above equations. These support pressures have been compared with the required minimum support pressures. The results have been tabulated as in Tables 6.1 and 6.2.

6.3 OBSERVED SUPPORT PRESSURE

The observed support pressure for seven case studies have also been compared with the anticipated pressures by various methods which are presented in Table 6.3.

TABLE 6.1 : COMPARISON OF REQUIRED AND ACTUALLY ACCOMMODATED SUPPORT PRESSURES FOR UNDERGROUND POWER HOUSES

S. No.	Name of Project	Rock Type	Required Minimum Rock Pressure kg/cm^2		Maximum rock Pressure (kg/cm^2)		Actual Support Pressure Provided by Supports kg/cm^2	Ratio of Supp. Pr. Accommodated & Minimum Rock Pr.
			Supp. Pr.	Method	Supp. Pr.	Method		
1.	Nathpa Jhakri H.E. Project, H.P.	Quartz Mica Schist (Moderately Blocky)	0.42	Q System	1.35-6.5	Terzaghi's Method	2.99	7.11
2.	Sardar Sarovar Project, Gujarat	Basalt (Moderately Blocky)	0.73	Q System	1.6-4.4	Terzaghi's Method	2.80	7.84
3.	Baspa H.E. Project, H.P.	Quartzite (Massive)	0.33	RSR Method	0-2.39	Terzaghi's Method	2.76	8.36
4.	Lakhwar H.E. Project, U.P.	Dolerite to Hornblende (Massive)	0.35	Q System	0-2.65	Terzaghi's Method	3.13	8.94
5.	Yamuna Stage II, Part I, U.P. (Chhibro Power House)	Stratified Limestone (Very Blocky and seamy)	1.0-2.0	Singh et. al. 1995	4.8-5.1	Terzaghi's Method	7.18	4.79
6.	Chamera H.E. Project, H.P.	Andesite Basalt (Massive)	1.70	Q System	1.65-5.87	Terzaghi's Method	3.3	1.94
7.	Kadamparai Pumped Storage H.E. Project, Tamil Nadu	Biotite Gneiss (Massive)	0.7-1.0	Singh et. al. 1995	0-2.84	Terzaghi's Method	2.16	1.52

TABLE 6.2 : COMPARISON OF REQUIRED AND ACCOMMODATED SUPPORT PRESSURES FOR WATER TUNNELS

S. No.	Name of Project	Rock Type	Required Minimum Rock Pressure, kg/cm^2		Maximum Rock Pressure, kg/cm^2		Actual Support Pressure Provided by Supports kg/cm^2	Ratio of Supp. Pr. Accommodated & Minimum Required Supp. Pr.
			Supp. Pr.	Method	Supp. Pr.	Method		
1.	Ramganga Project Tunnels, U.P.	Sand Rock & Clay Shale(Very Blocky & Seamy)	0.7-1.0	Singh et. al. 1995	1.75-6.74	Terzaghi's Method	3.41-6.71 (Av. 5.06)	3.41-6.71
2.	Narmada Sagar Project,M.P. (HRT)	Quartzite (Massive)	0.42	Q System	0-1.2	Terzaghi's Method	4.59	10.93
3.	Giri Project, H.P. (HRT)	Slates & Phylites (Squeezing)	0.84	RMR Method	6.1-11.6	Terzaghi's Method	5.29-16.04 (Av. 10.66)	12.70
4.	Uri Project J. & K. (HRT)	Quartzitic Schist (Moderately Blocky)	1.06	Q System	0.6-1.7	Terzaghi's Method	2.70	2.55
		Panjal Volcanics (Very Blocky & Seamy)	1.67	RMR Method	5.2	Terzaghi's Method	4.54	2.72
5.	Salal H.E. Project, J. & K.	Sheared Dolomite (Very Blocky and Seamy)	2.7	RMR Method	7-13.4	Terzaghi's Method	4.71	1.74
6.	Yamuna Stage I, Part I, U.P. (HRT)	Shales with bands of Quartzite & Limestones (Massive)	0.4-0.7	Singh et. al. 1995	0.7	Terzaghi's Method	3.24	4.63-8.1

TABLE 6.2 CONTINUED ...

S. No.	Name of Project	Rock Type	Required Minimum Rock Pressure kg/cm^2		Maximum Support Pressure, kg/cm^2		Actual Support Pressure Provided by Supports, kg/cm^2	Ratio of Supp. Pr. Accommodated & Minimum Required Supp. Pr.
			Supp. Pr.	Method	Supp. Pr.	Method		
7.	Yamuna Stage II, Part II, U.P. (HRT)	Red Shales & Black Clays (Squeezing)	1.54	RMR Method	4.08-7.79	Terzaghi's Method	17.17	9.92
8.	Maneri Bhali Project, U.P. (HRT)	Moderately Fractured Quartzites (Moderately Blocky)	0.61	RMR Method	0.3-0.8	Terzaghi & Deere's Method	11.30	11.77
		Foliated Metabasics (Moderately Blocky)	0.6	RMR Method	0.4-0.8	Terzaghi & Deere's Method	8.48	16.96
		Sheared Metabasics (Very Blocky & Seamy)	0.74	RMR Method	0.8-2.6	Terzaghi's Method	6.59	8.91
		Highly fractured Quartzites (Squeezing)	0.9	RMR Method	2.77-5.29	Terzaghi's Method	15.82	17.58
9.	Khara Project, U.P. (HRT)	Phylites (Massive)	0.4	Q System	1.48	Deere's Method	7.89	19.72
		Argillaceous Conglomerates (Squeezing)	3.25	Q System	3.5-6.68	Terzaghi's Method	15.77	4.85
		Calcareous Conglomerates (Moderately Blocky)	0.5-1.3	Terzaghi's Method	0.7-1.7	Q System	11.83	13.14
10.	Tehri Project, U.P. (Diversion Tunnels)	Phylite Gr. I (Massive)	1.2	Q System	0.95-1.9	Terzaghi's Method	4.83	4.02
		Phylite Gr. II (Moderately Blocky)	1.2	Q System	0.73-2.04	Terzaghi's Method	5.38	4.48
		Phylite Gr. III (Very Blocky & Seamy)	1.2	Q System	2.22-7.00	Deere's Method	3.51	2.93

TABLE 6.3 : COMPARISON OF CALCULATED AND OBSERVED SUPPORT PRESSURES

Rock Type	Project	Rock Description	Calculated Pressures in kg/cm^2 by				Observed Pressures kg/cm^2
			Terzaghi's Method	Deere's	RMR Method	Q System	
Massive to Moderately Jointed Rocks	Tehri Diversion Tunnels	Phylites Grade I	0-1.8	0.95-1.9	--	1.2	0.25
		Phylites Grade II	0.73-2.04	0.95-1.9	--	1.2	0.52
	Maneri Bhali Project Tunnel (HRT)	Foliated Metabasics	0.3-0.8	0.4-0.8	0.60	0.5-0.7	0.8
Very Blocky and Seamy Rocks	Salal Project Tunnel (TRT)	Highly Jointed Dolomites	2.23-7.0	--	2.2	2.3	1.1
	Tehri Diversion Tunnels	Phylites with bands of Argillaceous Materials	0.9-2.0	4.13-6.36	--	1.2	1.24
	Maneri Bhali Tunnel (HRT)	Sheared Metabasics	0.8-2.6	0.8-1.73	0.74	0.7-1.8	2.0
		Fractured Quartzites	0.3-0.8	0.8	0.61	0.5-0.7	0.6
Very Poor Rocks (Squeezing and Swelling)	Khara Project Tunnels	Argillaceous Conglomerates	4.4	--	--	3-3.5	0.75-3.00
	Giri Project Tunnel (HRT)	Slates	2.4-4.6	1.88-3.12	0.7	2.4	2.0
		Phylites	6.1-11.6	7.09	0.84	3.4	1.7
	Loktak Tunnel (HRT)	Sandstone Shale and Siltstone	6.1-11.6	7.0	1.17	4.7	5.4
	Chhibro Khodri Tunnel (HRT)	Red Shales	1.8-3.4	--	1.54	3.5	10.8
		Black Clays	1.8-3.4	--	1.73	7.0	3.2

CHAPTER 7
CONCLUSIONS

On the basis of this study the following conclusions can be drawn :

7.1 ROCK SUPPORTS

7.1.1 EXCAVATIONS OF WIDTH MORE THAN 12 M (POWER HOUSES)

The case records of the power houses which have been taken for study here, have been excavated in massive to very blocky and seamy rocks.

1. From Table 6.1 (chapter 6) it can be inferred that :

    a) Q system gives the minimum rock pressures.

    b) Terzaghi's method give the maximum rock pressures.

    c) The ratio of accommodated rock pressure to that of anticipated minimum support pressure varies between 4 and 8 except two cases where this ratio is of the order of 1.5 to 2.0.

7.1.2 EXCAVATIONS OF WIDTH UPTO 12 M (WATER TUNNELS)

1. From table 6.2 (chapter 6) it can be inferred that :

    a) For hard intact, massive and moderately blocky rocks Q system gives the minimum rock pressures.

    b) For very blocky, very seamy and shattered rocks, RMR gives the minimum rock pressures.

    c) For squeezing and swelling rocks, RMR gives the minimum rock pressures.

    d) The ratio of accommodated support pressure to the minimum anticipated support pressure for ten case studies varies between 2 to 14 in general.

    e) In case of about 90 per cent cases, Terzaghi's method gave higher rock pressures.

7.1.3 Observed Rock Pressures

From Table 6.3 (chapter 6) it can be inferred that :

a) In case of massive to moderately jointed rocks :

- Q method gave the minimum rock pressures for Tehri Diversion Tunnels (Phylites of grade I and II), and observed pressures are between 30 percent to 50 percent of the calculated values.
- For Maneri Bhali Tunnel the observed pressures are almost equal to the calculated pressures by Q system.

b) For very blocky, seamy and crushed rocks

- Observed pressures are more nearer to calculated rock pressures by Q system in case of Tehri Diversion Tunnels, and Maneri Bhali head race tunnel (Fractured Quartzite and sheared metabasics rocks).
- In case of Salal project tail race tunnel, observed rock pressure is about half of that given by Q system. The calculated rock pressures by RMR and Q systems are of the same order.
- Terzaghi's method gives higher rock pressures in all cases.

c) Squeezing and swelling rocks

- RMR method gives the least calculated rock pressures but the observed pressures are by and large nearer to those calculated by Q system.

Thus it can be inferred that if rock pressures are calculated by Q system, these are expected to envelop the observed pressures. Hence from this study it can be concluded that Q system has an edge over other rock classifications for rock support design.

7.2 CONCRETE LINING IN WATER TUNNELS

Lauffer and Seeber criteria has been used for the design of concrete linings and it seems that the results are well matching with the actually provided lining. The thickness of concrete lining worked out using this method is in close proximity with that provided in the tunnels.

However, reinforcement provided in the actual designs seems to be more than the recommended. It is because of the fact that the allowable tensile stress in the concrete has been taken on the lower side.

7.3 RECOMMENDATIONS AND FURTHER SCOPE

Most of the case studies taken in this study have been supported on steel sets with few exceptions. In one case study the fibre reinforced shotcrete has been used. Using rock bolts and shotcrete instead of steel sets, will be economical. Fibre reinforced shotcrete gives better flexibility.

Though twenty case histories have been taken but instrumented observed rock pressure data is available for seven case studies. It would be worthwhile to compare the calculated rock pressures with more observed rock pressures to give conclusive recommendations about which method is more applicable in different types of rocks.

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APPENDIX 3A

RSR CLASSIFICATION OF ROCK AT NATHPA JHAKRI POWER HOUSE

Basic Rock Type : Quartz Mica Schist (Type II)

Parameter		Rating
Parameter A		13
Parameter B		24
	Sum of Parameters A and B	37
Parameter C		15
	Total Rock Structure Rating (RSR)	52

APPENDIX 3B

RMR CLASSIFICATION OF ROCK AT NATHPA JHAKRI POWER HOUSE

A. Classification of parameters and their rating

S.No	Parameter	Rating
1.	Strength of intact rock material	12
2.	Drill core quality (RQD), 70 %	13
3.	Spacing of discontinuities	8
4.	Condition of discontinuities	25
5.	Groundwater inflow	7

B. Rating adjustment for discontinuities, orientation strike and dip orientation of discontinuities -5

	Total Rock Mass Rating	60
	Class of rock	III
	Description	Fair

APPENDIX 3C

RSR CLASSIFICATION OF ROCK AT BASPA STAGE II POWER HOUSE

Basic Rock Type : Quartzite (Type II)

Parameter		Rating
Parameter A		27
Parameter B		38
	Sum of Parameters A and B	65
Parameter C		23
	Total Rock Structure Rating (RSR)	78

APPENDIX 4A

FORMULAE FOR VALUES OF BENDING MOMENT, NORMAL THRUST, RADIAL SHEAR, HORIZONTAL AND VERTICAL DEFLECTION

The following notations are used for the design of linings.

- E - Young's modulus of elasticity
- I - Moment of inertia of the section
- K - Intensity of lateral triangular load at horizontal diameter
- P - Total rock load on mean diameter
- r - Internal radius of the tunnel
- R - Mean radius of tunnel lining
- t - Thickness of lining
- W - Unit weight of water
- W_c - Unit weight of concrete; and
- ϕ - Angle that the section makes with the vertical diameter at the centre measured from the invert.

The following design conventions shall apply for these formulae:

i) Positive moment indicates tension in inside face and compression in outside face.

ii) Positive thrust means compression on the section.

iii) Positive shear means that considering left half of the ring the sum of all the forces on the left of the section acts outwards when viewed from inside.

iv) Positive horizontal deflection means outward deflection with reference to centre of conduit.

v) Positive vertical deflection means downward deflection.

TABLE 3.1 VALUES OF BENDING MOMENTS

ϕ	Uniform Vertical Load	Conduit Weight	Contained water	Lateral Pressure
0°	+0.125 PR	$+0.4406\ W_c tR^2$	$+0.2203\ Wr^2R$	$-0.1434\ KR^2$
45°	Zero	$-0.0334\ W_c tR^2$	$-0.0167\ Wr^2R$	$-0.0084\ KR^2$
90°	-0.125 PR	$-0.3927\ W_c tR^2$	$-0.1963\ Wr^2R$	$+0.1653\ KR^2$
135°	Zero	$+0.0334\ W_c tR^2$	$+0.0167\ Wr^2R$	$-0.0187\ KR^2$
180°	+0.125 PR	$+0.3448\ W_c tR^2$	$+0.1724\ Wr^2R$	$-0.1295\ KR^2$

TABLE 3.2 VALUES OF NORMAL THRUST

ϕ	Uniform Vertical Load	Conduit Weight	Contained water	Lateral Pressure
0°	Zero	+0.1667 W_ctR	-1.4166 Wr^2	+0.4754 KR
45°	+0.250 P	+1.1332 W_ctR	-0.7869 Wr^2	+0.3058 KR
90°	+0.500 P	+1.5708 W_ctR	-0.2146 Wr^2	Zero
135°	+0.250 P	+0.4376 W_ctR	-0.4277 Wr^2	+0.2674 KR
180°	Zero	-0.1667 W_ctR	-0.5834 Wr^2	+0.3782 KR

TABLE 3.3 VALUES OF RADIAL SHEAR

ϕ	Uniform Vertical Load	Conduit Weight	Contained water	Lateral Pressure
0°	Zero	Zero	Zero	Zero
45°	-0.250 P	-0.8976 W_ctR	-0.4488 Wr^2	+0.3058 KR
90°	Zero	+0.1667 W_ctR	+0.0833 Wr^2	-0.0246 KR
135°	+0.250 P	+0.6732 W_ctR	+0.3366 Wr^2	-0.2674 KR
180°	Zero	Zero	Zero	Zero

TABLE 3.4 VALUES OF HORIZONTAL DEFLECTION

ϕ	Uniform Vertical Load* PR^3/EI	Conduit Weight * W_ctR^4/EI	Contained water * W r^2R3/EI	Lateral Pressure * KR^4/EI
0°	Zero	Zero	Zero	Zero
45°	+0.01473	+0.05040	+0.02520	-0.01750
90°	+0.04167	+1.14090	+0.06545	-0.05055
135°	+0.01473	+0.04216	+0.02108	-0.01624
180°	ZERO	Zero	Zero	Zero

TABLE 3.5 VALUES OF VERTICAL DEFLECTION

ϕ	Uniform Vertical Load * PR^3/EI	Conduit Weight * W_ctR^4/EI	Contained water * W r^2R3/EI	Lateral Pressure * KR^4/EI
0°	Zero	Zero	Zero	Zero
45°	+0.02694	+0.09279	+0.04640	-0.03176
90°	+0.04167	+0.13917	+0.06958	-0.04995
135°	+0.05640	+0.18535	+0.07268	-0.06810
180°	+0.08333	+0.26180	+0.13090	-0.09739

APPENDIX 4B
BASIC EQUATIONS FOR ANALYSIS OF TUNNEL LINING CONSIDERING IT AND SURROUNDING ROCK AS A COMPOSITE CYLINDER

The following notations shall apply for the equations:

P	= Internal hydrostatic pressure.
$\sigma_{t1}, \sigma_{t2}, \sigma_{t3}$	= Tangential stress in rock, concrete and steel respectively.
E_1, E_2, E_3	= Modulus of elasticity of rock, concrete and steel respectively.
m_1, m_2	= Poison's ratio of rock, concrete and steel respectively.
x	= Radius of the element.
B,C etc.	= Constants of integration.
A_s	= Areas of reinforcement for unit length of tunnel
a	= Internal diameter of the tunnel.
b	= External diameter of lining upto minimum excavation line.

Case 1 : Plain Cement Concrete Lining Considering that it is not Cracked

a) Basic equations :

$$\sigma_r = \frac{mE}{m^2-1}\left[B(m+1) - \frac{C}{x^2}(m-1)\right]$$

$$\sigma_t = \frac{mE}{m^2-1}\left[B(m+1) + \frac{C}{x^2}(m-1)\right]$$

$$U = Bx + \frac{C}{x}$$

b) Limit Conditions and Constants :

(1) When $x = \infty$, $\sigma_{r1} = 0$
(2) When $x = b$, $\sigma_{r1} = \sigma_{r2}$
(3) When $x = a$, $\sigma_{r1} = -p$
(4) When $x = b$ $U_1 = U_2$

Case 2 : **Plain Cement Concrete Lining Considering that it is Cracked**

a) Basic equations for rock:

$$\sigma_{r1} = \frac{m_1 E_1}{m_1^2 - 1}\left[B_1(m_1+1) - \frac{C_1}{x^2}(m_1 - 1)\right]$$

$$\sigma_{t1} = \frac{m_1 E_1}{m_1^2 - 1}\left[B_1(m_1+1) + \frac{C_1}{x^2}(m_1 - 1)\right]$$

$$U = Bx + \frac{C}{x}$$

b) Basic equations for concrete

$$\sigma_{r2} = \frac{a(\sigma_{r2})_{x=a}}{x}$$

$$\sigma_{t2} = 0$$

c) Limit Conditions :

(1) When $x = \infty$, $\sigma_{r1} = 0$
(2) When $x = b$, $\sigma_{r1} = \sigma_{r2}$
(3) When $x = a$, $\sigma_{r1} = -P$

d) Constants of integration are calculated as :

$$B_1 = 0$$

$$C_1 = \frac{abp(m_1+1)}{m_1 E_1}$$

$$(\sigma_{r2})_{x=a} = -p$$

Case 3 : **Reinforced Cement Concrete Lining Considering that it is not Cracked**

a) Basic equations :

$$\sigma_r = \frac{mE}{m^2 - 1}\left[B(m+1) - \frac{C}{x^2}(m-1)\right]$$

$$\sigma_t = \frac{mE}{m^2 - 1}\left[B(m+1) + \frac{C}{x^2}(m-1)\right]$$

$$U = Bx + O/x$$

$$\sigma_{t3} = \frac{E_3}{a}\left[B_2 a + \frac{C_2}{a}\right]$$

$$\sigma_{r3} = \frac{E_s A_s}{a^2}\left[B_2 a + \frac{O_2}{a}\right]$$

b) Limit Conditions and Constants :

(1) When $x = \infty$, $\sigma_{r1} = 0$
(2) When $x = b$, $\sigma_{r1} = \sigma_{r2}$
(3) When $x = a$, $\sigma_{r2} - \sigma_{r3} = -p$
(4) When $x = b$ $U_1 = U_2$

c) Constants are given by :

$$C_1 = B_2 b^2 + O_2$$

$$C_2 = \left[\frac{E_2 m_2 (m_1+1)}{E_1 m_1 (m_2+1)}\right] B_2 - \left[\frac{E_2 m_2 (m_1+1)^2}{E_1 m_1 (m_2-1)}\right] C_2$$

$$-p = \left[\frac{E_2 m_2}{(m_2-1)} - \frac{E_3 A_s}{a}\right] B_2 - \left[\frac{E_2 m_2}{a^2 (m_2-1)} + \frac{E_3 A_s}{a^3}\right] C_2$$

Case 4 : Reinforced Cement Concrete Lining Considering that it is Cracked Because of Radial Cracks it Cannot Take Tangential Stress

a) Basic equations :

$$\sigma_{t1} = \frac{E_1 m_1 C_1}{(m_1+1)^2 x^2}$$

$$\sigma_{r1} = -\sigma_{t1}$$

$$U = C_1/x$$

b) Basic equations for concrete

$$\sigma_{t1} = 0$$

$$\sigma_{r2} = \frac{a(\sigma_{r2})_{x=a}}{x}$$

$$U_2 = \frac{a(\sigma_{r2})_{x=a}}{E_2} \log \frac{b}{a}$$

c) Basic equations for steel :

$$\sigma_{t3} = \frac{a\,\sigma_{r3}}{A_s}$$

$$\sigma_{r3} = \frac{E_s A_s}{a^2}\left[B_2 a + \frac{C_2}{a}\right].$$

$$U_3 = \frac{a^2 \sigma_3}{E_s A_s}$$

d) Constants of Integration :

$$[\sigma_{r3}]_{x=a} = \frac{-pa\, m_1 E_1 E_2}{a\, m_1 E_1 E_2 + m_1 E_1 E_3 A_s \log(\frac{b}{a}) + (m_1 + 1) E_2 E_3 A_s}$$

$$C_1 = \frac{-ab(m_1 + 1)[\sigma_{r2}]_{x=a}}{m_1 E_1}$$

$$\sigma_r 3 = [\sigma_{r2}]_{x=a} + p$$