ROCK MASS CHARACTERIZATION AND EVALUATION OF SUPPORTS FOR TUNNELS IN HIMALAYA

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WATER RESOURCES DEVELOPMENT

By

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October, 2002

Dedicated to

CHANDRASHEKHAR BABA

(My Guru Maharaj)

CANDIDATE'S DECLARATION

I hereby certify that the work which is being presented in the thesis entitled **ROCK MASS CHARACTERIZATION AND EVALUATION OF SUPPORTS FOR TUNNELS IN HIMALAYA** in fulfilment of the requirement for the award of the Degree of Doctor of Philosophy and submitted in the **Water Resources Development Training Centre** of the **Indian Institute of Technology Roorkee** is an authentic record of my own work carried out during a period from **January 22, 1999** to **October 2002** under the supervision of **Prof. Gopal Chauhan, Dr. R. Anbalagan** and **Dr. N.K. Samadhiya.**

The matter presented in this thesis has not been submitted by me for the award of any other degree of this or any other institute/university.

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ABSTRACT

Engineers in the recent times look towards empirical methods for selection and design of tunnel supports due to the complexities and uncertainties inherent in the analytical design methods. Engineering rock mass classification (RMC) is the best-known empirical approach for assessing the stability of underground openings. This approach has got enormous potential and forms the backbone of present day rock engineering. As a matter of fact, almost all the modern underground constructions are utilizing RMC approach due to its simplicity and practical approach.

Out of several systems, Rock Structure Rating (RSR), Rock Mass Rating (RMR), and Rock Mass Quality (Q) systems present quantitative methods for describing the quality of a rock mass for selecting the appropriate ground support. Rock Mass index (RMi) system, recently proposed by Palmstrom (1995a) is also a quantitative method; applicability of which is yet to be established in Himalaya.

The head race tunnel (HRT) of Nathpa Jhakri Project (NJP) is 27.4 km long with a diameter of 10.15 m. It is the longest power tunnel in the world being constructed in Higher Himalaya. Excavation of this long tunnel has imposed many challenges to the field engineers. These include problems related to geothermic, heavy inflow of groundwater, excessive rock covers, flowing, slabbing and squeezing ground conditions. Rock types in the area encompassed by the project comprise a variety of metamorphic rocks like gneisses, schistose gneiss, schist, quartzite and basic intrusive (amphibolites), granite and pegmatite belonging to Jeori Wangtu Gneissic Complex of Precambrian age.

In the present study, different RMC systems such as RSR, RMR, Q and RMi have been applied besides use of Rock Mass Number (N) and Rock Condition Rating (RCR). In all 685 tunnel sections covering a total length of 22159 m have been considered. A new interactive

computer program **ROMAC** has been developed in C++ language for classification using RSR, RMR, Q and RMi systems. Correlations have also been developed between different classification systems.

The RSR concept may be viewed as an improvement of Terzaghi's method rather than an independent system. Wickham et al. (1972) described rock mass quantitatively in the form of RSR-value whereas Terzaghi (1946) described it qualitatively.

The RSR uses parameter 'A', which is estimated from the rock hardness and geological structure in addition to rock type origin. Wickham et al. (1972) have given qualitative terms for the hardness of rock such as hard, medium, soft and decomposed. In the present work, hardness has been related with uniaxial compressive strength (UCS) of intact rock. Deere and Miller (1966) classification has been utilized for the purpose.

Wickham et al. (1974) have given recommendations for estimation of spacing of steel ribs as per FPS system of units for different tunnel diameters for datum condition. Moreover, these recommendations have been made for tunnel diameters up to 9 m only. Therefore, to facilitate use of steel ribs manufactured in India as per Indian Standards, a chart has been developed to get the spacing of various steel ribs for datum condition, beyond 9 m also.

On the basis of recommendations given by Bieniawski (1989), equations have been developed to interpolate support details for a particular RMR-value. The support recommendations given by Bieniawski (1989) simply state light, medium and heavy ribs. It does not give formulation for estimation of pressure. It has been found from the present study that on transition from 'Fair' to 'Poor' rock class, there is a huge jump in terms of support capacity due to the introduction of steel ribs for 'Poor' rock class.

The recommendations given by Barton et al. (1974) do not suggest stress reduction factor (SRF) values for 'competent rock, rock stress problems' category for moderately jointed rocks in categories L, M and N. In the present work, SRF-values selected are 9, 15 and 20 respectively for these. But in all these sections rocks are moderately jointed (2 joint sets or more) not massive. Consequently, Q-values might be erroneous. Therefore, new SRF-values have been proposed in moderately jointed rocks and a correlation has also been developed for the same.

RMi uses a number of parameters and computations involved in its estimation are complex. In estimation of RMi, volume of block (V_b) plays a very significant role. According to Palmstrom (1995), RMi is a volumetric parameter indicating the approximate UCS of a rock mass. A comparison between UCS of rock mass (q_{cmass}) and RMi indicates that RMi-values are far less than q_{cmass} (Singh et al., 1992) and no correlation could be established between the two. Reason for this may be that V_b is very low for the rock mass considered in the present study. RMi consists of UCS of intact rock (q_c) and jointing parameter (JP), which is dependent on V_b and joint condition factor (jC). In jC roughness and alteration are almost similar to those considered by Barton (1974). Therefore, V_b is the only parameter, which might have influenced estimation of RMi. It means that RMi is very sensitive to V_b in jointed rock masses or in other words its proper estimation is heavily dependent on the correct evaluation of V_b.

Estimation as per Unal (1983) gives higher pressures in majority of sections compared to RCR, Q and N but lesser pressures compared to RSR in non-squeezing ground conditions. For squeezing ground conditions, these are far lower than the capacity of installed support. Earlier also, Goel and Jethwa (1991) reported that the estimated support pressures as per Unal (1983) were unsafe under squeezing ground conditions.

Pressures estimated from RMR as per Goel and Jethwa (1991) and from RCR as per Goel (1994) for non-squeezing ground conditions are slightly on lower side as compared to N. Therefore modified correlations have been proposed. Further, it has been found that pressure estimated from

Q has been less than N in squeezing ground conditions. An excellent correlation is obtained between Q and N, once all the corrections are applied in Q as suggested by Singh et al. (1992).

Palmstrom (2000a) proposed charts for the design of support in blocky and continuous grounds separately, without any correlation for estimation of roof pressure. In the present work, a correlation has been developed for estimation of roof pressure for blocky ground.

A general trend with regard to pressure estimation may be given as follows:

Support Pressure (Non-Squeezing): $P_{RCR} < P_Q \leq P_N < P_{RMR} < P_{RSR}$

Support Pressure (Squeezing): $P_{RMR} < P_{RCR} < P_Q < P_N$

It is difficult to compare four classification systems from support point of view since different types of supports have been recommended by them. Singh et al. (1995b) have proposed a semi - empirical method for the design of supports for tunnels and caverns. From the present study, it has been found that semi - empirical method is not applicable beyond 0.25 MPa pressure. A new correlation has been developed which may render original method applicable for higher pressures as well. Support charts for estimating capacity of shotcrete/SFRS and rock bolt systems have also been developed from the database contained in this thesis.

Capacity of support recommended by RSR and that installed by the Project Authorities match only in about 10 percent sections whereas in about 77 percent sections, capacity of support installed by the Project Authorities is less than that recommended by RSR. This indicates that in non-squeezing ground conditions RSR overestimates support requirements. Further it has been found that capacity of support recommended by RMR-system has been the least in both non-squeezing and squeezing ground conditions.

It has also been found that capacity of support recommended by the Q-system is less compared to that installed by the Project Authorities and interestingly capacity of support installed by the Project Authorities is less compared to pressure estimated from N.

Also it has been found that capacity of support recommended by the RMi-system has been highest of all. The capacity of support is so high that in about 82 percent sections, support installed by the Project Authorities are of lower capacity than that recommended by RMi in non-squeezing ground conditions. On the other hand in squeezing ground conditions, in about 57 percent sections, the Project Authorities have installed lesser supports and in the rest about 43 percent sections greater supports than those recommended by RMi.

A general trend with regard to support capacity may be given as follows:

Support Capacity (Non-Squeezing): $\mathbf{C_{RMR} < C_Q < C_{RSR} << C_{RMi}}$

Support Capacity (Squeezing): $\mathbf{C_{RMR} < C_Q < C_{RMi}}$

The Project Authorities have classified and designated the rock masses using the Q-system. Later, following the same designation they tried to choose supports using the RMR-system. From the study of tunnel sections in squeezing ground conditions, it has been found that majority of sections are unstable, since pressures after corrections as per Singh et al. (1992) are more than the support capacity. At one of the locations bending/twisting of ribs has been observed due to the fact that pressure after corrections is more than the support capacity notwithstanding the fact that pressure before corrections is less than the support capacity. It is feared that other sections might also show signs of instability in future where pressures exceeded capacities of supports installed.

A general trend with regard to position of supports installed by the Project Authorities may be given as follows:

Support Capacity (Non-Squeezing): $C_{RMR} < P_{RCR} < \mathbf{C_{PROJECT}} < P_N = C_Q < C_{RSR} < C_{RMi}$

Support Capacity (Squeezing): $C_{RMR} < P_{RCR} < C_Q < \mathbf{C_{PROJECT}} < P_N < C_{RMi}$

Rock mass – support interaction analysis is a well-known method of design of tunnel support system. It is considered as one of the most promising methods for understanding the mechanics of tunnel deformation and development of rock loads. The interactive computer programs have been developed in C++ language for determination of:

- Ground Response Curve (GRC) as per the formulations of Ladanyi (1974) and Brown et al. (1983).
- Support Reaction Curve (SRC) for shotcrete/rock bolt and steel rib/concrete supports.

From the study it has been found that: (i) GRC is very sensitive towards RMR or Geological Strength Index (GSI) values, (ii) In 'Good rock', all the classification systems recommended supports whereas through the analysis it has been found that hardly any support is needed, as deformations are very small, (iii) In 'Fair rock', supports recommended by RMR are usually inadequate whereas those by RMi and Q should be installed after some initial deformation, (iv) In 'Poor rock' only steel ribs, as recommended by RSR and RMR systems are adequate for stability whereas supports recommended by Q are inadequate, (v) In all the sections considered, rock mass behaviour has been found to be elasto-plastic in non-squeezing ground conditions also, where it should have been elastic, (vi) GRC using N lies far above GRC using RCR in all the sections indicating that RCR underestimates pressures in squeezing ground conditions and (vii) Supports recommended by RMR are adequate whereas those recommended by Q prove inadequate in squeezing ground conditions.

In the present work, it has been found from the rock mass - support interaction analysis that for 'Good' and 'Fair' rocks (using RMR-system), observed and predicted deformations are in close agreement. Deformation modulii have been computed from the correlation of Verman et al. (1993).

Therefore it may be concluded that deformation modulii estimated from RMR for 'Good' and 'Fair' rocks appear to be accurate.

According to Singh et al. (1998), Mohr's theory is not applicable for anisotropic and jointed rock masses on account of significant strength enhancement due to in-situ stress along tunnels. In the present work, an attempt has been made to evaluate the effect of strength enhancement in rock mass and then compare it with Mohr's criterion. The tunnel sections chosen for this purpose lie under a rock cover of more than 1000 m with a maximum value of 1430 m. Rock mass contains two or more joint sets implying that this study pertains to jointed rock mass only. The study shows that heavy rock burst predicted by Mohr's criterion is moderated to a great extent if Singh's criterion is adopted. During the excavation of these tunnel reaches also, it has been observed that behaviour of rocks actually follows Singh's criterion. At none of the sections, there were heavy rock burst since rock mass is jointed and also due to the enhancement of strength in rock masses, there have been no stress related problems, except slabbing with cracking noise.

Prediction of ground condition prior to the actual excavation can be extremely useful. In the present work therefore, correlations have been developed for prediction of ground conditions based on: (i) tangential stress $(\sigma_\theta)/q'_{cmass}$, (ii) Q and N and (iii) joint roughness number (J_r) and joint alteration number (J_a).

From the overall analysis of the classification performed in the present study, it has been found that revised Q-system is the best.

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a	half tunnel diameter or span
A_s	cross-sectional area of steel section
B	tunnel diameter or span
c	cohesion
C	a gravity adjustment factor for support in the roof or in the walls
C_g	competency factor
C_o	an adjustment factor for the main joint set
C_{os}	an adjustment factor for shear seam
$C_{PROJECT}$	capacity of support installed by the Project Authorities
C_Q	capacity of support recommended by the Q-system
C_{RMi}	capacity of support recommended by the RMi-system
C_{RMR}	capacity of support recommended by the RMR-system
C_{RSR}	capacity of support recommended by the RSR-system
d_b	diameter of rock bolt
D_b	equivalent block diameter
D_e	unsupported span
E	modulus of elasticity of rock mass
E_b	modulus of elasticity of block material
E_c	modulus of elasticity of concrete
E_d	modulus of deformation of rock mass
E_m	average value of E_d
E_r	modulus of elasticity of rock material
E_s	modulus of elasticity of steel
E_{sr}	E_d of the surrounding rock mass
E_{wz}	E_d of the weak zone
f(N), f(RCR)	correction factor for tunnel closure (Table 6.1)
f', f' and f''	correction factors for overburden, tunnel closure and the time after excavation respectively (Eq. 6.26)
f_σ	an adjustment factor for the scale effect of compressive strength in massive rock

F_{gt}	mobilization factor of grouted arch
F_s	mobilization factor of bolt or anchor
F_{sc}	mobilization factor for shotcrete
G_c	ground condition factor
H	tunnel depth or thickness of overburden
H_p	height of the loosened rock mass above tunnel crown developing load
H_t	height of opening
I_s	moment of inertia of steel section
jA	joint alteration factor
jC	joint condition factor
jL	joint size factor
jR	joint roughness factor
J_a	joint alteration number for critically oriented joint set
J_{am}	weighted average of J_a
J_n	joint set number
J_r	joint roughness number for critically oriented joint set
J_{rm}	weighted average of J_r
J_{rsr}	J_r of the surrounding rock mass
J_v	total number of joints per cubic metre
J_w	joint water reduction factor
K	ratio of horizontal stress and vertical stress
l_{bolt}	length of bolt or anchor
l_{gt}	thickness of grouted arch
l'	effective thickness of reinforced arch
m and s	constants dependent on the properties of the rock and the extent to which it has been fractured by being subjected to major and minor principal stresses
m_i	constant m for intact rock
m_r and s_r	m and s for broken rock mass
n_j	number of joint sets
N_j	an adjustment factor for the number of joint set
p_h	horizontal support pressure

p_o	in-situ stress
p_v	vertical support pressure
P_{bolt}	capacity of rock bolt
P_{gt}	capacity of grouted rock arch
P_N	support pressure estimated from N (Eqs. 6.29 and 6.30)
P_Q	support pressure estimated from Q (Eqs. 6.23 and 6.24)
P_{RCR}	support pressure estimated from RCR (Eqs. 6.14 and 6.15)
P_{rib}	capacity of steel rib
P_{RMR}	support pressure estimated from RMR (Eq. 6.12)
P_{RSR}	support pressure estimated from RSR (Eq. 6.6)
P_{sc}	capacity of shotcrete/SFRS
q_c	uniaxial compressive strength of the intact rock material
$q_{c.conc}$	uniaxial compressive strength of shotcrete, concrete or SFRS
q_{cmass}	uniaxial compressive strength of rock mass
q_{crm}	minimum uniaxial compressive strength of reinforced rock mass
q_{gt}	uniaxial compressive strength of grouted rock mass
q_{sc}	shear strength of shotcrete/SFRS
q_t	tensile strength of the intact rock material
Q	Barton's rock mass quality
Q_c	Q-value normalized by $q_c/100$
Q_m	mean value of Q
Q_{sr}	Q-value of the surrounding rock
Q_{wz}	Q-value of the weak zone
r	coefficient of correlation
R	a quantity related to the load-deformation characteristics of the anchor, washer plate and bolt head
S_{bolt}	centre to centre spacing of bolts/anchors
S_l	longitudinal rock bolt spacing
SL	a stress level adjustment
S_r	size ratio
S_{rib}	spacing of ribs

S_{rock}	average spacing of fractures in rock
S_{rs}	size ratio with shear seam
t	time after excavation
t_B	thickness of block
t_{sc}	thickness of shotcrete/SFRS
T	load carrying capacity or thrust of the reinforced rock arch
T_{bf}	ultimate strength of bolt system
T_s	thickness of shear seam
T_z	thickness of shear zone
u_a	radial tunnel closure
U	seepage pressure
V_b	block volume
W	flange width of steel set
X	depth of steel section
2α	angle between blocking points
β	block shape factor
γ	unit weight of the rock mass
δ	angle (dip) of the opening surface measured from horizontal
2θ	angle of bolted rock arch at the center (Fig 7.1)
ν	Poisson' ratio of rock mass
ν_c	Poisson's ratio of concrete
σ_1	major principal stress at failure
σ_2	intermediate principal stress
σ_3	minor principal stress
σ_h	horizontal stress
σ_v	vertical stress
σ_{ys}	yield strength of steel
σ_θ	tangential stress around the opening
ϕ	angle of internal friction
ϕ_p	peak angle of internal friction of rock mass (14^0-57^0)

LIST OF ABBREVIATIONS

CCA	Cast Concrete Arch
CEA	Central Electricity Authority
CF	Continuity Factor
Ch.	Chainage
CSMRS	Central Soil and MaterialsResearch Station
CWC	Central Water Commission
DRESS	Drainage-Reinforcement-Excavation-Support-Solution
ESR	Excavation Support Ratio
FAL	Fixed Anchor Length
FRL	Full Reservoir Level
GRC	Ground Response Curve
GSI	Geological Strength Index
GSI	Geological Survey of India
HHC	Higher Himalayan Crystalline
HRT	Head Race Tunnel
ISC	Immediate Shotcrete
ISRM	International Society of Rock Machanics
JP	Jointing Parameter
MCT	Main Central Thrust
MDDL	Minimum Draw Down Level
N	Rock Mass Number
NATM	New Austrian Tunneling Method
NGI	Norwegian Geotechnical Institute
NJP	Nathpa Jhakri Project
NMT	Norwegian Method of Tunneling
Q	Rock Mass Quality
RCR	Rock Condition Rating
RMC	Rock Mass Classification
RMi	Rock Mass index

RMR	Rock Mass Rating
RQD	Rock Quality Designation
RR	Rib Ratio
RRS	Reinforced Rib of Shotcrete
RSR	Rock Structure Rating
SDSC	Specially Designed Shotcrete
SFRS	Steel Fibres Reinforced Shotcrete
SRC	Support Reaction Curve
SRF	Stress Reduction Factor
SZ	Shear Zone
TBM	Tunnel Boring Machine
UCS	Uniaxial Compressive Strength

INTRODUCTION

India's large number of river systems, offer tremendous scope for hydropower development. As per studies carried out by Central Electricity Authority (CEA) during 1978-87, the economically exploitable hydropower potential from major/medium schemes in the country has been assessed as 84044 MW at 60 percent load factor. Out of this potential, only 16.7 percent have been developed so far and another 6.3 percent are under various stages of development as on 01. 09. 2001 (CEA, 2001).

Himalayan river systems, which benefit from both melting snow and southwest monsoon, lend themselves admirably to storage dams and run-of-the river development. Several major hydroelectric projects like Nathpa Jhakri (1500 MW), Tehri (2400 MW), Uri (600 MW), Dulhasti (390 MW), Baspa (300 MW) and Doyang (800 MW) are under construction with additional projects contemplated over the next 50 or more years. Figure 1.1 shows major hydroelectric projects in Himalaya. Majority of the hydroelectric potential lies in Himalaya. In northern region with a potential of 30155 MW, only 23.56 percent have been developed and under development so far. Similarly, Arunachal Pradesh in northeastern region, is having enormous hydropower potential of 26756 MW, and of this only 0.47 percent has been developed and under development so far (CEA, 2001).

During last forty years several kilometers of tunnels have been constructed in Himalaya in connection with hydropower projects. Most of these projects have faced problems related to excessive support pressures, occurrences of clay pockets, seepage water and hazards associated

Fig. 1.1 Major hydroelectric projects in Himalaya

with Himalayan tectonics like mountain-stresses, abnormal rock loads and geothermal gradients etc. At Pandoh-Baggi, Giri and Loktak tunnels, twisting and buckling of ribs was recorded (Singh and Mahajan, 1982). Problem of excessive temperature was reported at Sanjay Vidyut Pariyojna (Saini et al., 1989). Sudden on-rush of water has been found in Sundernagar-Slapper, Baira-Siul, Chhibro-Khodri and Maneri-Bhali tunnels (Dhar and Choubey, 1986).

Nathpa Jhakri Project (NJP), located in the state of Himachal Pradesh in India, is a run-of-river scheme on river Satluj. The head race tunnel (HRT) of this project is 27.4 km long with a diameter of 10.15 m. It is the longest power tunnel in the world being constructed in Higher Himalaya. Excavation of this long tunnel has imposed many challenges to the field engineers. These include problems related to geothermic, heavy inflow of groundwater, excessive rock covers, flowing, slabbing and squeezing ground conditions. Rock types in the area encompassed by the project comprise a variety of metamorphic rocks like gneisses, schistose gneiss, schist, quartzite and basic intrusive (amphibolites), granite and pegmatite belonging to Jeori Wangtu Gneissic Complex of Precambrian age. Rocks are folded and faulted, highly jointed, encountering shear zones and shear seams of varying thicknesses. Excavation of this tunnel, passing through such geology, has generated enormous amount of geotechnical data. In view of this, it was considered appropriate to take it as a case for rock mass characterization and evaluation of supports for tunnels in Himalaya.

1.1 POWER POTENTIAL OF SATLUJ BASIN

Himachal Pradesh has huge hydropower potential in its five river basins, which has been estimated as 11647 MW at 60 percent load factor, and of this only 22.7 percent have been developed and under development so far (CEA, 2001). Nathpa Jhakri Project, with an installed capacity of 1500 MW, is the largest power development contemplated on the Satluj basin in respect of hydropower development. This basin has an identified power potential of 9227 MW, which is about 45 percent of the total power potential of Himachal Pradesh. Out of this total potential, only 1332 MW has so far been exploited through major and medium projects like Bhakra (1200 MW), Sanjay Vidyut Pariyojna (120 MW) in addition to some small hydroelectric projects.

1.2 NATHPA JHAKRI PROJECT

Nathpa Jhakri Project utilizes a drop of 488 m between Nathpa, the dam site and Jhakri, the powerhouse site. The distance by road from Jhakri to Nathpa is about 45 km. The powerhouse is under construction and is located at Longitude 77^0 45′ and Latitude 30^0 45′ near NH-22, connecting Shimla with China border. The location of Nathpa Jhakri Project is indicated at number 16 in Fig. 1.1.

The project area is characterized by very rugged topography with lofty hills. The general elevation of the area is above 1000 m. The area is dissected by deep and narrow valleys and gorges, having steep cliffs and escarpment faces. It is drained by river Satluj and its tributaries. River Satluj, which is the principal tributary of river Indus, originates near lake Mansarovar at an altitude of 4570 m. The average bed gradient of the river between Nathpa and Jhakri is 1 in 60.

The HRT on the upstream is connected to the outlet tunnels taking off from desilting chambers, and on the downstream side it terminates at surge shaft. It conveys a design discharge of 405 cumec with a flow velocity of 5 m/sec. Rock cover along the tunnel, laid largely at a slope of about 1 in 275, 1 in 61 and 1 in 430, varies from 90 m to 1430 m except under Manglad creek where it is only 9 m. The internal pressure from the junction point of outlet tunnels from desilting chambers to the surge shaft varies between 400 kN/m^2 and 2200 kN/m^2 under static conditions and between 480 kN/m^2 and 3100 kN/m^2 under maximum upsurge conditions. The HRT has been planned to be concrete lined except for the two reaches where it crosses low rock cover zones under Manglad and Daj creeks. Steel liner of 8.5 m diameter with plate thickness of 30 to 40 mm shall be provided for a length of 710 m at Manglad creek and 375 m at Daj creek in these low rock cover reaches. Layout plan and longitudinal section of Nathpa Jhakri Project are shown in Figs. 1.2 and 1.3 respectively. Salient Features of the project are given in APPENDIX – I.

1.3 BRIEF REVIEW OF EARLIER WORK

Engineering rock mass classification (RMC) is the best-known empirical approach for the design of underground openings in rock. This approach has got enormous potential and forms the backbone of present day rock engineering. As a matter of fact, almost all the modern underground constructions are utilizing RMC approach due to its simplicity.

A number of RMC systems have been in use for the past five decades. Out of all these systems, Rock Structure Rating (RSR), Rock Mass Rating (RMR) and Rock Mass Quality (Q) systems present a quantitative method for describing the quality of a rock mass for selecting the appropriate ground support. Rock Mass index (RMi) system, recently proposed by Palmstrom (1995a) is also a quantitative method.

Terzaghi (1946) formulated the first rational method of classification by evaluating rock loads appropriate to the design of steel sets. Deere et al. (1970) modified Terzaghi's classification by introducing the Rock Quality Designation (RQD) as the lone measure of rock quality. Jethwa (1981) and Jethwa et al. (1982a) showed that Terzaghi (1946) and Deere et al. (1970) underestimate rock pressure for small tunnel openings in moderate squeezing conditions and overestimate them for large tunnel openings under high squeezing conditions. Singh et al. (1995) found that the support pressure in rock tunnels and caverns did not increase directly with excavation size as assumed by Terzaghi (1946).

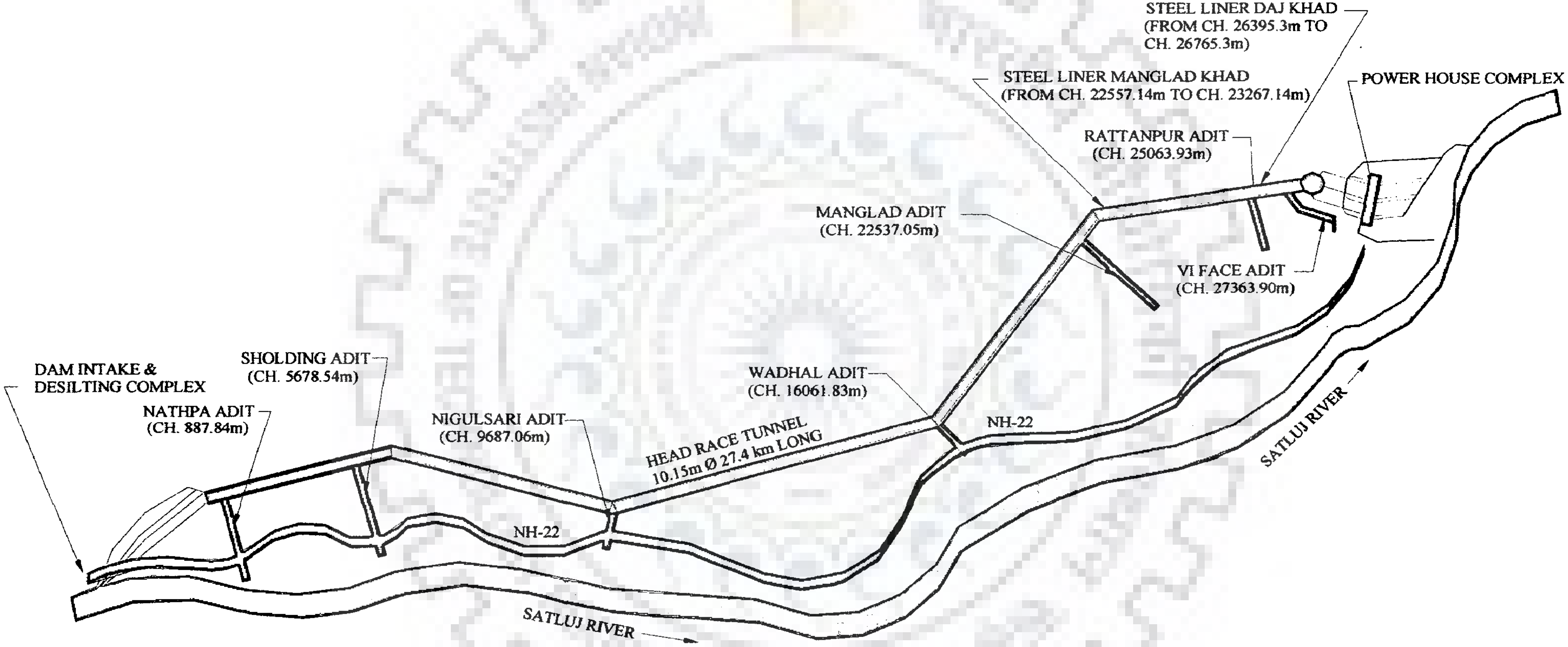


Fig. 1.2 Layout plan of Nathpa Jhakri Project

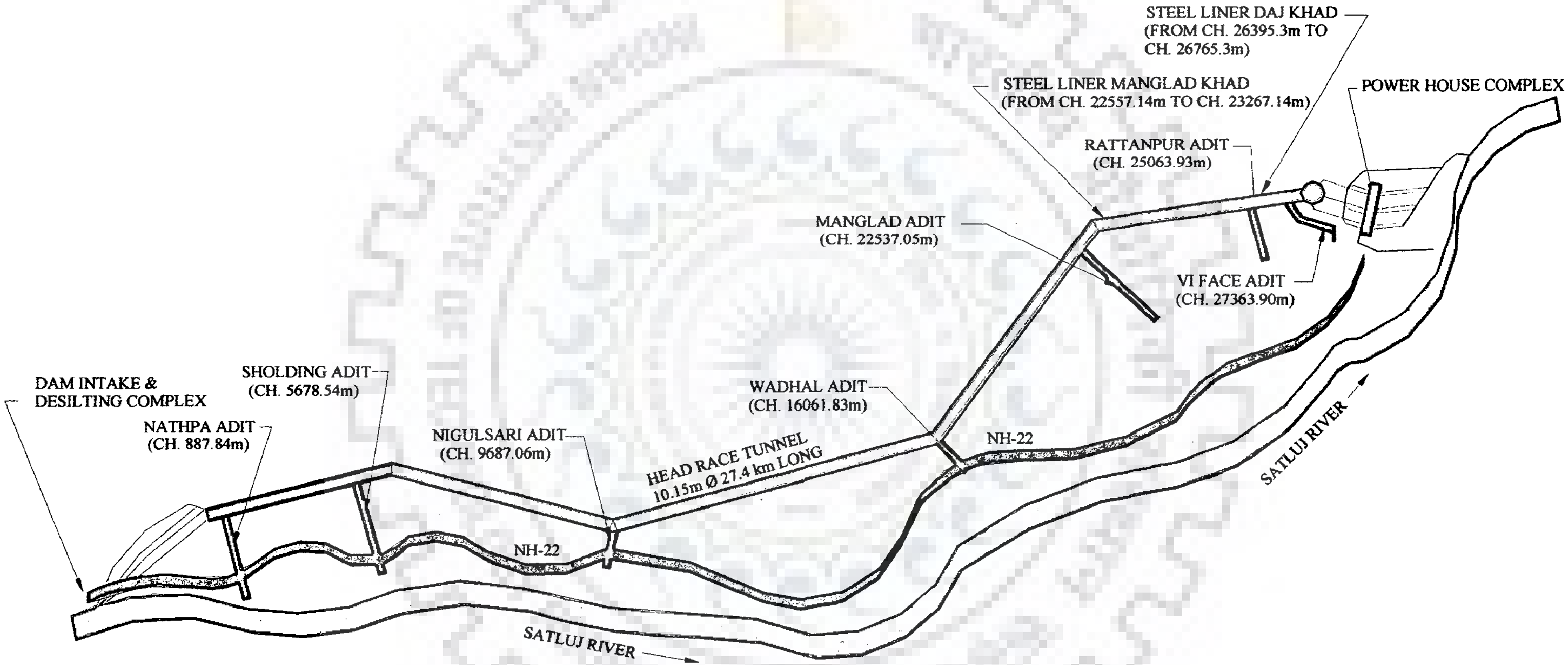


Fig. 1.2 Layout plan of Nathpa Jhakri Project

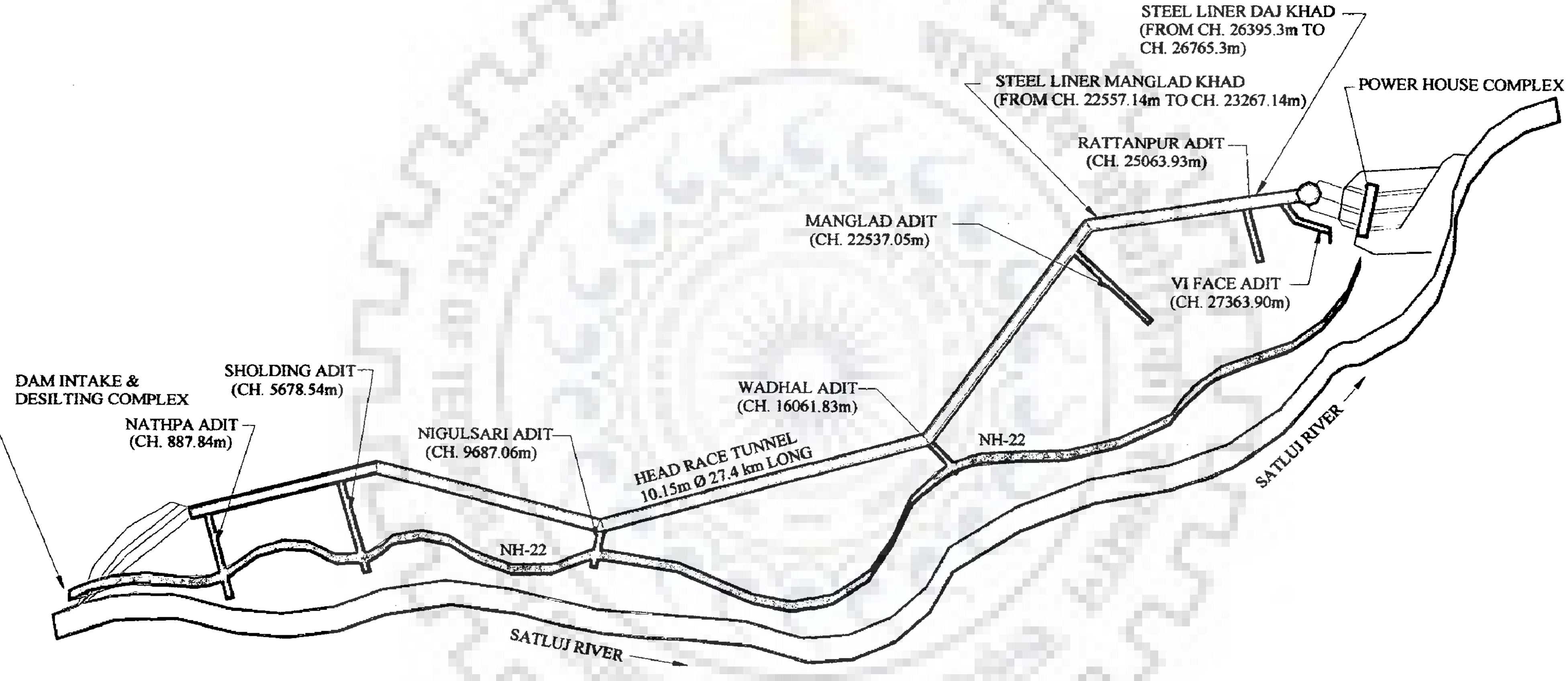


Fig. 1.2 Layout plan of Nathpa Jhakri Project

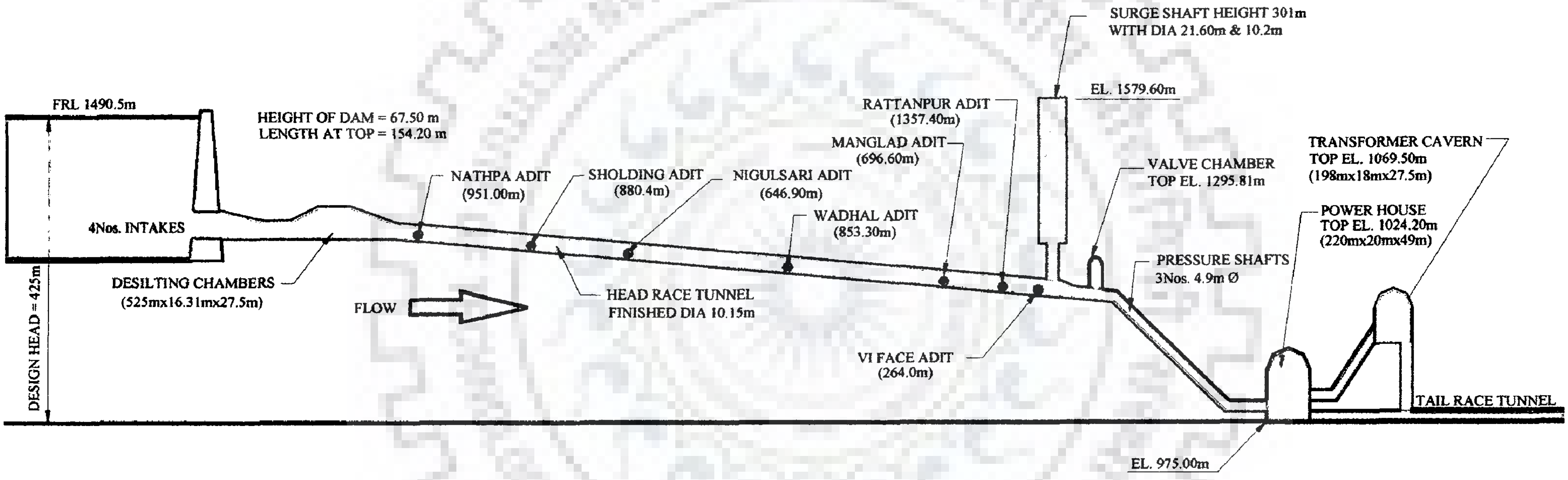


Fig. 1.3 Longitudinal section of Nathpa Jhakri Project

Wickham, Tiedemann and Skinner (1972) presented RSR-system. It was the first complete RMC system proposed since the one introduced by Terzaghi in 1946. The main contribution of the RSR concept is that it introduced a rating system for rock masses. Krishna Murthy (1986) found that RSR-system is not suitable for soft rocks with squeezing ground conditions.

Terzaghi (1946) had proposed rock load factor depending upon rock conditions, which has been only qualitatively described such as intact, stratified etc. In practice there are no sharp boundaries between these rock categories, and properties of the rocks indicated by each one of these terms can vary between wide limits. What Wickham et al. (1972) have really done is that they have described a particular rock condition in quantitative term in the form of RSR-value based on geologic and construction parameters. This rock condition or obtained RSR-value is then compared with the datum condition and rib supports varied accordingly. Rib ratio (RR) is used to facilitate the comparison, if it is 100 then rock condition existing would be similar to the datum condition i.e. loose, saturated, cohesionless sand (above water table), and if it is less than 100 better the rock. Therefore, it is better to estimate an RSR-value rather than describing rock qualitatively.

Bieniawski (1973) developed RMR-system, which provides guidelines for the selection of roof support for various rock mass classes. Dube et al. (1982, ref. 57) found that values of modulus of deformation of rock mass (E_d) estimated from RMR at Tehri Dam Project, agreed fairly well with those measured. Yudhbir (1986) demonstrated that the rock masses could be successfully classified using RMR methods in respect of their strength and deformability characteristics. Speers (1992) concluded that the RMR method should not be used for support design because it does not take sufficient account of the rock loads. However, the method should be used to provide initial estimates of the rock mass strength parameters c and ϕ. Goel et al. (1999) found that the correlations for pressure dependent E_d, given by Verman (1993) are quite reliable for nearly dry, weak and highly jointed rock masses.

Barton, Lien and Lunde (1974) proposed Q-system. The Q-system, according to Jethwa (1981) and Jethwa et al. (1982a), does not work satisfactorily under squeezing rock conditions. However, Jethwa et al. (1982b) concluded that the Q-system works well in non-squeezing ground conditions. Kaiser et al. (1986) opined that stress reduction factor (SRF) is probably the most contentious parameter. Goel (1994) and Singh et al. (1997, ref. 149) opined that estimating a correct value of SRF is difficult and incorrect selection of SRF-values may lead to an unreliable prediction. Goel et al. (1995a) proposed Rock Mass Number (N) and Rock Condition Rating (RCR). These indices are the modified versions of the two most popular classification systems, N from the Q-system of Barton et al. (1974) and RCR from the RMR-system of Bieniawski (1973).

Choubey (1986) considers that in squeezing and swelling ground, the ratio of rock stress to rock strength provides a useful guideline to rock behaviour. Grimstad and Barton (1993) have modified SRF-values given earlier by Barton et al. (1974) for hard, massive rock masses. According to Choubey et al. (1995), very high values of SRF may be applicable in special cases having high ratios of RQD/J_n. Grimstad and Barton (1993) reported that in areas affected by high stresses, no real stress problems were observed in areas characterized by heavily jointed or crushed rock.

Sheorey (1985) concluded that equation for estimation of support pressure, proposed by Barton et al. (1974), does not agree with practical observations in squeezing ground conditions since it does not include depth of cover as well as the broken zone size. The SRF-values recommended do not consider this fact effectively.

The correlation between rock mass quality and tunnel support pressure proposed by Barton et al. (1974) has proved generally rewarding except in cases of squeezing ground conditions (Singh, 1988). Singh et al. (1992) suggested modifications by correlating the support pressure with the rock mass quality, the overburden, the tunnel closure and the time after excavation. Goel et al. (1995a) also found that the approach of Barton et al. (1974) is unsafe in squeezing ground conditions and the reliability depends upon the rating of Barton's SRF.

Singh et al. (1995) divided the squeezing ground condition into three classes, on the basis of tunnel closures as mild, moderate and high squeezing ground conditions. Goel et al. (1995b) and Goel (2001) also suggested an empirical approach for the same using N.

Singh et al. (1995a, b) proposed a semi - empirical method for the design of support systems consisting of shotcrete/fibre reinforced shotcrete, rock bolts, steel ribs and grouted arches to support tunnels and caverns. They found that the method was suitable for low pressures only.

Palmstrom (1995a) proposed RMi to characterize rock mass strength as a construction material. Palmstrom (1995b) claimed that the application of RMi in rock support involves a more systematized collection and application of the input data. Palmstrom (2000a) further claimed that the use of the three-dimensional block volume would generally improve the characterization of the rock mass compared to the use of RQD and joint spacing.

Palmstrom et al. (2000) opined that the forces (water and overburden) acting on the rock mass should not form part of any initial rock mass characterization. According to them, RMR and Q are actually classification systems for the design of ground support rather than characterization systems.

Barton (2001) concluded that truncated characterization in RMi rejects the influence of water and stress in the estimation of deformation modulii and seismic velocities, which is illogical. Barton (1999, 2002a, b) emphasized that deformation modulii and seismic velocities do not correlate with jointing parameter alone, but with the complex non-linear response of these properties to the "internal" and "external" boundary conditions, which often give anisotropic as well as depth-dependent properties.

Singh and Palmstrom (2002) pose the question whether rock engineering shall be made according to a process where rock classification is based merely on a 'all-in-one' number representing the ground, or if it shall be a stepwise process where the process is divided into certain levels for the parameters to be evaluated, studied and/or compared.

Lombardi (1970), Daemen and Fairhurst (1970), Ladanyi (1974) and Lombardi (1977) considered rock mass – support interaction as one of the most promising methods for understanding the mechanics of tunnel deformation and development of rock loads. According to Daemen (1975), ground response curve is quite useful for designing the supports specially for tunnels through squeezing ground conditions. Goel et al. (1998) have developed an empirical approach for obtaining the ground response curve using N and RCR in squeezing ground conditions. Fairhurst (1993) considers that it is sufficient to observe the radial convergence of a tunnel as a function of time in order to assess the stability of the tunnel.

Singh et al. (1998) have suggested a new failure theory for jointed rock masses. According to them there is significant strength enhancement due to in-situ stress along tunnels which pre-stresses rock wedges and prevents their failure both in the roof and in the walls.

Franklin (1993) considers the development of computer-based 'expert systems' called 'Classex' (Butler and Franklin, 1990) to compute Q, RMR, RQD and RSR, an encouraging new direction in this field.

1.4 IDENTIFICATION OF PROBLEM AND OBJECTIVES OF STUDY

A critical review of the literature suggests that:

(a) RMC systems such as RSR, RMR and Q are suitable in their present form mainly for solid rocks; their suitability in jointed rocks of Himalaya may be limited, specially in regions of high geothermal gradients. RMi-system has only recently been developed in Norway, applicability of which is not yet established in many types of rock masses.

(b) There is difficulty in application of the RSR-system due to some ambiguous parameters, such as rock hardness and geological structure involved in it. Furthermore, its recommendations for steel ribs for datum condition do not conform to steel rib sections manufactured as per the Indian Standards.

(c) The RMR-system recommends only ranges for thickness of shotcrete, length and spacing of rock bolts and spacing of ribs for various rock mass classes.

(d) The semi - empirical method for the design of supports for tunnels is applicable for low pressures only.

(e) The Q-system does not suggest SRF-values for 'competent rock, rock stress problems' category for moderately jointed rocks in categories L, M and N.

(f) Mohr's theory is not applicable for anisotropic and jointed rock masses on account of significant strength enhancement due to in-situ stress along tunnels. Singh et al. (1998) have suggested a new failure theory for jointed rock masses, which needs to be verified in field.

The objectives of the proposed study therefore are:

(a) Characterization of rock mass using RSR, RMR, Q and RMi systems.

(b) Developing correlations among above classification systems including N and RCR.

(c) Estimation/comparison of support pressures and developing new correlations for the same.

(d) Design/comparison of supports recommended by classification systems and evaluation of supports installed at site.

(e) Verifying applicability of recently developed RMi-system for Himalaya.

(f) Performance evaluation through rock mass – support interaction analysis.

(g) Proposal for SRF-values and developing correlation for moderately jointed rocks.

(h) Comparison of Mohr's and Singh's criteria of strength of rock mass.

(i) Developing correlations for prediction of ground condition.

Apart from the above objectives, interactive computer programs have been developed in C++ language for:

- RMC using RSR, RMR, Q and RMi systems.
- Determination of ground response curve (GRC) as per the formulations of Ladanyi (1974) and Brown et al. (1983).
- Determination of support reaction curve (SRC) for shotcrete/rock bolt and steel rib/concrete supports.

1.5 ORGANISATION OF THESIS

The thesis has been organized in eight main chapters along with an introduction and a concluding chapter. Introduction of the work has been given in the present **CHAPTER – 1**.

The review of literature pertaining to RMC systems and rock mass – support interaction analysis is presented in **CHAPTER – 2**.

CHAPTER – 3 covers geological setting in three parts viz. regional, in and around project area and along the tunnel alignment.

The tunnelling problems faced during excavation of the HRT have been discussed in **CHAPTER – 4**.

CHAPTER – 5 is devoted to RMC. It also includes development of computer program **ROMAC** for RMC using RSR, RMR, Q and RMi systems. Correlations between various systems have also been presented.

CHAPTER – 6 deals with design of support systems and estimation of support pressures following recommendations given by various RMC systems. A comparison of support pressures has also been presented.

CHAPTER – 7 presents comparison of supports with modified semi - empirical method after incorporating modification in semi - empirical method of design of supports.

Rock mass – support interaction analysis is a well-known method of design of tunnel support system. An attempt has been made, using this method, to evaluate supports recommended by RMC systems and also those installed by the Project Authorities in **CHAPTER – 8**.

CHAPTER – 9 evaluates Q and RMi systems.

Finally, the work has been concluded in **CHAPTER – 10**.

REVIEW OF LITERATURE

The complexities and uncertainties inherent in the tunnel design compel the engineers to adopt empirical methods for selection and design of tunnel supports. Engineering RMC is the best-known empirical approach for assessing the stability of underground openings in rock. Such classification methods enable the designer to relate the experience on rock conditions and support requirements gained on other sites to the conditions anticipated on his own site. A number of RMC systems have been in use for the past five decades. Some of the commonly used RMC systems are briefly reviewed as follows.

2.1 ROCK MASS CLASSIFICATION SYSTEMS

2.1.1 Terzaghi's Theory

Terzaghi (1946) formulated the first rational method of classification by evaluating rock loads appropriate to the design of steel sets. The details of the system are given in APPENDIX – IIA. Terzaghi suggested the following equation for obtaining the support pressure:

$$p_v = \gamma . H_p \tag{2.1}$$

Cecil (1970) concluded that the classification was too general to permit an objective evaluation of rock quality and that it provided no quantitative information regarding the rock mass properties. Deere et al. (1970) modified Terzaghi's classification by introducing RQD as the lone measure of rock quality (APPENDIX – IIB). It may be seen that Terzaghi's rock load pressure increases with tunnel size. On the basis of more than 25 case records of large cavities, Cording and Deere (1972) concluded that increase in support pressure with increasing dimensions of cavity was not necessary. It would therefore be better if rock load factors were replaced by constant values of support pressures.

Jethwa (1981) and Jethwa et al. (1982a) compared the predicted and observed rock mass behaviour, and showed that the classification systems of Terzaghi (1946) and Deere et al. (1970) underestimate rock pressure for small tunnel openings in moderate squeezing conditions and overestimate them for large tunnel openings under high squeezing conditions.

Krishna Murthy (1986) observed that Terzaghi's (1946) method is not useful in present day tunnelling practices. Tunnels constructed at Chenani Hydroelectric Project have demonstrated that the expected rock loads did not develop, as the sections of tunnel stood without support for a long time even in the reaches with clayshales and claystone.

Terzaghi's approach was successful when conventional drill and blast method of excavation and steel-arch supports were employed in the tunnels of comparable size. This practice lowered the strength of the rock mass and permitted significant roof convergence, which mobilized a zone of loosened rock mass from the tunnel roof. With the advent of New Austrian Tunneling Method (NATM) and Norwegian Method of Tunneling (NMT), controlled blasting and machine excavation techniques as well as support system employing reinforced shotcrete and rock bolts are being increasingly used. These improvements in the tunnelling technology preserve the pre-excavation strength of the rock mass and use it as a load carrying structure in order to minimize roof convergence and restrict the height of the loosening zone above the tunnel crown (Singh and Goel, 1999).

2.1.2 Modified Terzaghi's Theory

Singh et al. (1995) compared support pressures measured from tunnels and caverns with estimates from Terzaghi's rock load theory and found that the support pressure in rock tunnels and caverns did not increase directly with excavation size as assumed by Terzaghi (1946) and others mainly due to dilatant behaviour of rock masses, joint roughness and prevention of loosening of rock mass by improved tunnelling technology. They subsequently recommended ranges of support pressures as given in APPENDIX - IIC.

2.1.3 Stand – Up Time Classification

Lauffer (1958) introduced the concept of an unsupported span and its equivalent stand-up time on the basis of the earlier work on tunnel geology by Stini (1950). He illustrated the influence of orientation of tunnel axis, shape of opening, method of excavation and support system on the unsupported span. Lauffer's original classification was modified a number of times by other Austrian Engineers, notably by Pacher et al. (1974), leading to the development of NATM. The main significance of the Lauffer-Pacher classification is that an increase in tunnel span leads to a major reduction in the stand-up time.

Rabcewicz (1964, 1965) introduced NATM. In NATM, shotcrete is not intended to be used like a conventional concrete lining on a fairly good rock far behind the face, where the rock is already decompressed and sufficiently safeguarded by rock bolting. On the contrary, the method aims to reduce anchoring to a minimum by preventing initial superficial loosening as a result of more effective interaction between shotcrete and rock. If shotcreting is not introduced as an integral part of the driving process its advantages will not be fully utilized.

Muller (1978) clarified that NATM is not so much a way of excavating and supporting, but rather a concept. Due of the foremost principles in it is to utilize the surrounding rock mass to become the main load-bearing component, with the lining establishing a load-bearing ring.

Brown (1981) emphasized that there is difference in near-surface and deep tunnelling in soft ground by NATM. The need to time the placement of the support so that sufficient deformation of the ground occurs and the support elements do not become overstressed is not the feature of NATM approach in near-surface tunnelling. In this case, in-situ stresses will be relatively small, the ground relatively weak and so unable to support redistributed loads, and the influence of gravity on the behaviour of the material in the roof relatively greater. Furthermore, deformations around opening are reflected on the surface, which is not the case with deep tunnelling. Therefore, it will be necessary to close the invert quickly to form a load-bearing ring and to leave no section of the excavated tunnel surface unsupported, even temporarily.

According to Ward (1978), there is nothing like the rock load on a tunnel support in rock and it all depends upon when and where the support was installed.

2.1.4 Rock Quality Designation (RQD) Index

Deere et al. (1967) proposed a quantitative index known as RQD based on modified core recovery procedure. A relationship between RQD and engineering quality of the rock has been proposed by Deere (1968).

Cording and Deere (1972) attempted to relate RQD index to Terzaghi's rock load factors and related RQD index with tunnel support. They found that Terzaghi's rock load concept should be limited to tunnels supported by steel sets, as it does not apply well to openings supported by rock bolts.

Merritt (1972) pointed out that RQD is a practical parameter for core quality estimation but it is not sufficient in itself to provide an adequate description of a rock mass.

Palmstrom (1982) suggested a correlation between RQD and the number of joints per unit volume, in which the number of joints per metre for each joint set is added.

2.1.5 Rock Structure Rating (RSR) - System

The RSR concept developed by Wickham, Tiedemann and Skinner (1972) presents a quantitative method for describing the quality of a rock mass for selecting the appropriate ground support. It was the first complete RMC system proposed since the one introduced by Terzaghi in 1946. Wickham et al. (1974) subsequently updated the RSR concept. The main contribution of the RSR concept is that it introduced a rating system for rock masses.

The RSR concept considers geological and construction parameters such as A, B and C (APPENDIX – III). The RSR-value is obtained by summing the weighted numerical values determined for each parameter. The RSR-values were correlated with actual support installations.

Rutledge and Preston (1978) concluded that in the design of steel sets for tunnels up to 8 m in width, the use of 75 percent of the support predicted by Wickham's method is justified provided possible problems of durability, swelling or squeezing ground, or in-situ stresses are recognized.

Krishna Murthy (1986), on the basis of experiences in Giri-Bata, Loktak and Yamuna tunnels, found that RSR-system is not suitable for soft rocks with squeezing ground conditions.

Sinha (1988) pointed out that it is not possible to prescribe the steel ribs or rock bolts without using the Terzaghi's system. Thus, the RSR concept may be viewed as an improvement of Terzaghi's method rather than an independent system.

2.1.6 Geomechanics Classification or Rock Mass Rating (RMR) - System

Bieniawski (1973) developed the Geomechanics Classification or the Rock Mass Rating (RMR) system on the basis of his experiences in shallow tunnels in sedimentary rocks. Subsequently, the classification has undergone several significant changes: in 1974–reduction of classification parameters from 8 to 6; in 1975–adjustment of ratings and reduction of recommended support requirements; in 1976–modification of class boundaries to even multiples of 20; in 1979–adoption of ISRM (1978) rock mass description etc.

Summation of the ratings for the five parameters listed in APPENDIX - IVA yield the basic RMR. Thereafter, the sixth parameter (the influence of strike and dip orientation of discontinuities) is included according to APPENDIX – IVB to give RMR-value. The rock mass is classified according to APPENDIX - IVC. APPENDIX – IVD gives the practical meaning of each rock mass class. To decide whether strike and dip orientations are favorable or not in tunnelling, APPENDIX - IVE may be referred. Guidelines for the selection of roof support are given in APPENDIX - IVF.

Bieniawski (1975) considered, the Geomechanics Classification as a very powerful tool for assessing rock mass conditions, selecting support or rock reinforcement measures and enabling effective communication on engineering projects.

Lauffer (1988) observed that the stand-up time improves by one class of RMR value in case of excavations by Tunnel Boring Machine (TBM). Singh and Goel (1999) cautioned that one should not unnecessarily delay supporting the roof in the case of a rock mass with high stand-up time as this may lead to deterioration in the rock mass, which ultimately reduces the stand-up time.

Yudhbir (1986), on the basis of detailed field studies conducted at the Khao Laem dam site in Thailand, demonstrated that the rock masses could be successfully classified using RMR methods in respect of their strength and deformability characteristics. Speers (1992) concluded that the RMR method should not be used for support design because it does not take sufficient account of the rock loads. However, the method should be used to provide initial estimates of the rock mass strength parameters c and ϕ.

Several researchers have suggested correlations for estimation of E_d. Prominent among them are Bieniawski (1978), Serafim and Pereira (1983), Nicholson and Bieniawski (1990), Mehrotra (1993), Mehrotra et al. (1998), Verman (1993), Verman et al. (1993, 1997), Mitri et al. (1994) and Hoek and Brown (1997).

Dube et al. (1982, ref. 57) conducted study at Tehri Dam Project, which has a huge cavern (250 m long, 50 m high and 22.5 m wide to house the power plant for generating 2400 MW). They compared the values of E_d estimated from RMR and diametrical tunnel closure observations with those measured by flat jacks and concluded that results agreed fairly well to give immense confidence in the classification approach.

Mehrotra (1993), Mehrotra et al. (1998) found that under saturation the reduction in E_d is as high as 90 percent for poor rocks and 70 percent for fair quality rocks. According to Verman (1993) and Verman et al. (1993, 1997), E_d of a dry and weak rock mass (q_c < 100 MPa) around underground openings located at the depth exceeding 50 m is dependent upon confining pressure due to overburden. Goel et al. (1999) found that the correlations for pressure dependent E_d, given by Verman (1993) are quite reliable for nearly dry, weak and highly jointed rock masses.

Hoek and Brown (1980b), and Hoek (1983) proposed a method for estimating rock mass strength for:

<u>Original rock mass:</u>

$$\sigma_1 = \sigma_3 + (m q_c \sigma_3 + s.q_c^2)^{0.5} \tag{2.2}$$

<u>Broken rock mass:</u>

$$\sigma_1 = \sigma_3 + (m_r q_c \sigma_3 + s_r.q_c^2)^{0.5} \tag{2.3}$$

Constants of rock mass can be estimated as per the correlations proposed by Hoek and Brown (1988) using RMR.

According to Ramamurthy et al. (1986), ambient pressure must also figure in the classification system to suggest whether a ground is likely to behave as an elastic, plastic or squeezing type.

Bieniawski (1993) identified extensions of the RMC systems for applications to underground nuclear repositories, as a prominent aspect for future development involving RMCs.

According to Verman et al. (1997), mobilization of a much higher cohesion takes place around underground openings than the values suggested by Bieniawski (1979). This indicates a strength enhancement around underground openings due to increased restraint in the freedom of propagation of fractures.

2.1.7 Rock Mass Quality (Q) - System

Barton, Lien and Lunde (1974) proposed the Q-system of RMC based on a numerical assessment of the rock mass quality using six different parameters (APPENDIX – V). The parameters are grouped into three quotients to give the overall rock mass quality Q.

Barton et al. (1974) suggested 38 support categories which give estimates of permanent support in the form of rock bolts, shotcrete, wire mesh and cast concrete arch (CCA). Grimstad and Barton (1993) suggested a different chart for design of support using steel fibre reinforced shotcrete (SFRS).

Barton et al. (1974) considered that the parameters J_n, J_r and J_a play more important role than the joint orientation, because the number of joint sets determines the degree of freedom for block movement (if any), and the frictional and dilatational characteristics (J_r) can vary more than the down-dip gravitational component of unfavourably oriented joints.

Jethwa (1981) and Jethwa et al. (1982a) concluded that the predicted short-term rock pressures by the Q-system are reasonably close to the measured values only for small tunnels under moderate squeezing conditions. It underestimated the short-term rock pressures for large tunnels under high squeezing conditions. Jethwa et al. (1982b) concluded that the predicted short-term rock pressures by the Q-system are almost equal to the measured values both for small and large tunnels in non-squeezing ground conditions.

According to Dhawan (1986), most of the classification systems do not take into account specific features of the work, except the Q-system, which takes into account the type and purpose in terms of the excavation support ratio (ESR). In all the classification systems, squeezing ground is the class of problems that is inadequately represented.

Sheorey (1985) compared ultimate roof support pressure estimates using the Q-system with the observed values in seven cases of squeezing ground in Himalayan region, where the depth of cover ranged between 240 m and 680 m. He concluded that equation proposed by Barton et al. (1974) does not agree with practical observations since it does not include depth of cover as well as the broken zone size. The SRF-values recommended do not consider this fact effectively. He suggested modified values of SRF, which consider rock mass rating RMR and the cover pressure to account for the effects of depth and broken zone size on support pressure.

According to Choubey (1986), the work in Japan (Tanimoto, 1980) on extremely and exceptionally poor ground conditions provided sufficient data for improving support recommendations in estimating SRF in the categories of squeezing and swelling ground. According to him, the ratio of rock stress to rock strength provides a useful guideline to rock behavior to both for the case, of highly stressed competent rocks and for the incompetent rocks, that frequently exhibits squeezing phenomena as opposed to rock bursting.

Grimstad and Barton (1993) have modified SRF-values given earlier by Barton et al. (1974) for hard, massive rock masses. According to them, hard massive rocks under high stress require more support than recommended for the corresponding values. The tunnels under high stresses in hard rock now include less bolting, but extensive use of SFRS: an unknown product when the Q-system was first published in 1974. The updating of the Q-system has shown that in the most extreme cases of high stress and hard massive rock, the maximum SRF-value has to be increased from 20 to 400 in order to give a Q-value, which correlates with the modern rock support.

Grimstad and Barton (1993) further stated that the rating of SRF in rock affected by high stress is difficult to estimate by visual observations. If possible SRF should be estimated by the ratio between the compressive strength (q_c) and the major principal stress (σ_1) or by the ratio between the tangential stress (σ_θ) and q_c as shown in APPENDIX - V, section 6b. Where the stress level is not measured at tunnelling projects, SRF has to be classified by means of the observed behavior of the rock and by good engineering judgment. According to Grimstad and Barton (1993), in areas affected by high stresses, no real stress problems were observed in areas characterized by heavily jointed or crushed rock.

As in the case of massive hard rocks, very high values of SRF may be applicable in the special cases of soft rocks, which have high ratios of RQD/J_n. This approach appears promising for establishing relationship between SRF and the ratio σ_θ/q_c (Grimstad and Barton, 1993).

Grimstad and Barton (1993) stated further that since crushed, squeeze prone rock masses will tend to have low Q-value (first five parameters) even before high values of SRF are applied, it is unlikely that the 'new SRF' values suggested for slabbing and bursting in hard massive rocks will apply. It is probable that for squeeze-prone rock masses, SRF-values in the range 5 - 10 will apply when σ_θ/q_c ratios are in the range 1 to 5, while SRF-values as high as 10 to 20 will be applicable when the σ_θ/q_c ratio exceeds 5.

According to Choubey et al. (1995) and Choubey (1998), very high values of SRF may be applicable in special cases having high ratios of RQD/J_n and this relationship is promising for establishing relationship between SRF and the ratio of σ_θ/q_c.

Grimstad and Barton (1995) suggested that in high-seismicity areas, SRF-values might be doubled in order to cope with the dynamic forces initiated by an earthquake.

Singh et al. (1995a, b) proposed a semi - empirical method for the design of support systems consisting of shotcrete/SFRS, rock bolts, steel ribs and grouted arches to support tunnels and caverns. They suggested empirical relations for mobilizing factors of shotcrete and bolts from the extensive data of the Norwegian Geotechnical Institute (NGI) tables and charts for support systems presented by Barton et al. (1974).

Grimstad and Barton (1993) developed a method for assessing support requirements for rock masses affected by shear zones. The weak zones and the surrounding rock are allocated their respective Q-values from which a mean Q-value can be determined using following formula:

$$\log Q_m = \frac{T_z . \log Q_{wz} + \log Q_{sr}}{T_z + 1} \tag{2.4}$$

According to Singh and Goel (1999), Eq. 2.4 may also be used for estimating the weighted average of joint roughness number (J_{rm}), after replacing logQ by J_r appropriately. Similarly, the weighted average of joint alteration number (J_{am}) may also be found out.

Further, multiplying Eq.2.4 by 25 in the numerator and replacing 25 logQ by E, average value of modulus of deformation (E_m) may be obtained as follows (Singh and Goel, 1999):

$$E_m = \frac{T_z . E_{wz} + E_{sr}}{T_z + 1} \tag{2.5}$$

Samadhiya (1998) demonstrated, through three-dimensional finite element analysis of underground powerhouse of Sardar Sarovar Hydroelectric Project, that the maximum deflections of wall are increased near the shear zone (T_z = 2 m) by a factor of E_{sr}/E_m. Further, the predicted support pressure on shotcrete near the shear zone are increased to about $2Q_m^{-1/3}/J_{rm}$ whereas the support pressure in the surrounding rock away from shear zone are approximately $2Q_{sr}^{-1/3}/J_{rsr}$.

According to Barton et al. (1980), E_d varies from 10 $\log_{10}Q$ to 40 $\log_{10}Q$. Choubey (1986), on the basis of an analysis of a large number of case records (Cecil, 1970), has suggested that the Q-value can give a reliable estimate of in-situ E_d and a range of likely deformation. According to Singh (1996), E_d varies from 5 $\log_{10}Q$ to 20 $\log_{10}Q$ for Himalayan rock mass. Singh (1997) gave correlation for E_d for weak and nearly dry rock masses.

Barton (1998), based on case records collected by Chen and Kuo (1997), proposed equations for absolute radial deformation in both the vertical and horizontal directions. These equations are not applicable for soft ground tunnels with exceptionally low compressive strengths and for exceptionally low local Q-values.

2.1.8 Rock Mass index (RMi) - System

Palmstrom (1995a, b, c, d, 1996a) proposed a Rock Mass index (RMi) to characterize rock mass strength as a construction material. RMi is a volumetric parameter indicating the approximate uniaxial compressive strength (UCS) of a rock mass by combining q_c and a jointing parameter (JP). JP represents the main jointing features viz. block volume plus roughness, alteration and size of the joints. The ratings of various parameters have been given in APPENDIX – VI.

Palmstrom (2000a) also proposed charts for the design of support with SFRS and rock bolts as main supporting elements for blocky and continuous grounds. The ground is classified as blocky or continuous on the basis of diameter of tunnel and block diameter.

2.1.8.1 Application of RMi

Some of the RMi applications are discussed as follows:

(i) Determining constants in the Hoek-Brown failure criterion for rock masses

In its original form, the Hoek-Brown failure criterion for rock mass is expressed in terms of the major (σ_1) and minor (σ_3) principal stresses at failure. For $\sigma_3 = 0$, the UCS of a rock mass (q_{cmass}) is indicated in Eq. 2.6:

$$\sigma_1 = q_{cmass} = q_c\, s^{0.5} \tag{2.6}$$

Since both RMi and Eq. 2.6 express q_{cmass}, RMi can be used to determine the constants s and m. The constant s can be found from the JP:

$$s = (JP)^2 \tag{2.7}$$

Wood (1991) and Hoek et al. (1992) introduced the ratio m/m_i. Palmstrom (1996c) showed that m, which varies with the jointing, can be expressed as follows:

Undisturbed rock mass:

$$m = m_i\, JP^{0.64} \tag{2.8}$$

Disturbed rock mass:

$$m = m_i\, JP^{0.857} \tag{2.9}$$

Applying Eqs. 2.7 and 2.8, Hoek and Brown (1980b) failure criterion for undisturbed rock masses can be written as,

$$\sigma_1 = \sigma_3 + \left[q_c . JP^{0.64} (m_i . \sigma_3 + q_c . JP^{1.36})\right]^{0.5} \tag{2.10}$$

Here, s and m have been replaced by JP and m_i respectively.

(ii) Rock burst and spalling in brittle rocks

Rock burst is also commonly known as spalling or popping. Selmer-Olsen (1964) and Muir Wood (1979) mentioned that great differences between horizontal and vertical stresses would increase rock burst activity.

Palmstrom (1996b) characterized failure modes in brittle, massive rock as indicated in Table 2.1, based on works by Hoek and Brown (1980b), Russenes (1974) and Grimstad and Barton (1993).

Table 2.1: Characterization of Failure Modes in Brittle, Massive Rock

Competency Factor $C_g = f_\sigma . q_c / \sigma_\theta = RMi/\sigma_\theta$	Failure Modes in Brittle, Massive Rocks
> 2.5	No rock stress induced instability
2.5 - 1	High stress, slightly loosening
1 - 0.5	Light rock burst or spalling
< 0.5	Heavy rock burst

(iii) Squeezing in continuous ground

Palmstrom (1996b) characterized squeezing ground using RMi as indicated in Table 2.2, based on studies by Aydan et al. (1993) of Japanese tunnels located in mudstones, tuffs, shales and other ductile rocks with $q_c < 20$ MPa.

Palmstrom (1995b) claimed that the application of RMi in rock support involves a more systematized collection and application of the input data. RMi also makes use of a clearer definition of the different types of ground. It probably covers a wider range of ground conditions and includes more variables than the two main classification systems, the RMR and the Q system.

Table 2.2 Characterization of Ground and Squeezing Activity

Squeezing Class	The Tunnel Behaviour According to Aydan et al. (1993)
No squeezing $RMi / \sigma_\theta > 1$	The rock behaves elastically and the tunnel will be stable as the face effect ceases
Light squeezing $RMi / \sigma_\theta = 0.7 - 1$	The rock exihibits a strain-hardening behaviour
Fair squeezing $RMi / \sigma_\theta = 0.5 - 0.7$	The rock exihibits a strain-softening behaviour, and the displacement will be larger
Heavy squeezing $RMi / \sigma_\theta = 0.35 - 0.5$	The rock exihibits a strain-softening behaviour at much higher rate
Very heavy squeezing $RMi / \sigma_\theta < 0.35$	The rock flows which will result in the collapse of the medium and the displacement will be very large

2.1.8.2 Benefits in the RMi support method

As claimed by Palmstrom (2000a), some of the benefits in the RMi support method are:

- The use of the three-dimensional block volume will generally improve the characterization of the rock mass and hence lead to better estimates, compared to the use of RQD and joint spacing applied in other support methods.
- The RMi support method includes all important ground parameters; more than other main classification systems used for rock support estimates.
- The use of different methods for support estimates in ground of different behaviour is reflected in different equations and calculations.

2.1.8.3 Limitations in the RMi support method

According to Palmstrom (2000a), some of the limitations in the RMi support method are:

- The RMi support method does not cover soil or soil-like materials, except where the material occurs in seams or small weakness zones with thickness less than a few metres.
- It is difficult to calculate the magnitude of tangential stresses in blocky ground, which in turn reduces the quality of the support assessment, specially to assess appropriate time-dependent rock support.
- The calculation of the parameters is more difficult than for the RMR and the Q systems as it involves exponential equations.

Palmstrom et al. (2000) opined that the forces (water and overburden) acting on the rock mass should not form part of any initial rock mass characterization. According to them, RMR and Q are actually classification systems for the design of ground support rather than characterization systems.

Barton (2001) concluded that water and stress are fundamental to rock mass characterization and classification. According to him, truncated characterization in RMi rejects the influence of these two fundamental parameters in the estimation of deformation modulii and seismic velocities, which is illogical. Barton (1999, 2002a, b) emphasized that deformation modulii and seismic velocities do not correlate with jointing parameter alone, but with the complex non-linear response of these properties to the "internal" and "external" boundary conditions, which often give anisotropic as well as depth-dependent properties.

Singh and Palmstrom (2002) pose the question whether rock engineering shall be made according to a process where rock classification is based merely on a 'all-in-one' number representing the ground, or if it shall be a stepwise process where the process is divided into certain levels for the parameters to be evaluated, studied and/or compared.

2.1.9 Rock Mass Number (N) and Rock Condition Rating (RCR)

Kaiser et al. (1986) opined that SRF is probably the most contentious parameter. They concluded that it might be appropriate to neglect the SRF during RMC and to assess the detrimental effects of high stresses separately. However, they have not given any alternate approach to assess effect of high stress.

Goel (1994) and Singh et al. (1997, ref. 149) opined that estimating a correct value of SRF is difficult and incorrect selection of SRF-values may lead to an unreliable prediction. According to Goel (1994), the problems in obtaining a correct value of SRF are mainly – for weakness zones intersecting an excavation: a large range of SRF-values are suggested if a shear zone only influences but does not intersect the excavation. For 'competent rock masses' the determination of SRF is based on σ_1, σ_3, q_c and q_t values and the suggested values of SRF have large ranges. For 'squeezing and swelling rock masses' it is practically difficult to even identify the category. Furthermore, Goel (1994) experienced that the SRF does not adequately represent the stress condition of a rock mass, at least for squeezing ground conditions.

Keeping this problem in mind, Goel et al. (1995a) proposed Rock Mass Number (N). They also proposed another method called as Rock Condition Rating (RCR). These indices are the modified versions of the two most popular classification systems, N from the Q-system of Barton et al. (1974) and RCR from the RMR-system of Bieniawski (1973).

N is stress-free rock mass quality Q. Stress effect has been considered indirectly in the form of overburden height. N is defined as in Eq. 2.11:

$$N = \frac{RQD}{J_n} \cdot \frac{J_r}{J_a} \cdot J_w \tag{2.11}$$

RCR is defined as RMR without ratings for the UCS of the intact rock material and the adjustment of joint orientation. This is given as follows:

$$RCR = RMR - (\text{ratings for UCS} + \text{adjustment of joint orientation}) \quad (2.12)$$

RCR is free from the UCS, which is a parameter sometimes difficult to obtain at the site. Moreover, parameter-wise, N and RCR become equivalent and can be used for the purpose of inter-relation. According to Goel et al. (1995c), RMR and Q are not equivalent, and a good correlation is obtained, as given in Eq. 2.13, if N and RCR are considered:

$$RCR = 8 \text{ Ln } N + 30 \quad (\text{coefficient of correlation, } r = 0.92) \quad (2.13)$$

Singh et al. (1992) suggested an empirical approach for predicting squeezing or non-squeezing ground condition based on Q and H:

$$H = 350\, Q^{1/3} \quad \text{m} \quad (2.14)$$

Singh et al. (1995) divided the squeezing ground condition into three classes, on the basis of tunnel closures as mild, moderate and high squeezing ground conditions (APPENDIX - IIC).

Goel (1986) found clear trends for the variation of rock pressure with overburden and tunnel closure. Predicted support pressure, even for squeezing rock conditions, match with the observed support pressure. Goel et al. (1995b) and Goel (2001) suggested an empirical approach for predicting squeezing and non-squeezing ground condition using N:

$$H = (275\, N^{0.33}).\, B^{-0.1} \quad \text{m} \quad (2.15)$$

where, B is in m.

Goel et al. (1995b) and Goel (2001) developed an additional criterion to demarcate self-supporting condition from the non-squeezing condition:

$$H = 23.4\, N^{0.88} . Bs^{-0.1} \quad \text{m} \quad (2.16)$$

$$Bs = 2\, Q^{0.4} \quad \text{m (Barton et al., 1974)} \quad (2.17)$$

Goel et al. (1995b) and Goel (2001) further proposed following Eqs. to distinguish between mild, moderate and high squeezing ground conditions:

Mild and moderate squeezing:

$$H = (450\ N^{0.33}).\ B^{-0.1} \quad m \tag{2.18}$$

Moderate and high squeezing:

$$H = (630\ N^{0.33}).\ B^{-0.1} \quad m \tag{2.19}$$

where, B is in m.

As claimed by Goel et al. (1995b, 2001), a practical benefit of correlations for prediction of ground condition involving N is that the tunnel depth, N and the tunnel width can be chosen to avoid a squeezing ground condition, thereby reducing problems and the construction time. Reduction in tunnel width by increasing the number of tunnels should be chosen as a last alternative only, when the options of realigning the tunnel through a better rock mass or under a reduced depth are not possible. Even when it is not possible to go in for two smaller tunnels due to design considerations, the information about possible occurrence of squeezing will be helpful in making advance planning for the use of a flexible support system and deciding the excavation size to accommodate desirable tunnel closure.

2.1.10 Geological Strength Index (GSI)

Hoek and Brown (1997) introduced the Geological Strength Index (GSI) based on following two correlations:

$$GSI = RMR - 5 \qquad \text{for } GSI \geq 18 \text{ or } RMR \geq 23 \tag{2.20}$$

$$GSI = 9\ L_n\ Q' + 44 \qquad \text{for } GSI < 18 \tag{2.21}$$

where, $Q' = \dfrac{RQD}{J_n} . \dfrac{J_r}{J_a}$ (2.22)

Hoek (1994), and Hoek and Brown (1997) found the following correlation for estimation of rock mass constants using GSI:

Original rock mass:

$$m = m_i \,.\, e^{\left(\frac{GSI-100}{28}\right)} \tag{2.23}$$

$$s = e^{\left(\frac{GSI-100}{9}\right)} \tag{2.24}$$

Broken rock mass:

$$m_r = m_i \,.\, e^{\left(\frac{GSI-100}{14}\right)} \tag{2.25}$$

$$s_r = e^{\left(\frac{GSI-100}{6}\right)} \tag{2.26}$$

Hoek and Brown (1997) recently proposed a chart for GSI as experts can classify a rock mass by visual inspection alone. In this classification, there are four main qualitative classifications, adopted from Terzaghi's classification (APPENDIX - IIA).

2.2 EFFECT OF TUNNEL SIZE ON SUPPORT PRESSURE

Prediction of support pressures in tunnels and effect of tunnel size on support pressure are the two important problems of tunnel mechanics, which were addressed by many researchers in the past. Some researchers demonstrated that the support pressure is independent of tunnel size (Daemen, 1975; Jethwa, 1981; Barton et al., 1974; Singh et al., 1992) whereas others advocated that the support pressure is directly dependent on tunnel size (Terzaghi, 1946; Deere et al., 1969; Wickham et al., 1972; Unal, 1983). Verman (1993) also suggested that the support pressure is practically independent of the tunnel width, provided support stiffness is not lowered.

2.2.1 Influence of Shape of Opening

Some empirical approaches have been developed for flat roof and some for arched roof. In case of an underground opening with flat roof, the support pressure is generally found to vary with the size of the opening, whereas in the arched roof the support pressure is found to be independent of tunnel size excepting the RSR-system of Wickham et al. (1972).

The hypothesis, that the stability of an excavation is independent of its size because the stresses induced in the rock around the excavation are independent of the size of excavation, may not be acceptable from practical point of view since the rock mass is not perfectly elastic and it contains fractures also. Even if the stresses are same, the stability of an excavation in a fractured and jointed rock mass will be controlled by the ratio of excavation size to the size of the blocks in the rock mass. Consequently, increasing the size of an excavation in a typical jointed rock mass may not cause an increase in stress, but it will almost certainly give rise to a decrease in stability (Hoek and Brown, 1980).

2.2.2 Influence of Rock Mass Type

The support pressure is directly proportional to the size of the tunnel opening in the case of weak or poor rock masses, whereas in good rock masses the situation is reverse. Hence, it can be inferred that the applicability of an approach developed for weak or poor rock masses has a doubtful application in good rock masses.

According to Goel (1994), the predicted support pressure by Unal (1983) is safe for medium size tunnels (diameter 6 - 9 m), but it is unsafe for small tunnels (diameter 3 - 6 m) and conservative for large tunnels (diameter 9 - 12 m). He concluded that the correlation of Unal (1983) would not be applicable for rock tunnels through hard rocks with an arched roof, as more emphasis on tunnel size has been given. On the other hand, for squeezing ground condition, the predicted support pressures are unsafe for all sizes of tunnels.

Goel et al. (1995a) evaluated the approaches of Barton et al. (1974) and Singh et al. (1992). They found that the approach of Barton et al. (1974) is unsafe in squeezing ground conditions and the reliability of the approaches of Singh et al. (1992) and that of Barton et al. (1974) depends upon the rating of Barton's SRF.

Goel et al. (1996) found that there is a negligible effect of tunnel size on support pressure in non-squeezing ground conditions, but the tunnel size could have considerable influence on the support pressure in squeezing ground condition.

Jethwa et al. (1980), based on the studies conducted at Chhibro-Khodri tunnel of the Yamuna Hydroelectric Scheme near Dehradun in the Lower Himalayan region, concluded that:

- Tunnel depth and the effect of opening size are not considered adequately in the empirical approaches of Terzaghi (1946) and Barton et al. (1974). As such, these methods failed to account for rock conditions. A deeper tunnel under these conditions is likely to attract higher rock loads unless greater closures are allowed.
- Stiffer supports were found to attract higher rock loads under squeezing rock conditions. A flexible support system, consisting of steel arches with loose backfill, caused significant load shedding.

2.3 IMPORTANT CONSIDERATIONS IN THE SUPPORT DESIGN

Jethwa et al. (1978) concluded, on the basis of experience gained from 4.75 m diameter tunnel of Maneri-Bhali Hydroelectric Project, that reinforcement in the concrete liner should be eliminated to induce tension cracks so that when the tunnel is emptied these cracks have a moderating effect to make the externally applied stress field hydrostatic. In such an event, plain cement concrete lining is capable of withstanding the entire hoop load, provided they are hydrostatic, otherwise lining may fail in bending.

Ward (1978) opined that tunnelling in weak rock is much more of an art than soft ground tunnelling, that's why it is important to monitor the performance. There is no such thing as the load on a support in weak rock. He further concluded that the consequences of attempting to be conservative by adopting stronger and stiffer supports in deep tunnels, when yielding is seen to develop, might be extremely expensive and often futile.

According to Braun (1980), in deep tunnel support system, horizontal gaps are left in the concrete lining between crown and tunnel walls and telescopic sliding joints are provided in the steel arch, which allow for rock movements of up to 500 mm.

According to Dube (1979), the radius of broken zone is a very important parameter in influencing the rock pressure. Dube et al. (1982, ref. 56) presented a graphical method to determine the average radius of broken zone. They concluded that the radius of the broken zone increased with the tunnel face advance and the same was stabilized at a distance equal to 5 to 15 times the radius of the tunnel opening. They further concluded that the radius of the broken zone is 5 to 8 times the radius of the tunnel.

According to Ward et al. (1983), loading on a support is dependent on how close the support is constructed to the face and on the stiffness of the support. The steel ribs with their compressible lagging which is less stiff support constructed close to the face but allowed the rock to yield, only supported a small load.

Einstein et al. (1983) applied Terzaghi's approach, RQD, RSR, RMR and Q systems at Porter Square Station, a 21 m span subway station cavern in Cambridge Argillite in four phases (boring records, exploration shaft, pilot tunnel, cavern survey). Results of study showed that rock classification and support prediction for average rock conditions were comparable.

According to Jethwa et al. (1984), the rock support interaction modifies the elasto-plastic stress distribution, which in turn helps the failed rock mass within the broken zone to relax elastically. The failed rock mass thus gets compacted adjacent to the supports. They found that the ultimate rock pressure vary from 13 to 33 percent of the cover pressure for most rock masses, but may exceed 40 percent of the cover pressure for tunnels through clays or clayey fault gouges.

Jethwa and Dhar (1996) opined that in failed rock mass, a thinner support permits a faster stress release and quicker stabilization of the opening. The ultimate pressure distribution around a tunnel lining is likely to become uniform, which would therefore be subjected to hoop compression in contrast to a thicker lining inviting bending moments.

Sharma et al. (1987) and Sharma (1990) compared the convergence through rock mass – support interaction analysis with measurements in squeezing ground for a tunnel in Himalayan region, and found that the recorded rock pressures were close to those calculated.

According to Fairhurst (1993), the important principle in rock mass – support is that it is sufficient to observe the radial convergence of a tunnel as a function of time in order to assess whether or not the tunnel will stabilize. If observation of the tunnel convergence with time shows increasing convergence rate, the logical step is to increase the support. On the other hand if the support had been designed to yield, then equilibrium would eventually be reached. Carranza-Torres and Fairhurst (2000) consider convergence-confinement method is a useful tool not only for the design of supports in tunnels, but also as a simple illustrative model that allows a better understanding of the complex problem of transference of loads in the vicinity of the tunnel face.

Franklin (1993) considers the development of computer-based 'expert systems', an encouraging new direction in this field. An expert system called 'Classex' (Butler and Franklin, 1990) may be used to compute Q, RMR, RQD and RSR.

2.4 JUSTIFICATION OF THE PROBLEM

The review of literature reveals the following major gaps of importance:

(a) Six RMC systems such as RSR, RMR, Q, RMi, N and RCR have not been applied so far in an underground excavation. In the present study, these systems have been applied in the longest power tunnel in the world, which encountered all kinds of ground conditions.

(b) The development of computer program for RMC using RSR, RMR, Q and RMi systems may help the field engineers in selecting most appropriate support system and in better understanding of rock mass.

(c) RMC systems such as RSR, RMR and Q are suitable in their present form mainly for solid rocks; their suitability in jointed rocks of Himalaya under severe ground conditions has to be thoroughly investigated. RMi-system has only recently been developed in Norway, applicability of which is not yet established in many types of rock masses.

(d) The RSR-system contains ambiguous parameters such as hardness of rock and geological structure. Furthermore, it has given recommendations for estimation of spacing of steel ribs as per FPS system of units for different tunnel diameters for datum condition. Moreover, these recommendations have been made for tunnel diameters up to 9 m only.

(e) In the support recommendations given by Bieniawski (1989), only ranges have been indicated for thickness of shotcrete, length and spacing of rock bolts and spacing of ribs for various rock mass classes.

(f) The Q-system does not suggest SRF-values for 'competent rock, rock stress problems' category for moderately jointed rocks in categories L, M and N.

In view of above discussion, problem of rock mass classification and evaluation of supports was taken up in the proposed work for detailed investigation.

GEOLOGICAL SETTING

The Indian sub-continent is divisible into three well-marked geographic regions. They are Peninsular shield in the south, Extra-Peninsular in the north and the Indo-Gangetic alluvial plain, lying between the Indian Peninsula and Extra-Peninsula.

The Peninsular shield is composed primarily of Precambrian rocks. The Extra-Peninsula includes the mountains that extend from Baluchinstan ranges (NNE-SSW) in Pakistan through Himalaya (mainly E-W) in India, Nepal and Bhutan, to Arakan-Yoma ranges (N-S) on the borders regions of India and Myanmar. These mountain ranges, which are in the form of arcs with convexity towards the Peninsula, have formed due to collision of the Indian Plate with the Eurasian Plate during Tertiary age. The Indo-Gangetic alluvial plains are comprised of over 4 km thick alluvial deposits of Quaternary age (Gupta et al., 1996).

3.1 HIMALAYA

'Himalaya' got its name from Sanskrit word 'Him', which means snow and 'alaya', which means home or abode. Himalayan arc, which is about 2500 km long with a variable width (160 to 400 km), comprises of a series of several more or less parallel or converging ranges intersected by enormous valleys and extensive plateaus. A number of antecedent rivers that originate in the central parts of Himalaya cut across the high ranges, dividing Himalaya into four main regions viz. the Punjab Himalaya (550 km), the Kumaon Himalaya (320 km), the Nepal Himalaya (800 km) and the stretch of Himalaya lying between rivers Teesta and Brahmaputra.

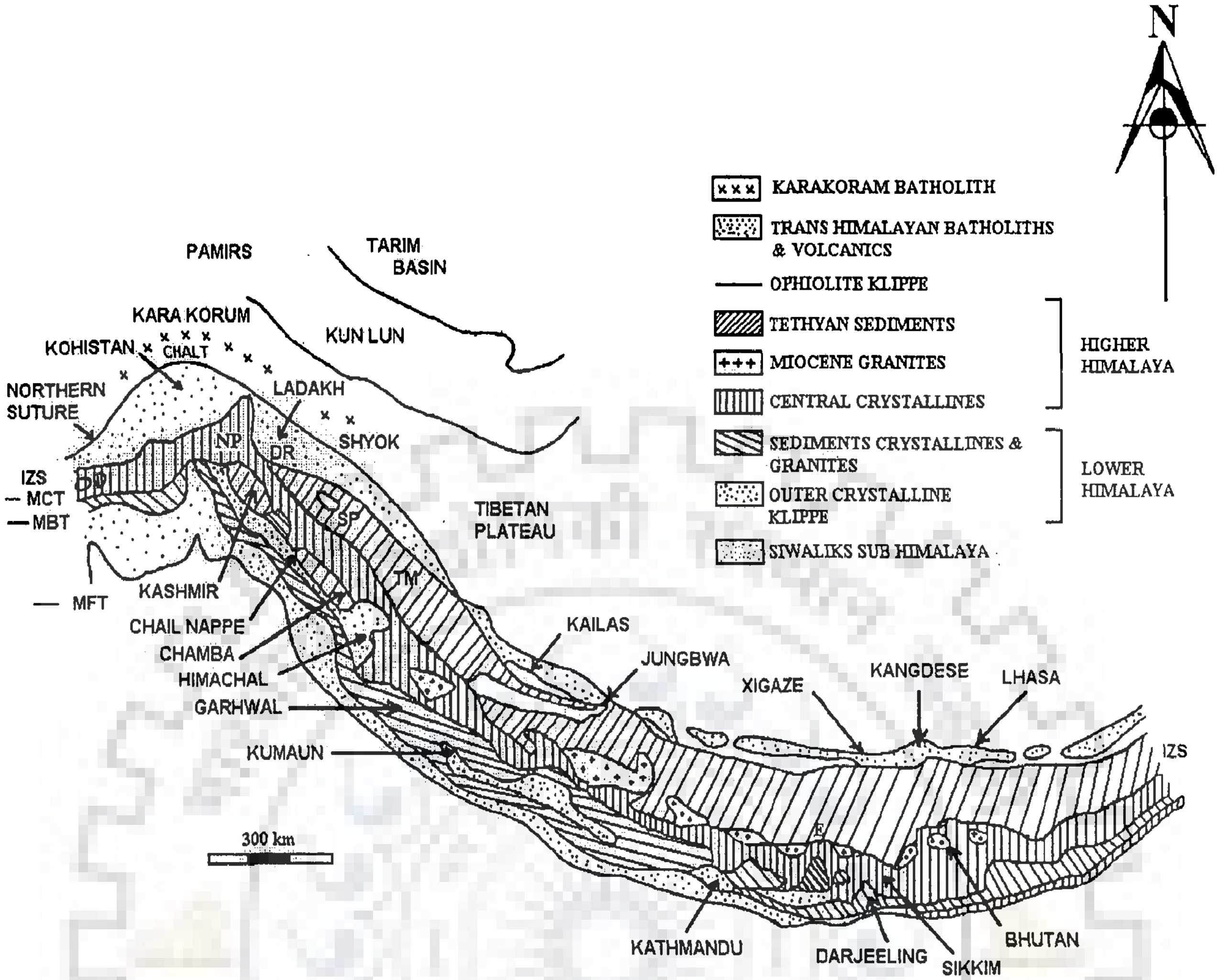
The evolution of Himalaya is a result of repeated deformation of the sedimentary successions accumulated in the Tethys Sea. Topographically the highest but the youngest in age, Himalaya is characterised by its distinctive structural architecture and unique sedimentancy and tectonic history.

From the physiographical point of view, Himalaya have been divided into four lateral zones (Figs. 1.1 and 3.1). These are, from south to north: (1) the Outer Himalaya (also known as Sub-Himalaya), (2) the Lesser Himalaya (also known as Lower Himalaya), (3) the Higher Himalaya and (4) the Tethys Himalaya (also known as Tibetan Himalaya). Northern limit of Himalaya is marked by Indus Suture Zone, which is the junction of the Indian and Eurasian Plates. The ranges beyond the Suture Zone are referred to as the Trans Himalaya. Considering the disposition of tectonostrategraphic units in nearly east-west trending belts and the important dislocation zones which separate them, the above diversion roughly correspond to various tectonic belts: (1) Foot hill tectonic belt, (2) Inner tectonic belt, (3) Axial tectonic belt and (4) Trans–axial tectonic belt respectively.

3.2 REGIONAL GEOLOGY

In Himachal Pradesh, low to medium grade regionally metamorphosed pelitic and semipelitic rocks have been classified variously as the Chail, Jutogh and Salkhala Group. The rocks exposed in the area belong to the Outer Crystalline Unit of the Higher Himalaya. The project area lies in the Jutogh Group of rocks, surrounded by the rocks of Salkhala Group and Rampur Group.

The Jhakri-Wangtu region along the Satluj valley provides a good cross section of the Higher Himalayan Crystalline (HHC). The Higher Himalayan metamorphic belt along the Satluj valley comprises of garnetiferous mica schist in the immediate vicinity of Jutogh thrust and is



D = DARGAI, OPHIOLITE, DR = DRASS, E = EVEREST, NP = NAGA PARBAT, SP = SPONGTANG OPHIOLITE, TM = TSO MARARI, IZS = INDUS YARLUNG ZANGBO SUTURE, MCT = MAIN CENTRAL THRUST, MBT = MAIN BOUNDARY THRUST, MFT = MAIN FRONTAL THRUST

Fig. 3.1 Map of Himalaya showing main tectonic zones and key localities (Windley, 1983) followed by mica schist and quartz mica schist, amphibolite and gneiss at higher structural level (Figs. 3.2, 3.3 and 3.4).

The metamorphism in these rocks is essentially of regional type as rocks show the same grade of metamorphism over an extensive area, except local variations.

The project area lies over two major lithotectonic units, which are discussed as follows.

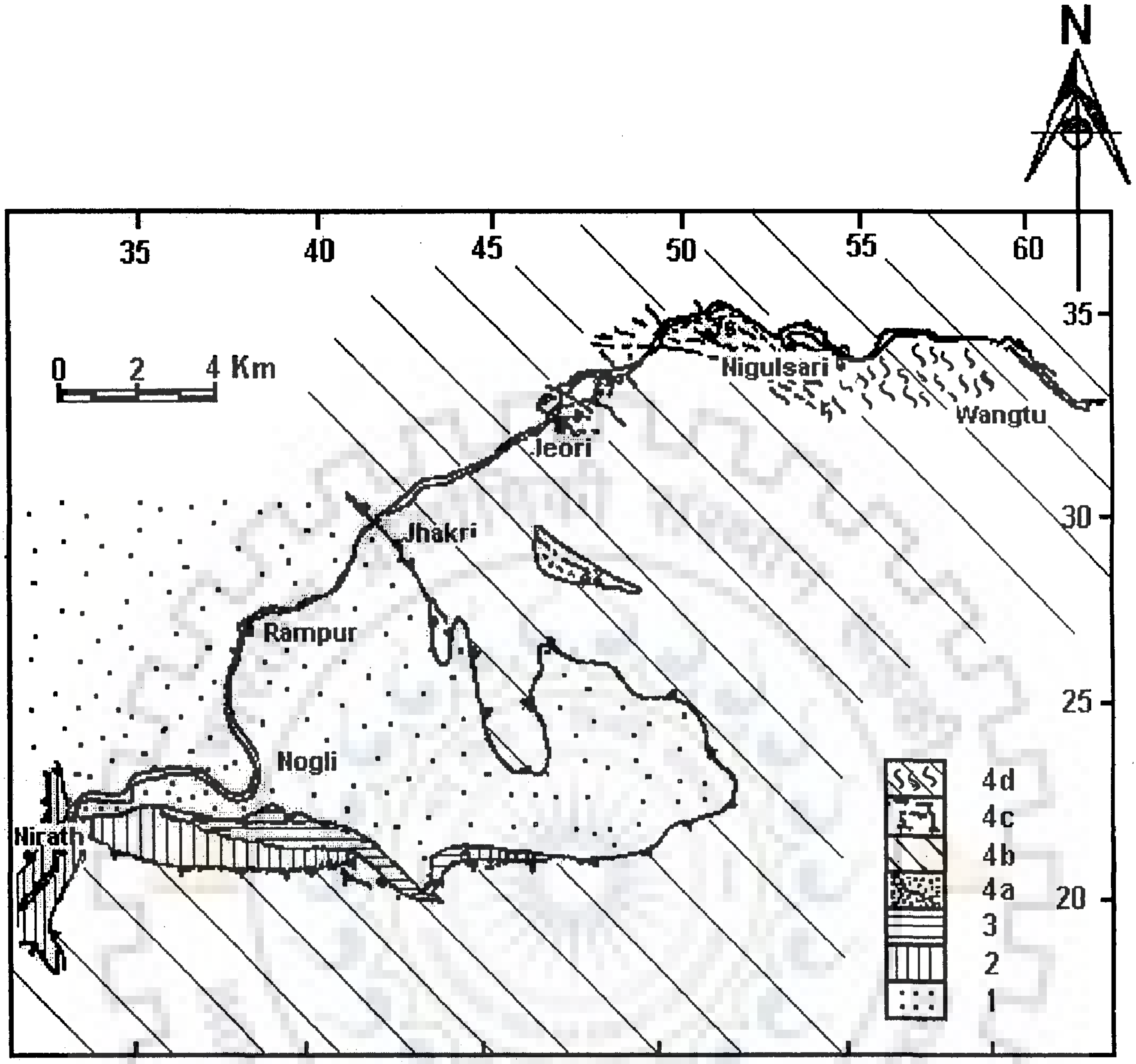
3.2.1 Wangtu Gneissic Complex

The gneissic complex is regarded as the basement of the Higher Himalayan rocks. In the northeastern part of the area the rocks are predominantly gneisses with basic (amphibolite) and acid (granite, pegmatite and quartz vein) intrusives. These gneisses are known as Wangtu gneisses and are of three types: (i) massive, (ii) banded and (iii) augen gneisses. In composition, first two types are rather poor in feldspar, while augen gneiss contains well-developed augens of feldspar. These Wangtu gneisses (Fig. 3.3) have a tectonised contact with the underlying Jeori Formation along the Chaura Thrust.

The acid intrusive are generally discordant in nature and the basic intrusive, which are amphibolites, are concordant and occur mostly along foliation of the country rock. These have been affected by subsequent metamorphism.

3.2.2 Jeori Formation

The HHC belt includes the Jeori Formation (Fig. 3.3) near its base and is present towards the southeastern part of the project area. This Formation lies between Main Central Thrust (MCT) and the newly recognized Chaura Thrust. Earlier this formation has been designated as the Sarahan Series or as a part of the Wangtu gneiss. This formation consists of garnetiferous mica schist, quartz muscovite schist and amphibolite. The schists are intruded by quartz veins of varying



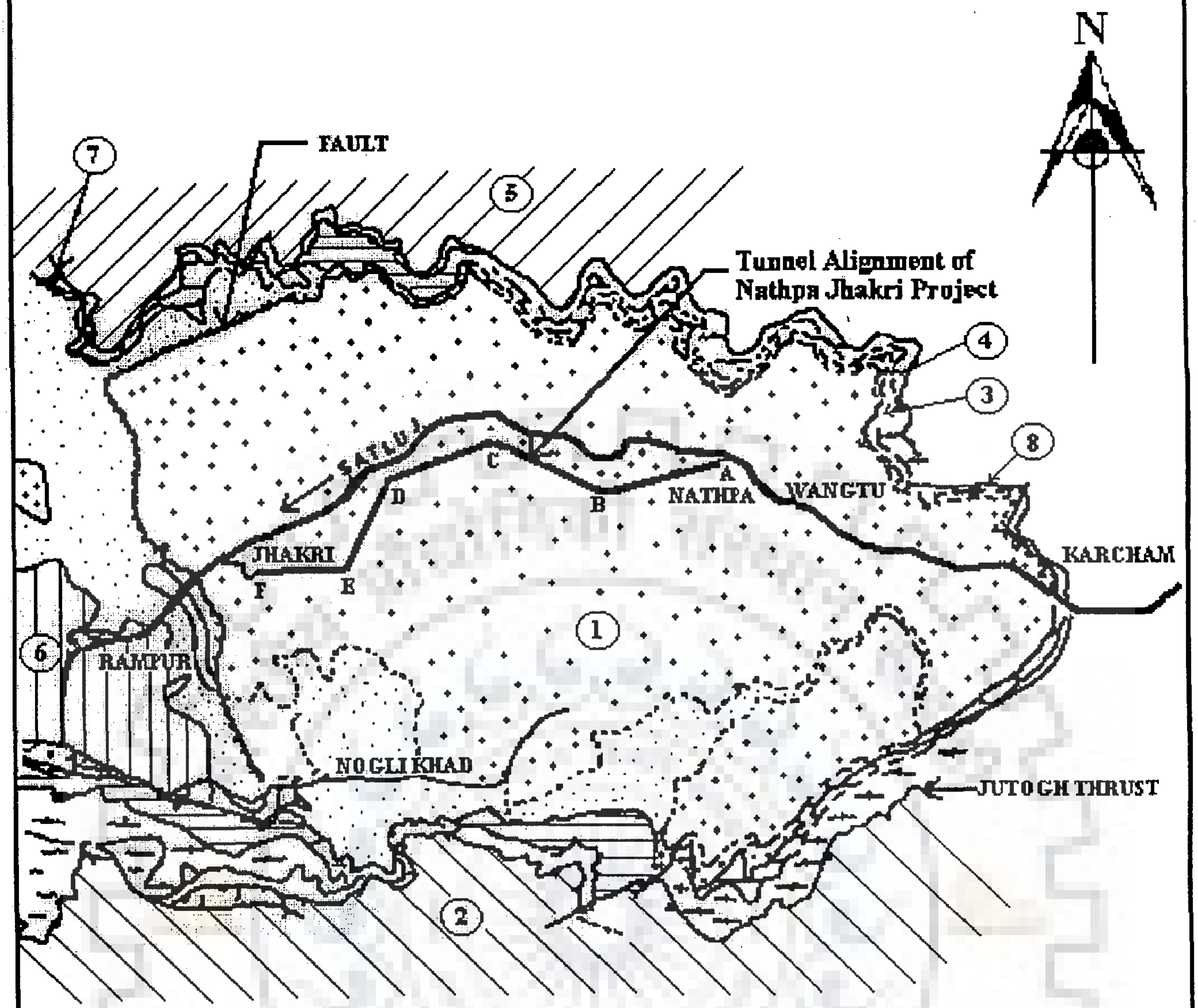
1 RAMPUR-KULU WINDOW 2 BAJURA-KULU NAPPE

3 CHAIL NAPPE 4 JUTOGH NAPPE

4a QUARTZ MICA SCHIST 4b GARNETIFEROUS MICA SCHIST

4c AMPHIBOLITE 4d WANGTU GNEISSES

Fig. 3.2 Lithotectonic succession along Satluj valley



ABCDEF : THE HEAD RACE TUNNEL OF NATHPA JHAKRI PROJECT

1 JEORI WANGTU GNEISSIC COMPLEX (PRE CAMBRIAN)		
2 JUTOGH GROUP (PRE CAMBRIAN)		
3 KHAMRADA FORMATION		
4 GOHR FORMATION	SALKHALA GROUP (PRE CAMBRIAN TO SILURIAN)	
5 KULU FORMATION		
6 PATBOBA FORMATION		
7 GREEN BED FORMATION	RAMPUR GROUP	
8 MANIKARAN FORMATION		

Fig. 3.3 Geological plan of the project area

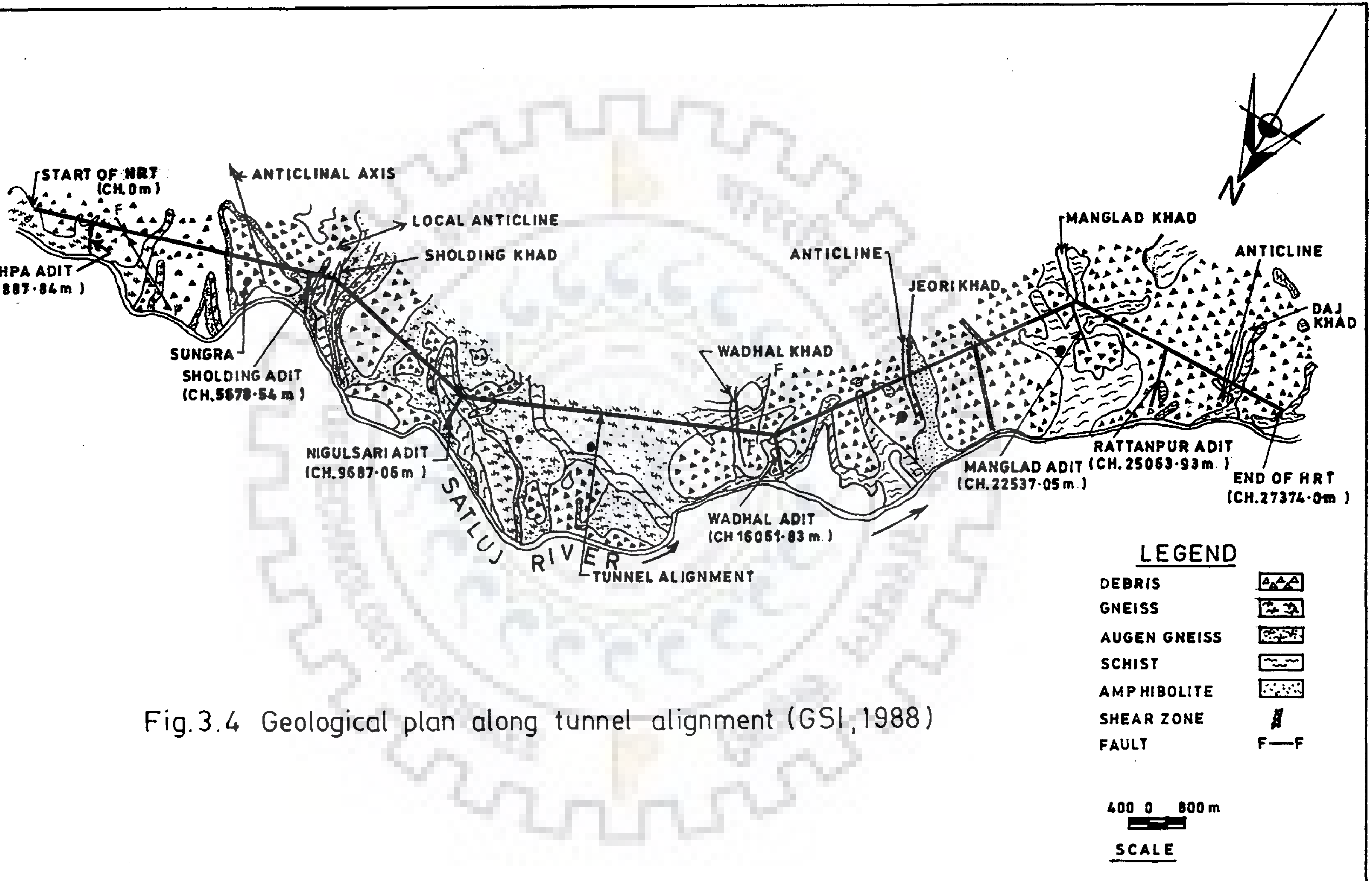


Fig.3.4 Geological plan along tunnel alignment (GSI, 1988)

thicknesses. Quartz boundins are also in abundance. These boundins are formed due to metamorphic sweating and these are at places more than a metre in length.

(i) Garnetiferous Mica Schist

It is the most abundant lithology in the basal part of the HHC in immediate vicinity of the MCT between Jhakri and Jeori. A metapelite sequence with a few psammitic bands is made up of light to dark grey, garnetiferous mica schist with very thin alternating bands of quartzofeldspathic and mica rich gneiss. These are extremely well foliated, medium grained schists and gneisses. The main foliation is characterized by strong preferred orientation of biotite and muscovite with interspeared quartz. Along the Manglad Khad, discordant fine-grained grey granite gneiss, aptite and pegmatite intrude the metapelite as concordant to discordant igneous bodies.

(ii) Quartz Mica Schist

A small outcrop of quartz mica schist is exposed along the upper reaches of Manglad Khad, a tributary of river Satluj. It is light grey quartz rich schist and contains quartz, muscovite and chlorite.

(iii) Amphibolite

Fine to medium grained, dark coloured and highly foliated amphibolite bodies are occasionally exposed along the Jeori Formation. These are mostly 1 to 10 m thick, except for a mapable band of about 300 m thickness near Jeori along NH-22.

3.2.3 Jutogh Group

According to Jalote et al. (1988) the rocks of Jutogh Group form the northern limb of a big syncline. But according to Jangi and Gaur (1972-73), the quartzite in Rampur, having northeasterly strike with southeasterly dips, and those in Karchham area having northerly strike and easterly dips

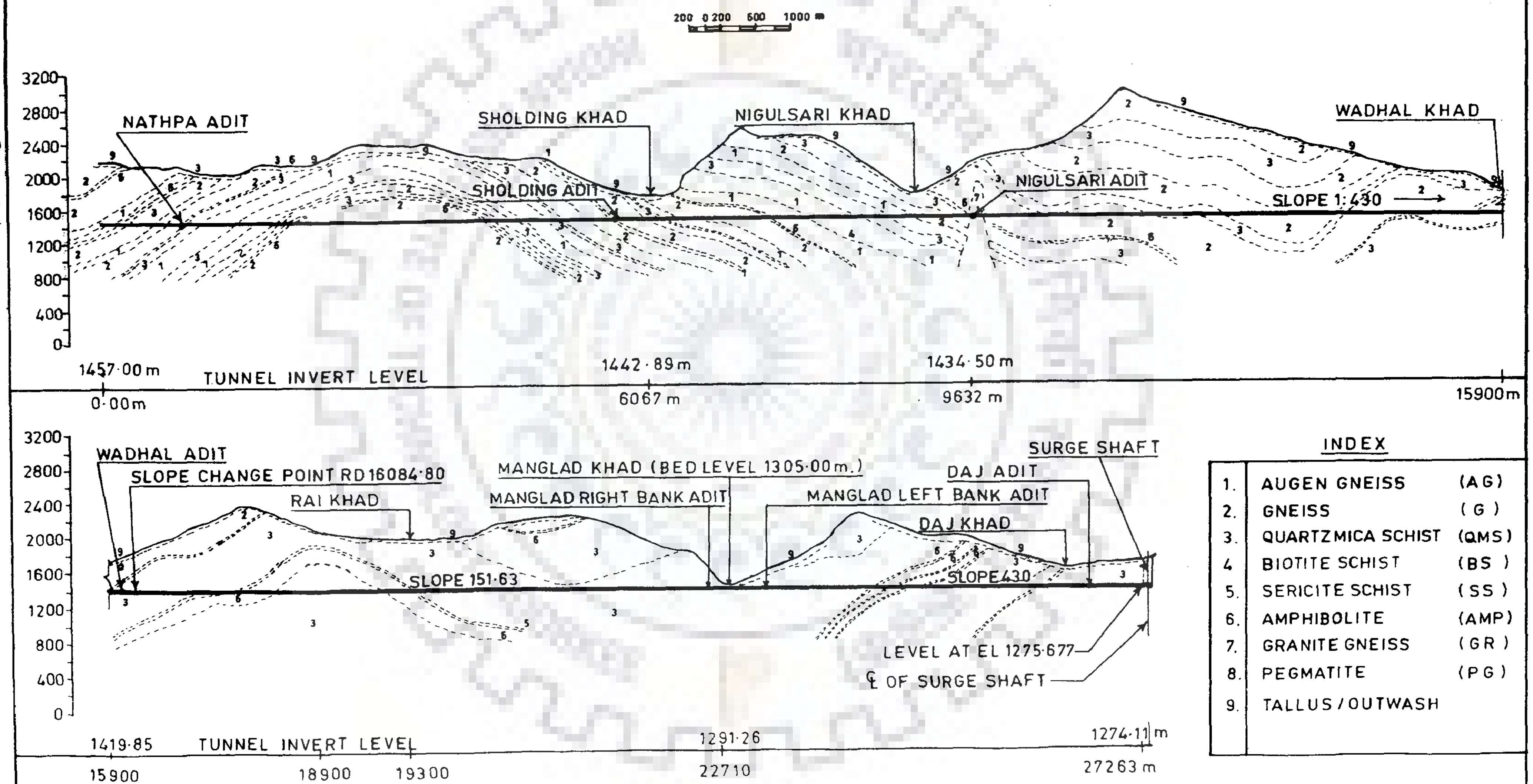


Fig. 3.5 Geological section along tunnel alignment (GSI, 1988)

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3.2.3 Jutogh Group

According to Jalote et al. (1988) the rocks of Jutogh Group form the northern limb of a big syncline. But according to Jangi and Gaur (1972-73), the quartzite in Rampur, having northeasterly strike with southeasterly dips, and those in Karchham area having northerly strike and easterly dips are believed to be in strike continuity. This change in attitude is attributed to an anticlinal structure with its axis passing through Karcham and plunging in the easterly direction.

The rocks of Jutogh Group are folded with four generation of folds and are intersected by a number of steeply dipping faults and shear zones. Major fold axis trends in NW-SE to E-W direction. Superimposed on these broad folds are numerous minor folds with varying trends. The schists of the area are more foliated than gneisses. The foliation trend generally varies from $N70^0W-S70^0E$ to $N70^0E-S70^0W$ having average dip of the order of 35^0 to 70^0 towards north. More than five joint sets have been recorded in the project area. In places gougy seams are also associated with some of the joints specially the foliation joints.

3.3 GEOLOGY IN AND AROUND THE PROJECT AREA

The area west of Jhakri occupied by Rampur Group of rocks having quartzites as the major rock unit is characterized by sharp crested ridges showing tonal banding and well-developed bedding with mostly dip slopes. It has also characteristic joint trellis drainage. Flatirons are prominently developed indicating presence of hard and competent rock with well-developed joints.

The area east of Jhakri up to Sarahan is characterized by rounded ridges and valley, occupied by poorly resistant rocks and has dendritic drainage pattern. These features are due to the presence of essential schists with subordinate gneissic bands. The rocky outcrops form ridge near east of Jeori. These ridges are, however, not as sharply crested as those in the quartzites of Rampur Formation. Also the flatirons are not present in this area. Bands of gneisses are seen within schist near Sungra. The rocks are folded into antiform and synform structures. These folds show a plunge towards NW direction. In addition a number of faults are also present in the project area.

3.4 GEOLOGY ALONG TUNNEL ALIGNMENT

Nathpa Jhakri Project area is located in the Outer Crystalline unit of the Higher Himalaya, which is characterized by very rugged topography with lofty hills (Fig. 1.1). The general altitude of the area is above El. 1000 m. Rock types in the area encompassed by the project comprises a variety of metamorphic rocks belonging to Jeori Wangtu Gneissic Complex of Precambrian age.

Seven construction adits have been provided along the tunnel alignment to facilitate construction from thirteen working faces as per the layout shown in Fig. 1.2. On the basis of geological structure, lithology and location of construction adits, the tunnel alignment has been divided into following sections.

3.4.1 Nathpa – Sholding Section

The section is represented mainly by medium to coarse-grained gneisses and augen gneisses with quartz mica schist and amphibolites. The gneisses become schistose at places due to increase in mica content.

Rocks between Nathpa and Sholding are structurally disposed in a broad anticline with its fold axis running roughly in NW–SE direction (Fig. 3.5). The foliation joints in this part of the tunnel show variation from northeast near Nathpa downstream to southwest in Sholding upstream. The HRT passes through a rock cover of 880 m to 340 m. A number of shear seams of 5 cm to 1 m thickness are present in this area. In Nathpa downstream, a steep dipping shear zone with associated crushed and shattered zone was met for a length of about 47 m. The shear zone comprised mainly of moderately to highly altered gneisses and augen gneisses bounded on either side by fractured zones. An over break cavity was formed at intersection of this shear zone with the tunnel. Moderate to high squeezing has been observed in the shear zone during and after the excavation, and tunnelling conditions in this portion were quite difficult. Another small shear zone

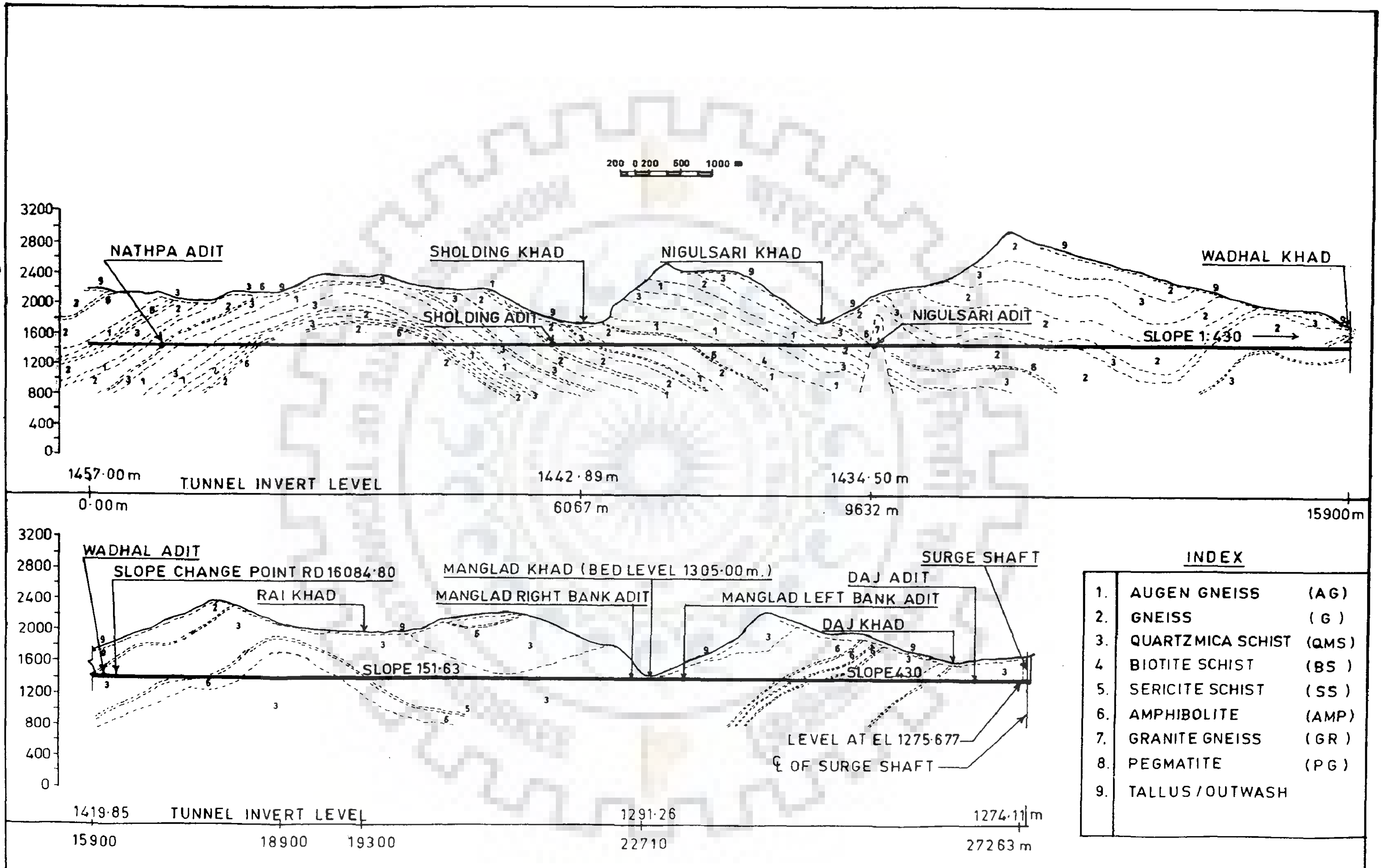

Fig. 3.5 Geological section along tunnel alignment (GSI, 1988)

of 4 m width was encountered 100 m downstream of aforesaid shear zone. In addition to foliation joints, easterly dipping shears, southwesterly dipping shears and northwesterly dipping shears were also encountered with a filling up to 150 cm of crushed rock and gouge. Foliation in Nathpa downstream is striking at an acute angle to the tunnel alignment causing some overbreaks on the right crown of the tunnel. In Sholding upstream reach, a rock fall took place due to concentration of shears and altered rock conditions. Hot water seepage and hydrothermal alteration of litho units has been noticed in this part of the HRT.

3.4.2 Sholding Downstream

In this stretch, the area has steep rocky slopes and escarpments. Part of the HRT in Sholding downstream shows similar lithology as in Nathpa – Sholding section (section 3.4.1), but with more of quartz mica schist and amphibolite containing intrusions of pegmatite. Rock cover above the tunnel goes up to 1000 m. Rocks of this area lie on the western limb of the anticlinal fold discussed in Nathpa – Sholding section (section 3.4.1). A local anticline designated as Tranda anticline is also present in this area. The main foliation joints continue to dip towards southwest to north at an angle of 20^0 to 85^0 with almost similar sheared joints. At a place in this reach, rock fall took place in the tunnel due to presence of chlorite and biotite rich rock with pegmatite intrusion and associated shears. Wherever shear zones were encountered in tunnels, particularly those that are parallel to tunnel, overbreak conditions resulted (Chaddha et al., 1999).

3.4.3 Nigulsari Upstream

The foliation in Nigulsari upstream part of the HRT, continue to show southwesterly dips with dip amounts of 30^0 to 70^0 (dipping towards hill side). Rocks encountered are amphibolite, mica rich gneisses and quartz mica schist in this section. Amphibolite was hard but heavily jointed.

Near the junction with Nigulsari adit, the tunnel encountered a zone consisting of biotite schist with gneisses and quartz mica schist rich in biotite with bands of amphibolite. Due to very mixed nature of this zone, intersection of various sheared joints coupled with heavy ground water seepage, the excavation of tunnel was quite difficult.

3.4.4 Nigulsari Dowstream

Nigulsari downstream excavation shows change in foliation from southwest to northeast after a kilometer of the excavation. This part of the HRT passes through a maximum cover of ± 1430 m. Lithology remains more or less same as in previous sections with pegmatite intrusion becoming more frequent in this part of the tunnel. A number of closely spaced anticlines and synclines are present in certain areas. After change in the dip of the foliation to northeast, the foliation strikes sub-parallel to the tunnel axis for some length of the tunnel, which was responsible for tunnel overbreaks in a few locations.

3.4.5 Wadhal Upstream

Wadhal upstream part of the tunnel encounters quartz mica schist and gneisses with thin bands of amphibolites. The tunnel passes through a rock cover of 200 m to 1000 m. Quartz mica schists sometimes become rich in muscovite, biotite and also at times show mica rich bands. Gneisses become mica rich at places and at times augen gneisses are also encountered. In whole of this section foliation dips towards northeast at an angle of 40^0 to 75^0. This section of tunnel crosses two perennial streams, few seasonal streams and topographic depressions. These streams are the source of continuous seepage during tunnel excavation. Also in the vicinity of these stream crossings, rock condition deteriorated causing collapse of tunnel faces at several locations.

3.4.6 Wadhal – Manglad Section

Rocks encountered in this section of the HRT are quartz mica schist, quartzite and amphibolite. The tunnel in this reach passes through a rock cover of 220 m to 920 m. The foliation joints in general are northwesterly to northeasterly dipping except for some portion in Wadhal downstream where they dip towards southeast. In addition, other joints dipping in easterly, southwesterly and northwesterly direction have also been encountered. Many of these joints were sheared and filled with gouge and crushed rock.

Structurally, rocks form an open anticlinal fold with local flexures. In this section, a stream known as Jeori Khad runs sub-parallel to the anticlinal fold axis and in its upper reaches it flows along the foliation trend of rocks. Numerous cross folds are also present in this section.

3.4.7 Manglad – Rattanpur Section

This section is represented mainly by quartz mica schist which becomes rich in mica in general and quartz boudins at places with the bands of amphibolite. The joint sets earlier encountered in the Wadhal – Manglad section (section 3.4.6) continue in this section also. After the bend in the tunnel (little downstream of Manglad adit junction), the foliation joints strike sub-parallel to the tunnel alignment and dip at 15^0 to 55^0 towards northeast (towards valley). Along with this, another set of joint striking sub-parallel to the tunnel axis and dipping towards southwest (towards hill side) is encountered. Rocks form one limb of a fold.

In this reach, the HRT crosses Manglad stream, which is a perennial drainage having a deep and narrow gorge. The HRT just below the Manglad creek passes through quartz mica schist with a total cover of 9 m. Due to low rock cover below Manglad creek, the tunnel is steel lined for a length of 710 m from Ch. 22557 m to Ch. 23267 m.

During the excavation of Giri-Bata tunnel, the gas was encountered and many persons had severe burn injuries due to fire. In Loktak Hydroelectric Project, some 15 persons were killed and many had burn injuries of high degree due to explosion and fire near the heading.

Sudden on-rush of water has been reported in Sundernagar-Slapper tunnel of Beas Satluj Link Project, Baira-Siul tunnel, Chhibro-Khodri tunnel and Maneri-Bhali tunnel, to name a few (Dhar and Choubey, 1986).

In spite of extensive investigations conducted for the HRT of Nathpa Jhakri Project, several problems were faced during its construction. Some of the most prominent problems faced and tackled are being discussed in the following paragraphs (Source: The Project Authorities).

4.1 GEOTHERMICS

The Precambrian metamorphic rocks of the Satluj valley upstream of Rampur (12 km downstream from the Jhakri powerhouse site) have been intruded by both acid and basic magma. The granites, pegmatites and quartz veins represent acid igneous activity and the amphibolites are the basic igneous rocks.

There are a number of hot springs on the banks of river Satluj and the adjacent areas. Wherever, deeper geo-thermal aquifers are tapped by tectonic dislocations such as faults, shear zones and joints, hot springs are present.

Most of the hot springs in the project area, except the one at Jeori, are at river level. Since the springs at higher levels are cold, it seems that the hot ground area is below river level and the phenomenon is only manifested in the form of hot springs in area of major structural discontinuities, which act as routes for the discharge of hot water.

According to the geophysical surveys, with the exception of the Jeori Khad area, nowhere in the project area resistivity values are low, indicating the presence of geothermal zones. The

Wadhal – Manglad section, which includes the Jeori area, is thus the most vulnerable area as far as geothermal problems are concerned along with the Nathpa – Sholding section (GSI, 1988).

From the geothermic point of view, the tunnel sections, which have been proved vulnerable, are discussed in the following subsections.

4.1.1 Nathpa - Sholding Section

In this area, five hot springs are located, close to the bank of the river. There are three hot springs on the right bank, 400 m to 800 m downstream of the Nathpa dam axis and two hot springs on the left bank about 400 m and 500 m downstream of the dam axis. Temperatures recorded at these springs vary from mere lukewarm water to as high as 60^0C. During the excavation, temperature measurements indicated variations from warm water to 52^0C (Table 4.1). The temperature gradient estimated in this section was 0.06^0C/m. Temperature as high as 52^0C was encountered at Ch. 3631–3644 m (Table 4.1) in augen gneiss rocks with rock cover of 860 m.

4.1.2 Wadhal - Manglad Section

This is a major problematic zone in respect of geothermics. A hot spring present at Jeori (Ch. 18500 m) has a temperature of 46^0C at the surface. A number of drill holes in the area indicated temperature rise with depth. The maximum temperature gradient obtained was 0.12^0C/m against a normal gradient of 0.03^0C/m (GSI, 1988) in one of the drill holes, whereas in the others, it was in 0.05-0.09^0C/m range.

Seepage water in the initial 700 m stretch had a normal temperature. From Ch. 16860 m onwards, rocks encountered in the hot water zone were largely amphibolite and quartzite, with a few bands of biotite schist and quartz mica schist.

Table 4.1 The Tunnel Reaches Encountering High Temperature Groundwater (Source: The Project Authorities)

S.No.	Chainage (in m)	Temperature	S.No.	Chainage (in m)	Temperature
1.	1279-1356	Hot water dripping	14.	4278-4378	Warm water dripping
2.	1856-1860	Warm water	15.	4474-4478	Hot water dripping
3.	1962-1991	Hot water dripping	16.	4750-4760	Warm water flowing
4.	2155-2202	Hot water dripping	17.	7600-7608	Warm water dripping
5.	2387-2457	30 - 35°C	18.	7870-7886	Warm water flowing
6.	3022-3037	Warm water	19.	12523-12531	Lukewarm water
7.	3631-3644	50 - 52°C	20.	14646-14655	18 - 26°C
8.	3657-3678	50 - 52°C	21.	14655-14764	18 - 26°C
9.	3696-3712	44°C	22.	17040-17070	55°C
10.	3712-3723	51°C	23.	17837-17840	42°C
11.	3751-3818	Hot water dripping	24.	18531-18535	65°C
12.	4170-4178	Warm water dripping	25.	18774-18777	57.4°C
13.	4178-4208	36°C	26.	19200-19700	48.7°C at Ch.19559, 19576, 19586 m

During excavation of the tunnel in this section, temperature of water started rising gradually from Ch. 16971 m onwards and the discharge suddenly increased at Ch. 17068 m. At this location i.e. Ch. 17068, a deep cavity was formed on the right side of the tunnel which entered a 25 m thick band of chlorite schist. The discharge of around 100 lit/sec, with a water temperature of about 55^0C, caused collapse of the face and flooding of the tunnel for about 300 m. The work could be

3.4.6 Wadhal – Manglad Section

Rocks encountered in this section of the HRT are quartz mica schist, quartzite and amphibolite. The tunnel in this reach passes through a rock cover of 220 m to 920 m. The foliation joints in general are northwesterly to northeasterly dipping except for some portion in Wadhal downstream where they dip towards southeast. In addition, other joints dipping in easterly, southwesterly and northwesterly direction have also been encountered. Many of these joints were sheared and filled with gouge and crushed rock.

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In this reach, the HRT crosses Manglad stream, which is a perennial drainage having a deep and narrow gorge. The HRT just below the Manglad creek passes through quartz mica schist with a total cover of 9 m. Due to low rock cover below Manglad creek, the tunnel is steel lined for a length of 710 m from Ch. 22557 m to Ch. 23267 m.

3.4.8 Rattanpur – Jhakri Section

This section is represented mainly by quartz mica schist which becomes rich in mica in general, and quartz boudins at places, with the bands of amphibolite. The foliation joints strike sub-parallel to the tunnel alignment and dip at angle of 30^0 to 85^0 towards northeast as in the Manglad – Rattanpur section (section 3.4.7).

This section encounters Daj Khad and Daj shear zone where tunnelling conditions were very difficult. Due to the low rock cover below Daj creek, the tunnel is steel lined for a length of 375 m from Ch. 26390 m to Ch. 26675 m.

PROBLEMS FACED DURING EXCAVATION OF TUNNEL

During last forty years several kilometers of tunnels have been constructed in Himalaya. Most of these tunnels have been constructed in the Outer and the Lesser Himalaya in connection with hydropower projects. Presently, many tunnelling projects are under construction stage and a large number of these are under the planning and investigation stages. A list of major hydroelectric projects in Himalaya is shown in Fig. 1.1.

Most of the projects located in Himalaya have faced problems during excavation. The tunnelling hazards in Himalaya are of various types viz. excessive rock pressures, occurrences of clay pockets, seepage water and hazards associated with Himalayan tectonics like mountain-stresses, abnormal rock loads and geothermal gradients etc.

At Pandoh-Baggi tunnel of Beas Satluj Link Project, twisting and buckling of ribs was recorded after the rock cover exceeded about 1400 m. Similar phenomenon is observed in Giri tunnel, where twisting, buckling and shearing of ribs started at places exceeding rock cover of about 250 m. Loktak tunnel has faced hazards similar to those of Giri tunnel (Singh and Mahajan, 1982).

A very serious problem of excessive temperature (39^0 to 40^0C) was reported recently in the power tunnel of 3.2 m excavated diameter of Sanjay Vidyut Pariyojna. The project lies within the zone of the Lesser and the Higher Himalaya. Most of the hot springs are also located along this zone (Saini et al., 1989).

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Most of the hot springs in the project area, except the one at Jeori, are at river level. Since the springs at higher levels are cold, it seems that the hot ground area is below river level and the phenomenon is only manifested in the form of hot springs in area of major structural discontinuities, which act as routes for the discharge of hot water.

According to the geophysical surveys, with the exception of the Jeori Khad area, nowhere in the project area resistivity values are low, indicating the presence of geothermal zones. The resumed at the face only after two months of dewatering, removal of muck, bulk grouting of the cavity and other remedial measures (Singh et al., 1998).

As excavation advanced through quartzite and amphibolite, hot water continued to drip from tunnel crown and along the joints. In the tunnel reach from Ch. 18531 m to Ch. 18535 m, temperature of water recorded was 65^0C and the discharge 125 lit/min.

4.1.3 Remedial Measures Adopted

The following remedial measures were adopted:

- An additional 200 kW exhaust fan was installed at the inlet of the adit, with a capacity of 180 x 10^3 m^3/hour, which reduced the temperature at the tunnel face.
- Blocks of ice were placed near the tunnel face at a rate of 25 to 30 tonnes per day.
- Three new pipes with diameters of 30.5 cm and two with 35.5 cm replaced the original 25.4 cm pipe, to improve the pumping of hot water out of the tunnel.
- While full-face excavation continued from the Wadhal adit (Ch. 16061 m) downstream, from Ch. 17068 m onwards the excavation was reduced to top heading.
- The workforce at the face was replaced after periods of 2 to 4 hours.

4.2 HEAVY INFLOWS OF GROUNDWATER

The project area falls in the southern part of the Higher Himalayan Zone, which is characterized by a monsoon climate. Humid summers and cold dry winters are characteristics of the area.

In the project area, maximum rainfall occurs between mid June and mid September. During winter season the higher peaks receive snow, while low-lying areas receive moderate rains between mid December and the month of January. The area gets fairly high annual precipitation as rainfall and snow in higher reaches. The tunnel alignment crosses five major cross-drainages i.e. Sholding Khad (Ch. 6220 m), Wadhal Khad (Ch. 15400 m), Jeori Khad (Ch. 19050 m), Manglad Khad (Ch. 22800 m) and Daj Khad (Ch. 26580 m). The minimum discharge of cross-drainages is of the order of 2 cumec. All these drainages get their discharge from snowmelt as well as ground water seepages. There are about 57 to 60 cross-drainages and 50 to 70 cold water springs in the proximity of the NH-22 lying above the HRT grade. The fractured nature of rock mass and depth of weathering facilitates deep permeation of considerable part of such precipitation, thereby creating pockets of groundwater. During the tunnel excavation many groundwater pockets under high head have been encountered, particularly along major structural weak zones of long continuity. Flowing and dripping groundwater conditions have been observed in several tunnel reaches as indicated in Table 4.2. Heavy discharges up to 300 lit/min were observed in Ch. 3631-3678 m, Ch. 8253-8266 m and several other locations causing rock falls and cavity formation. At Ch. 17068 m, heavy discharge of hot water was responsible for the face collapse and flooding of the tunnel for about 300 m causing a delay of about two months. Also in the Daj shear zone from Ch. 26105 m to Ch. 26623 m, heavy ground water discharges were witnessed and its combination with the poor rock mass created a number of collapses in the tunnel. A detailed account of this Daj shear zone, which was tackled by the Drainage-Reinforcement-Excavation-Support-Solution (DRESS) methodology, has been presented in section 4.4.

Table 4.2 The Tunnel Reaches Encountering Heavy Groundwater Inflows (Source: The Project Authorities)

S.No.	Chainage (in m)	Remarks	S.No.	Chainage (in m)	Remarks
1.	50-53	shear zone	27.	7600-7608	warm water
2.	110-129		28.	7678-7687	
3.	267-269	shear zone	29.	7870-7886	
4.	681-687	shear zone	30.	8253-8266	250 lit/min
5.	958-966		31.	8266-8268	cavity formation
6.	966-987		32.	8268-8278	
7.	987-1015		33.	8742-8759	
8.	1272-1279		34.	9106-9111	shear zone
9.	1823-1838		35.	9435-9449	
10.	1856-1860	shear zone	36.	9449-9507	shear zone
11.	1962-1991		37.	9878-9882	
12.	2112-2137		38.	9987-10044	
13.	2387-2457		39.	10167-10177	
14.	3631-3678	300 lit/min; hot water	40.	14191-14200	shear zone
15.	3696-3712		41.	14646-14764	
16.	3751-3818	Hot water	42.	14968-15069	
17.	3818-3953	shear zone	43.	16330-16360	
18.	4278-4378	warm water	44.	16500-16585	
19.	4378-4474		45.	17040-17070	200 lit/min; hot water
20.	4750-4760	50 lit/min; warm water; shear zone	46.	18531-18535	125 lit/min; hot water
21.	6518-6523	shear zone	47.	22047-22098	
22.	6784-6820		48.	22121-22500	
23.	7387-7436		49.	26105-26190	shear zone
24.	7490-7511		50.	26288-26343	shear zone
25.	7530-7543	warm water	51.	26478-26545	shear zone
26.	7585-7600		52.	26608-26623	shear zone

4.3 TUNNELLING THROUGH ROCK COVERS MORE THAN 1000 m

4.3.1 Geology

The HRT from Ch. 10900 m to Ch. 12700 m lies under a rock cover of more than 1000 m with a maximum value of 1430 m at Ch. 11435 m. Rock types exposed in this reach of the HRT are gneiss, schistose gneiss and quartz mica schist with amphibolite, pegmatite and quartz intrusions. The foliation planes (primary joint set) are moderate to closely spaced (average spacing 75 to 300 mm), strike parallel to the tunnel axis from Ch. 11430 m to Ch. 12267 m and strike at angle of 28^0 to 53^0 to the tunnel axis for the rest of the portion. Foliations dip 50^0 to 70^0 in N10^0-30^0 E direction. Apart from the foliation planes, steep dipping transverse joints are commonly associated in the excavation, dipping northeasterly or southwesterly having continuities generally between 2 m and 12 m with average spacing of 200 to 900 mm. Apart from these joints, sheared, steep dipping transverse joints (thickness < 10 mm) are also observed which have a frequency of 20 to 25 m of tunnel length and contain clay or chloritic material and/or granular materials most of the time. Along with these minor discontinuities, three major shear zones at (i) Ch. 11430-11435 m, (ii) Ch. 11602-11615 m and (iii) Ch. 12523-12531 m, are also encountered in this tunnel reach.

4.3.2 Behaviour of Rocks Under High Stresses

Different rock masses have behaved differently during the tunnel excavation. The behaviour of gneiss and schistose gneiss, which are massive to less jointed, was observed to be brittle giving cracking sounds and show deformation by spalling. Quartz mica schist on the other hand may undergo some deformation before spalling and gives no sound on deformation. Generally jointed rocks have given few problems than brittle massive gneiss. Heavy spalling has been observed on the contact of these two different rock types because of the difference in their modulii of elasticity, with gneisses taking the major part of the load and hence are more prone to spalling. In the tunnel reaches where the rock mass is highly jointed, there is little or no stress related problems.

Most of the tunnel sections under consideration are dry. This is typical for tunnels in high stress regimes since joints tend to close. The major principal stress at these locations has been in sub-vertical direction which is indicated by the spalling being confined to the right arch of the tunnel because of the dip of the foliation towards the valley side or right side resulting in transfer of greater vertical load to the outer part of the mountain side.

The orientation of the foliation planes has played a very important role in deciding the locations and magnitudes of support to be provided. Strike of the foliations being almost oblique and parallel to the tunnel axis and their dip towards the valley side or right side of the tunnel has resulted in directing the maximum stresses to the right crown and walls of the tunnel and hence almost all of the spalling, open joint planes, rock falls etc. have taken place on the right crown and wall of the tunnel during the excavation of these reaches. Therefore, more emphasis was given on supporting the right crown and wall in comparison to the left crown and wall.

In gneiss and schistose gneiss, the rock mass showed signs of stressing even after 10 to 15 days or after about 30 m of tunnel advancement. Quartz mica schist showed signs of crumpling on the right arch after the tunnel advancement of 10 m only when the rock cover exceeded 1300 m, but with the lesser rock cover the crumpling effect was delayed for a longer period of time and with longer tunnel advancement (100 m or more). All the major shear zones encountered in this part of the HRT have been provided with rib support and later on back filled with M40 concrete. There has been no sign of movement / buckling of the ribs.

Signs of overstressing due to high rock covers, which were visible at the site, are summarized below (After discussion with the Project Authorities):

(a) Cracking sound

Cracking noises were heard between Ch. 11230 m and Ch. 11430 m where the rock cover exceeded 1300 m and the rocks being predominantly massive to less jointed gneiss and schistose gneiss.

(b) Mild rock burst

This phenomenon was observed between Ch. 11230 m and Ch. 11430 m in the massive gneiss and schistose gneiss rock. It was in the form of ejection of sharp edged thin slabs of rock from the tunnel periphery. Sizes of rock columns liberated were 50 to 250 mm in thickness and usually 0.05 to 1.0 m^2 in surface area.

(c) Rock falls

Majority of rock falls took place on the right crown and in the massive gneiss and schistose gneiss whenever one or two joint sets intersect foliation planes. These rock falls took place about 10 to 30 days after the excavation and ranging in size from 1 to 3 m^3. These rock falls were confined between Ch. 11230 m and Ch. 11430 m with the maximum number of rock falls recorded between Ch. 11300 m and Ch. 11400 m.

(d) Open foliation planes

This was the most common phenomenon observed on the right arch and wall of the tunnel where the foliation planes dip. Although rock bolts were installed immediately in every cycle of the excavation, there were signs of opening up of foliation planes on the right crown and wall only. This phenomenon was observed both for massive and jointed rock masses and was more pronounced between Ch. 11300 m and Ch. 12350 m. The openings range from a few mm to about 20 mm.

(e) Spalling

This was again a very common phenomenon observed during the excavation of the tunnel and was largely confined towards the right arch and wall. Openings and later detachment of rock slabs along the periphery of the tunnel took place. Depth of spalling varied from 200 to 2000 mm. The reach where the thickness of spalling was at its maximum was attributed to the presence of thin (10 to 15 cm) biotite schist bands/shears (thickness < 10 to 20 mm) in the rock mass, resulting in variable strengths.

(f) Rock bolt plate buckling / breaking away

This phenomenon was confined in reaches between Ch. 11300 m and Ch. 11700 m on the right arch only. An instance of rock bolt plates breaking away was observed at Ch. 11467-11483 m, where spalling had taken place in the gneiss rock.

4.3.3 Remedial Measures Adopted

In the reaches where heavy spalling took place i.e. on the right arch, the rock bolt density was increased along with application of 200 to 500 mm thick shotcrete to provide tunnel stability. No further spalling was observed after that.

In reaches where there was deformation in the rock bolt plates, 6 m long rock bolts were installed in juxtaposition with the 4 m long rock bolts, mainly on the right arch. These reaches also showed a stable nature with no further sign of a stress related problem. A 300 to 500 mm thick shotcrete was sprayed along with rock bolts, where there were openings (10 to 20 mm) in the foliation planes enhancing the stability of the rock mass.

4.4 TUNNELLING THROUGH FLOWING AND SQUEEZING CONDITIONS

The excavation of major portion of the tunnel has been carried out with heading and benching method. Exceptionally poor ground conditions were encountered in the reach between Rattanpur and VI Face (Fig. 1.2), and further advance of excavation was not possible with the available equipments. Mande and Vohra (1998) reported that there were serious distortions in the tunnel supporting system due to squeezing in this reach. Therefore, a method termed as DRESS methodology (covered in section 4.4.2) was used which allowed excavation of tunnel in 65 percent heading. The HRT in this reach passes through a major shear zone of 60 m width (Ch. 26288-26348 m) accompanied by fractured and shattered rock mass, small multiple shears and flowing ground. A total length of 388 m (Ch. 26133-26521 m) has been treated by the DRESS methodology.

4.4.1 Geology

It is the axial portion of anticlinal fold with a N-S axial trend, which runs along Daj Khad. The rocks in the shear zone encountered are quartz mica schist, sericite schist, biotite and amphibolite bands. It is mostly rich in sericite and biotite schist with multiple small shears filled with clay and squeezing in nature. The shear zone comprises of blocky and fractured rock with kaolinised and seriticised material, clayey gouge having very low strength and easily crushable with hands. The shearing of rock mass and subsequent weathering has resulted into distortion of the foliation. The heavy ingress of water has caused the shear zone material to flow.

4.4.2 The DRESS Methodology

The details of the excavation and support system are shown in Fig. 4.1. For complete description of the methodology, please refer to the account given by Sirkek et al. (1999). A brief discussion on the methodology is as follows.

As a first step, before opening of the face, the advance drainage is done all around and ahead of the face with drainage holes up to 24 m length with hydraulic drilling rig (forepoling machine) in alternate forepoling blocks. The main function of hydraulic drilling rig is to provide forepoling (casing) of steel pipes simultaneously with drilling ahead of the face before excavation of the face. In the second step, the stability of the face and ahead is improved by cement grouting or shotcreting and grouted anchor bars depending upon the rock condition. In the third step, the crown of the tunnel above springing level is supported with steel pipe forepoles over the first rib of the block. After drilling and installing of the forepoles, they have been made solid by placing the cement grout.

In the next step, after stabilizing the crown and ahead of tunnel face by steel pipe forepoles, drainage holes, shotcreting and grouting, the tunnel advance in one forepoling block of 12 m length is carried out up to 8.75 m length in a variable diameter of excavation from 11.65 m to 13.45 m before the next block of forepoling. In this 8.75 m length of the tunnel advance, a total number of 12 sets of ribs of ISMB 300 x 140 @ 750 mm c/c spacing are provided in a sequential advance of 0.75 m to 1.5 m depending upon the stand up time of rock strata. The excavated section is further supported with cement grouted rock bolts of 25 mm diameter 6 m long. This completes the heading excavation and supporting.

The benching excavation is done when the face is advanced by more than 50 m i.e. after complete excavation and supporting of at least 5 blocks of forepoling. Integration of rock reinforcement behind the face in heading is done with 25 mm diameter 6 m long cement grouted

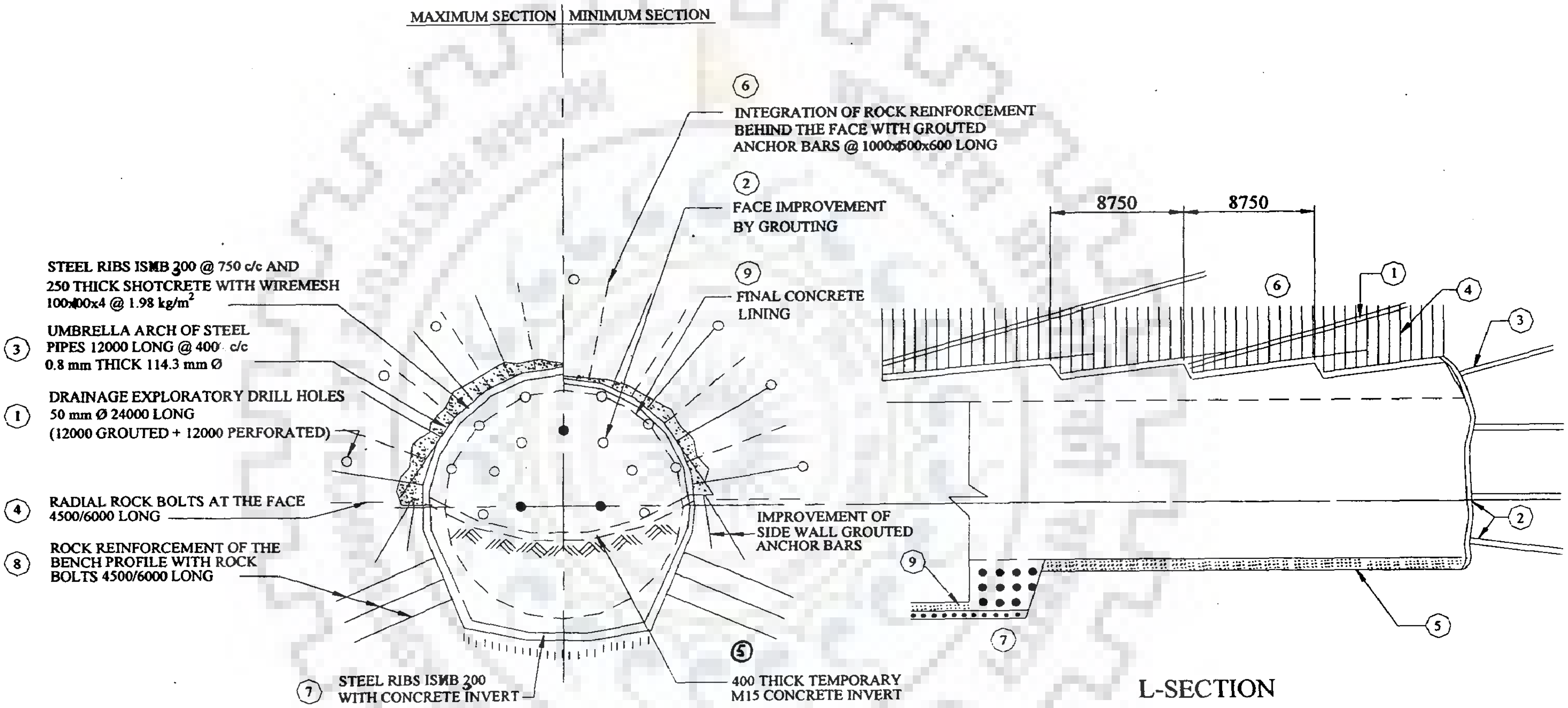


Fig. 4.1 Excavation and supporting system as per DRESS methodology (Class VI)

bars @ 1000 x 1500 mm spacing before benching excavation of the block. The benching is excavated in an advance of 3 m with hydraulic hammering technique. Thereafter, the side walls are protected with initial layer of 50 mm thick shotcrete, followed by wiremesh fixing, extension of heading ribs in the benching and providing steel ribs arch invert. The spaces between the rock surface and intrados of the ribs on wall sides are filled with shotcrete and the invert steel arch has been encased in 400 mm thick M20 concrete. The benching profile has been further supported with cement grouted rock bolts of 25 mm diameter 6 m long.

A maximum progress of 25 m in a month has been achieved in exceptionally poor rock condition with the DRESS methodology using the hydraulic drilling rig. The methodology is completely flexible and effective in responding to extremely heterogeneous rock mass with large rock loads and squeezing behaviour under high water head with risk of huge inflow in unstable excavation profile and has a high reliability with reference to time and cost estimates. Initial investment for hydraulic drilling rig for installation of long forepoles may be more, but taking into consideration all the factors such as progress of work, safety of workers, stability of the tunnel etc. in exceptionally poor rock mass conditions, the method is suitable, economical and appropriate.

ROCK MASS CLASSIFICATION

The term rock mass refers to any in-situ rock with all inherent geomechanical anisotropies (John, 1962). A rock mass is a network of rock blocks separated by discontinuities. It may be considered anisotropic and non-homogeneous material built-up of smaller and larger blocks of rock. As a great variety exists both in the composition of the rock and in its discontinuities, the rock mass is a material exhibiting a wider range of composition and mechanical properties. The design and construction of an underground excavation in rock mass becomes extremely complex due to the uncertainties involved in the determination of its geotechnical parameters.

Description of rock mass forms an initial step for an engineering assessment, which may be done from a geological and engineering point of view. The geological classification of rocks (lithology) is based on mineralogy fabric, degree of weathering or alteration, density or porosity and texture of rock. The name of rock provides an indication of its origin, mineralogical composition and texture, and from an engineering point of view, strength, hardness, seismic velocity and modulus of elasticity are important characteristics.

In the present study, different RMC systems such as RSR, RMR, Q and RMi have been applied besides use of N and RCR. In all 685 tunnel sections covering a total length of 22159 m have been considered. Out of 685 sections, there are 50 sections facing squeezing ground conditions and 69 shear zones encountered in the HRT. Total length of tunnel facing squeezing ground conditions is 1133 m with length varying from 3 m to 63 m whereas total length of shear zones is 943 m with length varying from 2 m to 63 m. Apart from this, the HRT from Ch. 10900 m to Ch. 12700 m covering a length of 800 m, lies under a rock cover of more than 1000 m with a maximum value of 1430 m at Ch. 11435 m. High rock covers pose stress related problem such as spalling, popping and rock burst conditions.

5.1 GEOMECHANICAL PROPERTIES OF ROCK TYPES

The stability of underground excavations depends on the inherent strength of rock, the presence of mechanical defects or structural discontinuities such as fractures, shear zones etc., the ground water condition and on the tectonic stresses. The creation of a cavity in rock mass disturbs the existing stress condition and a new stress pattern is created depending upon the shape of the cavity. It is, as such, essential to know the physical and engineering properties of the rock material as well as that of the rock mass. The data helps in evolving design of support system and the final lining of the underground work.

The engineering properties of rock in the HRT have been determined at several locations along the tunnel alignment and other project locations by in-situ tests aided by laboratory tests. In-situ and laboratory tests were conducted by Central Soil and Materials Research Station (CSMRS), New Delhi and the results are presented below.

5.1.1 In - Situ Tests

The tests carried out were: flat jack, plate loading, hydraulic fracturing and overcoring tests. The flat jack and plate loading tests were conducted as per ISRM (1979), the hydraulic fracturing as per Haimson (1984) and the overcoring tests as per Enever and Walton (1987). The results of the flat jack and plate loading tests are given in Table 5.1.

The results of the hydraulic fracturing tests conducted at desilting chambers are presented in Table 5.2.

Table 5.1 Results of Flat Jack and Plate Loading Tests (GSI, 1988)

Name of Test	Location	Number of Tests	Rock Type	Rock Cover	Test Results
Flat jack test	Drift at Nathpa dam site	Two tests, one in horizontal and one in vertical direction	Biotite gneiss with lenses of quartz	30 m	Modulus of deformation Vertical: 4.56 GPa Horizontal: 9.00 GPa
Plate loading test	Drift at Nathpa dam site	Three tests in vertical direction only	Biotite augen gneiss	Varying from 34 m to 45 m	Modulus of deformation Vertical: 2.54 GPa
Plate loading test	Drift at Jhakri	One vertical and one horizontal	Biotite gneiss	45 m	Modulus of deformation Vertical: 3.66 GPa Horizontal: 2.87 GPa

Table 5.2 Results of Hydrofracturing Tests at Desilting Chambers (CSMRS, 1994b)

Rock Cover (in m)	Minimum Stress (in MPa)	Maximum Stress (in MPa)	Vertical Stress (in MPa)
310	6.11	8.89	8.21
410	7.69	10.77	10.86
490	8.94	12.27	12.97

From Table 5.2, values of K in three tests conducted are computed as 1.08, 0.99 and 0.95 respectively giving an average value as 1.0.

The HRT is a high pressure tunnel which passes through reaches of low rock cover; one at Manglad and the other at Daj. Starting and end points of the liners have been fixed based on the results of hydrofracturing tests. Results of these tests are given in Tables 5.3 and 5.4 respectively.

The Manglad adit junction is located at Ch. 22537.05 m. In Table 5.3, in initial four locations of the tests, values of K computed are as 0.92, 1.09, 1.39 and 1.18 respectively giving an

Table 5.3 Results of Hydrofracturing Tests at Manglad (CSMRS, 1996b)

Location / Rock Cover	Average Minimum Stress (in MPa)	Average Maximum Stress (in MPa)	Vertical Stress (in MPa)
Ch. 22457 m (304 m)	4.97	7.24	7.90
Ch. 22487 m (275 m)	4.96	7.80	7.15
Ch. 22517 m (248 m)	5.90	8.94	6.44
Ch. 22547 m (236 m)	5.18	7.24	6.13
Ch. 23227 m (196 m)	5.00	8.40	5.09
Ch. 23267 m (224 m)	6.90	10.35	5.80

Table 5.4 Results of Hydrofracturing Tests at Daj (CSMRS, 1997a, b)

Location / Rock Cover	Average Minimum Stress (in MPa)	Average Maximum Stress (in MPa)	Vertical Stress (in MPa)
Ch. 25830 m (408 m)	7.32	12.86	10.60
Ch. 26738 m (225 m)	5.52	8.87	5.85
Ch. 26785 m (222 m)	5.11	9.03	5.77
Ch. 26818 m (220 m)	5.72	10.73	5.72
Ch. 26823 m (237 m)	3.91	6.71	6.16

average value approximately as 1.1 whereas in the last two locations, values of K computed are 1.65 and 1.78 giving an average value approximately as 1.7. However average of all the test locations is approximately 1.3, which has been considered to be applicable in the Rattanpur upstream and downstream directions.

The Rattanpur downstream tunnel and the VI Face upstream tunnel driving meet at Ch. 26274 m. Therefore all the test locations indicated in Table 5.4 except the first one i.e. Ch. 25830 m fall in the VI Face upstream direction. Values of K at these locations are computed as 1.21, 1.52, 1.56, 1.87 and 1.09 respectively giving an average value of approximately 1.45.

Overcoring tests were conducted for measurement of in-situ stresses in the powerhouse drift. The results of these tests are presented in Table 5.5.

Table 5.5 Results of Overcoring Tests at Powerhouse (CSMRS, 1994a)

Principal Stress	Magnitude (in MPa)	Bearing (in degree)	Dip (in degree)
σ_1	13.15	187	16
σ_2	8.00	311	63
σ_3	4.36	90	21

Value of K computed for the powerhouse area is 1.64.

From hydrofracturing tests conducted at desilting chambers, Manglad and Daj and overcoring tests at powerhouse location, it has been observed that values of K increase in the direction from Nathpa towards Jhakri. Value of K at the desilting chambers at Nathpa is 1.0, average K at Manglad upstream and downstream directions is 1.3, at Daj it is 1.45 whereas it increases to 1.64 at the powerhouse location. No stress measurement tests were conducted between Nathpa and Manglad, therefore value of K as 1.0 has been considered for the tunnel reaches between Nathpa and Wadhal. The summary of values of K considered along the tunnel alignment at various locations is presented in Table 5.6.

Table 5.6 Values of K along Tunnel Alignment

Location	Value of K
Nathpa u/s and d/s, Sholding u/s and d/s, Nigulsari u/s and d/s and Wadhal u/s and d/s	1.0
Manglad u/s	1.1
Manglad d/s	1.7
Rattanpur u/s and d/s	1.3
VI Face u/s	1.45

Longitudinal wave (P-wave) and transverse wave (S-wave) velocities were determined for different rock types by geophysical methods and the Poisson's ratios were evaluated. The results for the same are presented in Table 5.7.

Table 5.7 Dynamic Poisson's Ratio of Different Rock Types (GSI, 1988)

Rock Type	Poisson's Ratio
Augen gneiss	0.35
Biotite / augen gneiss	0.31
Biotite gneiss	0.36
Amplibolite	0.29
Biotite schist	0.24
Gneissose schist	0.32

5.1.2 Laboratory Tests

The following laboratory tests were conducted on core samples of NX sizes (CSMRS, 1996a, c, d, e):

(a) Index Properties: dry and saturated unit weights, specific gravity, water content at saturation and apparent porosity (tests conducted as per IS 13030 : 1991).

(b) Engineering Properties: UCS in saturated condition and shear strength parameters by triaxial tests (tests conducted as per IS 9143 : 1979 and IS 13047 : 1991 respectively).

Results of these tests are presented in Table 5.8.

5.2 COLLECTION OF CLASSIFICATION PARAMETERS

RMC provides an index of the rock mass behavior, which is then utilized for developing the empirical design in rock engineering, which has great potential and utility for recommending the permanent support systems for tunnels.

RMC has been done for the HRT using RSR, RMR, Q and RMi systems. Other methods of RMC such as N and RCR have also been applied in the present work.

5.2.1 Rock Structure Rating (RSR) - System

The concept provides a method of rating the quality of a rock structure with respect to its need of structural support during tunnelling operations. The RSR concept considered two general categories of factors influencing rock mass behavior in tunnelling; geological parameters and construction parameters (APPENDIX – III).

Table 5.8 Results of Laboratory Tests Conducted on Rock Cores

Location	Rock Type	Index Properties					Engineering Properties								
		Dry Unit Weight (kN/m^3) x10	Sat. Unit Weight (kN/m^3) x10	Grain Unit Weight (kN/m^3) x10	Water Content at Saturation (%)	Porosity apparent (%)	Uniaxial Compressive Strength (MPa)				Tensile Strength (MPa)	Triaxial Compressive Strength			
							No. of Samples	Minimum	Maximum	Average		No. of Reading	Cohesion (MPa)	Angle of Shearing Resistance (Deg.)	Hoek Constant, m
Nathpa	biotite gneiss with pegmatite band	2.598	2.610	2.650	0.51	1.297	7	18.97	53.13	33.0	2.2	9	4.0	50	7.55
	biotite gneiss	2.606	2.620	2.710	0.52	1.350	12	42.11	64.95	53.0	3.8	7	7.0	50	7.55
	pegmatite	2.583	2.595	2.650	0.54	1.385	5	35.00	60.00						
	gneissose schist	2.590	2.610	2.653	0.74	1.890	7	32.36	89.10	51.0					
	gneissose schist with pegmatite band	2.600	2.610	2.700	0.46	1.200	6	31.47	99.73	60.0	5.6	7	11.0	55	10.06
Sholding	quartz mica schist & its variations	2.653	2.666	2.716	0.53	1.390	6	13.30	26.70	18.0	3.8	10	7.1	37	4.02
	gneissose schist	2.905	2.919	2.956	0.48	1.440	6	64.20	129.50	85.0	4.4	10	12.0	65	20.34
	amphibolite	2.919	2.929	2.959	0.43	1.320	5	39.44	92.40	70.0	5.2	8	11.0	59	13.75
	biotite gneiss	2.679	2.690	2.708	0.44	1.170	4	41.30	57.10	50.0	4.9	6	8.8	45	5.83
Wadhal	quartz mica schist	2.820	2.832	2.863	0.42	1.330	4	10.70	57.80	31.7	4.6	7	9.0	30	3.00
	quartz mica schist with variations	2.760	2.772	2.833	0.49	1.310	4	12.80	92.80	42.4	0.7	9	1.3	58	12.16
	gneissose schist	2.580	2.590	2.625	0.42	1.080	5	26.10	50.70	34.0					
	quartzite	2.627	2.640	2.677	0.16	0.410	5	52.80	165.50	125.0					
Manglad	quartz mica schist	2.675	2.690	2.715	0.60	1.590	1			16.5	3.2		6.0	47	6.44
	quartz mica schist with biotite	2.740	2.750	2.770	0.53	1.450									
	quartz mica schist with muscovite	2.635	2.645	2.695	0.33	1.370									
	muscovite rich quartz mica schist	2.650	2.670	2.700	0.60	1.580	1			9.5	3.2		6.0	47	6.44
	quartz mica schist with muscovite & sericite	2.615	2.625	2.675	0.36	0.950									
	quartz mica schist with more % of quartz	2.610	2.620	2.690	0.52	1.340	5	9.10	27.00	17.8	3.2		6.0	47	6.44

5.2.1.1 Evaluation of parameters

(i) Rock type origin

Rocks encountered during the tunnel excavation are gneiss, biotite gneiss, amphibolite, biotite schist, gneissose schist, augen gneiss, chlorite schist, quartz mica schist, quartzite, quartz biotite schist and sericite schist. All these rock types fall under the category of metamorphic rocks.

(ii) Rock hardness

Rock hardness is the second parameter required along with the rock type origin to place the rock type encountered in appropriate category (Types 1 through 4, APPENDIX – IIIA). Wickham et al. (1972) gave qualitative description only for the hardness of rock such as hard, medium, soft and decomposed. In the present work, it has been considered appropriate to relate the rock hardness with its UCS.

Deere and Miller (1966) had proposed engineering classification of intact rock on the basis of its strength (Table 5.9). Accordingly, rocks have been categorized for classification using RSR according to their UCS values, and are given in Table 5.10.

Table 5.9 Engineering Classification of Intact Rock (Deere and Miller, 1966)

Class	Description	UCS (in MPa)
A	Very high strength	> 220
B	High strength	110 - 220
C	Medium strength	55 - 110
D	Low strength	27 - 55
E	Very low strength	< 27

Table 5.10 Proposed Classification of Intact Rock for RSR-System

Description	UCS (in MPa)
Hard	> 100
Medium	50 - 100
Soft	25 - 50
Decomposed	< 25

Thus, basic rock type is decided on the basis of rock type origin and its hardness.

(iii) Geological structure

It is difficult to recognize existing structure in terms of its degree of faulting and folding. Wickham et al. (1972) have not made it clear whether the terms (massive, slightly, moderately and intensely faulted or folded) are applied regionally or locally. However it seems judicious that these are applied locally. It is quite possible that particular tunnel reach is intensely faulted in an otherwise better regional geological structure and vice-versa. Massive word has been clubbed with degree of faulting and folding which is a phenomenon related to the tectonic stresses, whereas massive term is related with the degree of jointing. Thus, it may be assumed that these terms are also associated with the degree of jointing. In view of this, a brief summary is presented in Table 5.11 pertaining to geological structure encountered along the tunnel alignment.

Most striking feature of this table is that there is no tunnel reach in the category of massive geological structure. In all the reaches in Table 5.11, one or two degree higher category of faulting or folding has been considered, depending upon several factors such as encountering of shear zones, presence of shear joints etc.

Table 5.11 Rock Structures along Tunnel Alignment

Tunnel Reach	Degree of Faulting and Folding	Reasons / Comments
Ch. 0-949 m	Slight	-
Ch. 949-2250 m	Moderate to intense	Presence of shear zones, sheared joints, wedging etc.
Ch. 2250-3878 m	Slight	-
Ch. 3878-4760 m	Moderate	Axial region of an anticline; axis in NW-SE direction intersects tunnel at Ch. 4408-4413 m
Ch. 4820-5678 m	Slight	-
Ch. 5678-6578 m	Moderate	Sholding khad intersecting at Ch. 6220 m; presence of sheared joints
Ch. 6578-7481 m	Slight	-
Ch. 7481-7798 m	Moderate	Presence of local anticline in the vicinity (axis in NE-SW direction); crenulated foliations
Ch. 7798-9004 m	Slight	-
Ch. 9004-10659 m	Moderate	Presence of a number of anticlines and synclines; reversal of dips of foliations from SW to NE direction indicates the presence of an anticline
Ch. 10659-12629 m	Slight	-
Ch. 12629-13616 m	Moderate	Presence of a number of filled shear seams indicate greater degree of ground activity
Ch. 13619-14178 m	Slight	-
Ch. 14204-14512 m	Moderate	Presence of a number of filled shear seams indicate greater degree of ground activity
Ch. 14512-15069 m	Slight	-
Ch. 15287-16400 m	Moderate	Wadhal Khad intersecting at Ch. 15400 m; crenulated foliations; presence of sheared joints
Ch. 16400-17040 m	Slight	-
Ch. 17040-19700 m	Moderate	A local anticline (plunging 25^0 in NW direction) intersecting at Ch. 18362-18370 m near Jeori Khad (Ch. 19050 m); presence of hot springs
Ch. 19700-27346 m	Moderate to intense	Presence of shear zones; squeezing ground conditions; highly jointed rock mass

(iv) Spacing of joints

The spacing of adjacent discontinuities largely controls the size of individual blocks of intact rock. Several closely spaced sets tend to give conditions of low mass cohesion whereas those, which are widely spaced, are more likely to yield interlocking conditions. These effects depend upon the persistence of the individual discontinuities.

In exceptional cases a close spacing may change the mode of failure of a rock mass from transnational to circular or even to flow. With exceptionally close spacing the orientation is of little consequence as failure may occur through rotation or rolling of the small rock pieces.

Spacing of joints has been taken from three-dimensional logs and face logs prepared by the Project Authorities during the excavation.

(v) Joint orientation and direction of tunnel drive

This parameter reflects the significance of various discontinuity sets present in a rock mass. The main set usually controls the stability of the excavation. In case of tunnel, it will be the set whose strike is parallel to the tunnel axis. The amount of dip and relation between the dip direction of joints and the direction of drive are also considered to be other important factors.

Joint orientations have been taken from three-dimensional logs and face logs prepared by the Project Authorities during the excavation. Direction of tunnel drive in relation to the dip direction of joints has been decided from longitudinal tunnel axis and the dip direction of joints. From this, it can be derived if the direction of tunnel drive is with dip or against dip. This needs to be determined for the tunnelling condition only when the strike of joints is perpendicular to the longitudinal tunnel axis. In the present work, strike of joints is considered perpendicular to the tunnel axis when the intersection angle between the two is more than 30^0, otherwise it is considered parallel to the tunnel axis.

(vi) Joint condition

Wickham et al. (1972) gave only three categories for the joint condition: Good, Fair and Poor. In the subsequent RMC systems such as RMR and Q, condition of joints was investigated in greater details, specially in the Q-system. The condition of joints recorded during the geological mapping of the HRT by the Project Authorities was also based on the description of the Q-system (APPENDIX - V, section 4: Joint Alteration Number J_a). An attempt has been made to correlate joint condition based on the Q-system with that of the RSR-system (Table 5.12).

Table 5.12 Proposed Correlation for Joint Condition for Q and RSR Systems

Category of Joint Condition for Q-system	Category of Joint Condition for RSR-system
A and B	Good
C, D, E, F, G, H and J	Fair
K, L, M, N, O, P and R	Poor

(vii) Anticipated water inflow

One of the major problems of the underground excavation is the flow of water into the tunnel through joints, thus inducing rock instability by eroding soft infilling materials.

The groundwater inflows were rated by the Project Authorities following the RMR-system (APPENDIX - IVA). According to this system, there are five categories for the groundwater conditions i.e. completely dry, damp, wet, dripping and flowing. On the other hand, Wickham et al. (1972) considered only four categories i.e. none, slight, moderate and heavy water inflows. Since the groundwater inflows were rated following RMR-system as stated earlier, it becomes necessary to correlate these two systems with regard to groundwater conditions. The proposed correlation is presented in Table 5.13.

Table 5.13: Proposed Correlation for Groundwater Condition for RMR and RSR Systems

Category of Groundwater for RMR-system	Category of Groundwater for RSR-system
Completely dry	None
Damp	Slight
Wet	Moderate
Dripping	Heavy
Flowing	Heavy

5.2.2 Rock Mass Rating (RMR) – System

In the present work, classification has been done following Bieniawski's (1989) approach. The six parameters used to classify a rock mass are: (i) UCS of intact rock material, (ii) RQD, (iii) spacing of joints, (iv) condition of joints, (v) groundwater condition and (vi) orientation of joints.

5.2.2.1 Evaluation of parameters

(i) UCS of intact rock material

According to Bieniawski (1988), this is a necessary parameter because the strength of intact rock material constitutes the upper strength of the rock mass. Results of laboratory UCS tests on the cores of rocks have been presented in Table 5.8.

(ii) Rock Quality Designation (RQD)

Although RQD does not include the influence of discontinuity orientation or joint condition, it is a quantitative index widely used on tunnelling projects (Bieniawski, 1988), thus enabling comparison of rock behavior in varied tunnelling situations.

RQD is probably the most common method for characterizing the block size, the degree of jointing, or the density of joints (Palmstrom, 2000). It may be due to the fact that it is easy to observe or measure. RQD values at all the tunnel reaches have been collected which were measured and recorded during the excavation of the tunnel by the Project Authorities.

(iii) Condition of joints or discontinuities

This parameter includes roughness, continuity and separation of the discontinuity surfaces along with weathering of wall rock and infilling material. The RMR-system describes condition of discontinuity in five categories (APPENDIX - IVA). Since condition of discontinuities recorded during the geological mapping by the Project Authorities was as per the description contained in the Q-system, it is necessary to correlate these two classification systems with regard to the condition of discontinuities. The proposed correlation has been presented in Table 5.14.

Table 5.14 Proposed Correlation for Joint Condition for Q and RMR Systems

Category of Joint Condition for Q-system (J_a)	Category of Joint Condition for RMR-system
A and B	Very rough surfaces, Not continuous, No separation, Unweathered wall rock
C	Slightly rough surfaces, Separation < 1 mm, Slightly weathered wall
D and E	Slightly rough surfaces, Separation < 1 mm, Highly weathered wall
F, G, H and J	Slickensided surfaces or Gouge < 5 mm thick or Separation 1 - 5 mm, Continuous
K, L, M, N, O, P and R	Soft gouge > 5 mm thick or Separation > 5 mm, Continuous

(iv) Groundwater condition

The groundwater inflows measured during the tunnel excavation by the Project Authorities may be easily used for RMR-system to estimate the numerical rating for the groundwater condition.

(v) Orientation of joints or discontinuities

According to APPENDIX - IVB, adjustment rating for evaluation of final RMR-value is done on the basis of qualitative terminology like very favorable, unfavorable etc. associated with the strike and dip orientations of discontinuities. To help decide this issue APPENDIX - IVE is referred, which is based on studies by Wickham et al. (1972).

5.2.3 Rock Mass Quality (Q) - System

The Q-system is based on a numerical assessment of the rock mass quality using six different parameters: (a) RQD, (b) number of joint sets (c) roughness of critically oriented joint set, (d) degree of alteration or filing along critically oriented joint set, (e) water inflow and (f) stress condition. These six parameters are grouped into three quotients to give the overall rock mass quality Q as follows:

$$Q = \frac{RQD}{J_n} \cdot \frac{J_r}{J_a} \cdot \frac{J_w}{SRF} \tag{5.1}$$

5.2.3.1 Evaluation of parameters

(i) Number of joint set (J_n)

Both the mechanical behaviour and the appearance of rock mass will be dominated by the number of sets of discontinuities that intersect one another. The mechanical behaviour is specially affected since the number of sets determines the extent to which the rock mass can deform without involving failure of the intact rock. The appearance of the rock mass is affected since the number of sets determines the degree of overbreak that tends to occur with excavation by blasting.

According to Barton et al. (1974), the parameter J_n representing the number of joint sets will often be affected by foliation, schistocity, slatey cleavage or bedding etc. If strongly developed, these parallel joints should be counted as a complete joint set. However, if there are few joints visible, or only occasional breaks due to these features, then these are counted as random joints when evaluating J_n.

Number of joint sets at all the tunnel reaches have been collected from the Project Authorities, which they had estimated and recorded during the excavation of the tunnel.

(ii) Joint roughness (J_r) and alteration (J_a)

The wall roughness of a discontinuity is a potentially important component of its shear strength, specially in the case of undisplaced and interlocked features (e g. unfilled joints). The importance of wall roughness declines as aperture or filling thickness increases. In general terms the roughness of discontinuity walls can be characterized by waviness and unevenness.

Joint alteration or filling is the term for rock material separating the adjacent rock walls of discontinuities, e.g. calcite, chlorite, clay, silt, fault gouge, breccia etc.

According to Barton et al. (1974), the parameters J_r and J_a should be relevant to the weakest significant joint set or clay filled discontinuity in a given zone. However, if the joint set or discontinuity with the minimum value of $\left(\frac{J_r}{J_a}\right)$ is favorably oriented for stability, then a second, less favorably oriented joint set or discontinuity should be used when evaluating Q from Eq. 5.1.

Joint roughness and alteration measured during the tunnel excavation by the Project Authorities can be easily used for the Q-system (APPENDIX – V, sections 3 and 4) to estimate the numerical rating for both J_r and J_a.

(iii) Joint water reduction factor (J_w)

As mentioned in section 5.2.1.1 (vii), the groundwater inflows were rated by the Project Authorities following RMR-system. On the other hand, Barton et al. (1974) have considered six categories in this regard (APPENDIX - V, section 5). Therefore, a correlation has been proposed for these two classification systems for evaluating groundwater conditions, which is presented in Table 5.15.

Table 5.15 Proposed Correlation for Groundwater Condition for RMR and Q Systems

Category of Groundwater for RMR-system	Category of Groundwater for Q-system
Completely dry	A
Damp	A
Wet	B
Dripping	C
Flowing	D, E or F

(iv) Stress Reduction Factor (SRF)

According to Barton et al. (1974), when a rock mass contains clay, the factor SRF appropriate to loosening loads should be evaluated (APPENDIX - V, section 6a). In such cases the strength of the intact rock is of little interest. However, when jointing is minimal and clay is completely absent, the strength of intact rock may become the weakest link and the stability will then depend on the ratio of rock stress and rock strength (APPENDIX - V, section 6b).

Barton et al. (1974) further suggested that for evaluation of compressive and tensile strength of the intact rock, the test sample should be saturated if this condition is appropriate to present or future in-situ conditions and a conservative estimate of strength should be made for those rocks that deteriorate when exposed to moist or saturated conditions.

Kirsten (1983) presented a formula for the calculation of the SRF from the rock loads and the UCS of the rock material:

$$SRF = 0.244\left(\frac{\sigma_1}{\sigma_3}\right)^{0.346}\left(\frac{H}{q_c}\right)^{1.322} + 0.176\left(\frac{q_c}{H}\right)^{1.413} \tag{5.2}$$

In the present thesis, decisions were taken according to the nature of excavated tunnel reaches regarding selection of SRF from APPENDIX - V, section 6.

5.2.4 Rock Mass index (RMi) – System

Palmstrom (1995a, b, 1996a) mentioned that Hoek and Brown (1980b), and Bieniawski (1984) have indicated the need for a strength characterization of rock masses. Therefore, Palmstrom (1995a, b, c, d, 1996a) has developed RMi to characterize the strength of the rock mass for construction purposes. RMi considers only intrinsic parameters of the rock mass viz. compressive strength of intact rock, block volume and joint characteristics such as roughness, alteration and size. RMi is expressed as follows:

<u>Jointed rock:</u>

$$RMi = q_c . JP \tag{5.3}$$

JP can be found from Eq. 5.4:

$$JP = 0.2\sqrt{jC}.(V_b)^D \tag{5.4}$$

where, V_b is in m^3 and

$$D = 0.37\ jC^{-0.2} \tag{5.5}$$

jC is correlated with jL, jR and jA as follows:

$jC = jL\ (jR/jA)$ (Ratings are shown in APPENDIX - VIA) (5.6)

Massive rock:

$$RMi = q_c\, f_\sigma \quad (5.7)$$

$$\text{where, } f_\sigma = \left(\frac{0.05}{D_b}\right)^{0.2} \quad (5.8)$$

where, D_b is in m and

$$D_b = \sqrt[3]{V_b} \quad \text{m} \quad (5.9)$$

where, V_b is in m^3.

When $JP < f_\sigma$, Eq. 5.3 is applied.

5.2.4.1 RMi parameters

Hoek et al. (1992) are of the opinion that the strength characteristics for jointed rock masses are controlled by the block shape and size, and their surface characteristics determined by the intersecting joints. This does not imply that the properties of the intact rock material should be disregarded in the characterization.

5.2.4.2 Evaluation of parameters

(i) Block volume (V_b)

The block volume is intimately related to the intensity or degree of jointing. Each one of such blocks is more or less completely separated from other by various types of discontinuities. The greater the block size the smaller will be the number of joints penetrating the rock mass. Hence, there is an inverse relationship between the block volume and the number of joints.

Palmstrom (1995a, d) has given following correlation between block volume (V_b) and volumetric joint count (J_v):

$$V_b = \frac{\beta}{J_v^3} \cdot \frac{1}{\sin\gamma_1 . \sin\gamma_2 . \sin\gamma_3} \tag{5.10}$$

where, β is block shape factor and γ_1, γ_2 and γ_3 are the angles between the joint sets.

In the present work, γ_1, γ_2 and γ_3 have been kept as 90^0. So, Eq. 5.10 simplifies to

$$V_b = \frac{\beta}{J_v^3} \tag{5.11}$$

According to Palmstrom (2000*b*), it is difficult to define the β-value for the various types of blocks such as flat, long, very flat, very long etc., and it has not been intended that the block shape factor should be measured accurately. β-values used in the present work are indicated in Table 5.16 as suggested by Palmstrom (1996c).

Table 5.16 Block Shape Factor (β) Recommended by Palmstrom (1996c)

Number of Joint Set	Block Shape Factor, β
One	180
One + random	34
Two	100
Two + random	36
Three or more	27

Often, it is not possible to observe the whole individual block on an outcrop or on the surface of an underground opening, specially where less than three joint sets occur. Random joints or cracks formed during the excavation process often result in defined blocks. In such cases a spacing of random joints say 5 to 10 times the spacing of the main set can be used to estimate block volume (Palmstrom, 2000a).

Palmstrom (1995a, d) has derived following equation to compute β:

$$\beta = \frac{(\alpha_2 + \alpha_2\alpha_3 + \alpha_3)^3}{(\alpha_2.\alpha_3)^2} \tag{5.12}$$

where, $\alpha_2 = S_2/S_1$ and $\alpha_3 = S_3 / S_1$, provided $S_3 > S_2 > S_1$ and S_1, S_2, S_3 etc. are the spacing in mm between the individual joints in each set.

In the present work, J_v considered for different number of joint sets is as follows:

(a) One Joint Set

From Eq. 5.12, for β = 180, spacing of other two imaginary joint sets comes out as 10 times the spacing of main joint set. J_v is computed as follows:

$$J_v = \frac{1000}{S_1} \tag{5.13}$$

(b) Two Joint Set

From Eq. 5.12, for β = 100, spacing of other one imaginary joint sets comes out as 10 times the spacing of main joint set. J_v is computed as follows:

$$J_v = 1000\left(\frac{1}{S_1} + \frac{1}{S_2}\right) \tag{5.14}$$

(c) Two Joint Set + random

In the absence of spacing of random joints, from Eq. 5.12 for β = 36, a spacing of 2.6 times the spacing of the main set is arrived at. Hence,

$$J_v = 1000\left(\frac{1}{S_1} + \frac{1}{S_2} + \frac{1}{2.6S_1}\right) \tag{5.15}$$

(d) Three or More Joint Sets

$$J_v = 1000\left(\frac{1}{S_1} + \frac{1}{S_2} + \frac{1}{S_3} + \ldots\right) \tag{5.16}$$

(ii) Joint roughness factor (jR)

The RMi-system describes joint roughness in greater details as compared to the Q-system. However, jR in the RMi-system and J_r in the Q-system are broadly similar. Large-scale waviness in the RMi-system has been grouped in five categories whereas the Q-system considers only three categories. Similarly, small-scale unevenness in the RMi-system has been grouped in four categories whereas the Q-system considers only three categories (APPENDICES - V and VI). Since roughness of joints recorded by the Project Authorities was as per the description in the Q-system, it is necessary to correlate the two classification systems with regard to the roughness of joints (Table 5.17).

Table 5.17 Proposed Correlation for Joint Roughness for Q and RMi Systems

Category of Joint Roughness for Q-system (J_r)	Category of Joint Roughness for RMi-system (jR)
B	Undulating very rough and rough
C	Undulating smooth
D	Undulating slickensided
E	Planar very rough and rough
F	Planar smooth
G	Planar slickensided
H and I	Filled joints

(iii) Joint alterations factor (jA)

Descriptions in both the RMi and the Q systems are same regarding the joint alteration. In the Q-system, there are sixteen categories for the joint alteration whereas, 14 categories have been considered in the RMi-system (APPENDICES - V and VI). Since alteration of joints recorded during the geological mapping by the Project Authorities was as per the description contained in the Q-system, it was necessary to correlate these two classification systems with regard to the alteration of joints. The proposed correlation has been presented in Table 5.18.

Table 5.18 Proposed Correlation for Joint Alteration for Q and RMi Systems

Category of Joint Alteration for Q-system (J_a)	Category of Joint Alteration for RMi-system (jA)
A	Healed joints, filling of quartz etc
B	Fresh joint walls, no coating or filling, except from staining
C	Altered joint walls, one or two grade higher alteration than the rock
D	Coating / thin filling of sand, silt, calcite etc (non-softening)
E	Coating / thin filling of clay, chlorite, talc etc.
F	Thin filling (< 5 mm) of sand, silt, calcite etc (non-softening)
G	Thin filling (< 5 mm) of hard cohesive materials
H	Thin filling (< 5 mm) of soft cohesive materials
J	Thin filling (< 5 mm) of swelling clay materials
N	Thick filling of sand, silt, calcite etc (non-softening)
K and O	Thick filling of hard cohesive materials
L and P	Thick filling of soft cohesive materials
M and R	Thick filling of swelling clay materials

(iv) Joint size factor (jL)

Continuity or persistence implies the areal extent or size of a discontinuity within a plane. It can be quantified by observing the discontinuity trace length on the surface of exposures. It is one of the most important rock mass parameters. The discontinuities of one particular set will often be more continuous than those of the other sets. Minor sets will therefore, tend to terminate against the primary features, or they may terminate in solid rock.

Continuity or size of joints, in each joint set encountered, have been collected from the three-dimensional logs and face logs prepared by the Project Authorities.

5.3 COMPILATION OF CLASSIFICATION PARAMETERS AND OTHER DATA

The total tunnel sections, 685 in numbers and covering a length of about 22.2 km out of a total length of 27.4 km have been compiled for RMC using RSR, RMR, Q and RMi systems. Typical sections have been indicated in Table 5.19. The Table includes: location, longitudinal tunnel axis, tunnel diameter, K, chainage, overburden, shear zone details, geological structure, rock type, UCS, RQD, number of joint sets, joint properties, ground water and rock stress. A brief geological description is also given in the table. Some abbreviations used in the table are described in Table 5.20.

5.4 ROMAC: A COMPUTER PROGRAM FOR ROCK MASS CLASSIFICATION

An interactive computer program **ROMAC** has been developed in C++ language for RMC using RSR, RMR, Q and RMi systems. The flow charts for **ROMAC** have been given in APPENDIX - VIIA. **ROMAC** has been utilised to compute RSR, RMR, Q and RMi values, beside other related details. Typical results of **ROMAC** are presented in APPENDIX – VIIB in which

Table 5.19 Rock Mass Classification Data

Location : Nathpa u/s									Longitudinal Tunnel Axis : 073°			Tunnel Dia. : 10.15 m			K = 1.0			
Chainage (in m)	Over-burden (in m)	Shear Zone		Geologic Structure (faulted or folded)	Rock Type	UCS (MPa)	RQD (%)	No. of Joint Sets	Joint Properties							Ground Water	Rock Stress	
		Width (in m)	Dip / Strike (in deg.)						Dip / Strike (in deg.)	Drive	Spacing (in mm)	Continuity (in m)	Opening (in mm)	Roughness	Alteration		Class	SRF
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
0-50	680			moderately	biotite gneiss	53	76	3	42/110	W	500	>12	0-2	US	slight	dry	b	
	Biotite gneiss, biotite schist band. Foliation planes dip towards left crown & wall and are prone to slabbing on that side.								82/030	A	1000	5-12	0-2	PR	slight			
									74/295	A	1500	5-12	0-2	PR	unalt			
50-53	700	1	65/150	moderately	biotite gneiss	53	70	2.5	28/115	W	500	>12	0-2	USI	ss	dripping	a	2.5
	A shear with minor clay gouge & rock flour.								75/320	A	800	5-12	0-2	PS	slight			
53-110	720			slightly	biotite gneiss	53	75	2.5	31/110	W	600	>12	0-2	US	slight	damp	b	
	Biotite gneiss, biotite schist band. Foliation planes gently dip towards left crown & wall and are prone to slabbing on that side.								70/330	A	1000	5-12	0-2	PR	slight			
110-229	720			intensely	biotite gneiss	53	62	3	35/120	W	150	>12	2-5	PS	RF	dripping	b	
	Biotite gneiss, biotite schist band. Highly jointed rock mass, 3 joint sets intersecting each other. Foliation planes gently dip towards left crown & wall and are prone to slabbing on that side.								73/350	A	50	5-12	2-5	PS	RF			
									62/235	N	100	5-12	2-5	PS	RF			
229-267	720			slightly	biotite gneiss	53	88	2.5	30/100	N	400	>12	0-2	US	slight	damp	b	
	Foliation planes gently dip towards left crown & wall and are prone to slabbing on that side.								85/280	N	1100	5-12	0-2	PR	slight			
267-269	720	0.5	75/160	moderately	biotite gneiss	53	75	2.5	28/100	N	500	>12	0-2	USI	ss	dripping	a	2.5
	A shear (20-50cm thick) with minor clay gouge & rock flour.								90/295	A	700	5-12	0-2	PR	slight			
269-473	720			slightly	biotite gneiss	53	78	2.5	28/105	W	600	>12	0-2	US	slight	damp	b	
	Foliation planes gently dip towards left crown & wall and are prone to slabbing on that side.								80/330	A	800	5-12	0-2	PR	slight			
473-475	670			moderately	gneiss	53	70	2.5	25/070	N	500	>12	0-2	USI	ss	damp	b	
	Low dipping foliations at the left crown & wall resulting planar failure and intersection of which with transverse joints forming wedges at the crown. A shear with >10cm filling did not pose any problem.								30/070	N	1500	5-12	0-2	PS	slight			
475-681	680			slightly	gneiss	53	75	2.5	35/060	N	300	>12	0-2	US	slight	damp	b	
	Biotite gneiss with amphibolite, pegmatite & biotite schist. Joints are usually across the foliation with sub-vertical dips & form wedges at the left crown and wall. Foliation planes prominent on left arch.								45/245	N	800	5-12	0-2	PS	slight			

Table 5.20 Abbreviations used in Table 5.19 for Rock Mass Classification Data

Parameter	Abbreviation	Meaning
Drive	W A --	With dip Against dip Joint strike is parallel to the tunnel axis
Rock type	QMS GS	Quartz mica schist Gneissose schist
Joint roughness	UR US USl PR PS PSl	Undulating rough Undulating smooth Undulating slickenisided Planar rough Planar smooth Planar slickensided
Joint alteration	RF CG SS	Rock flour Clay gouge Sandy, silty coating / filling

- the RSR-system contains values of RSR, RR and roof pressure beside other details.
- the RMR-system contains values of basic RMR, RMR, Hoek RMR and RCR beside other details. Hoek RMR has been defined in the present work as basic RMR for completely dry groundwater condition.
- the Q-system contains Q-values for roof and walls, values of RQD/J_n, J_r/J_a and J_w/SRF and value of N beside other details.
- the RMi-system contains values of RMi for roof and walls beside other details.

Classification values for typical sections out of a total 685 sections have been presented in Table 5.21 beside other details. The minimum and maximum values obtained following all the classification systems in both the non-squeezing and squeezing ground conditions and their locations have been presented in Table 5.22.

Table 5.21 Classification Values and Roof Pressures Estimated from Classification Systems

S. No.	Chainage (in m)	Value						Roof Pressure (in MPa)					Squeezing Condition	Shear Zone
		RSR	RMR	RCR	Q	N	RMi	RSR	RMR	Q	N	RCR		
1	0-50	67	75	72	0.442	6.333	7.25	0.065	0.079	0.177	0.111	0.033		
2	966-987		42	40	0.111	0.556	0.9		0.182	0.477	0.831	0.252	Mild	SZ
3	1375-1587	56	61	62	0.741	6.667	7.99	0.135	0.123	0.12	0.106	0.044		
4	2057-2112	39	54	57	0.833	7.5	2.42	0.288	0.145	0.116	0.103	0.053		
5	2155-2202		25	23	0.027	0.275	0.03		0.236	0.756	2.007	0.895	Moderate	SZ
6	3631-3644	38	51	52	0.044	0.667	0.52	0.299	0.154	0.373	0.284	0.066		
7	6208-6338	59	60	62	21.25	10.625	15.5	0.114	0.126	0.049	0.076	0.038		
8	9687-9697	49	59	55	0.156	3.125	0.83	0.190	0.129	0.246	0.149	0.056		
9	11435-11446	47	55	61	0.583	11.667	2.69	0.208	0.142	0.098	0.094	0.052		
10	11634-11643	53	63	64	0.438	8.75	1.11	0.158	0.116	0.143	0.105	0.046		
11	11932-12044	63	62	68	0.417	8.333	3.11	0.088	0.120	0.109	0.106	0.040		
12	12044-12070	47	56	62	0.583	11.667	1.79	0.208	0.138	0.098	0.091	0.049		
13	12087-12223	47	55	61	0.419	8.375	2.04	0.208	0.142	0.145	0.106	0.050		
14	12359-12428	59	60	61	0.469	9.375	2.43	0.114	0.101	0.14	0.099	0.045		
15	12544-12608	53	60	61	0.438	8.75	1.63	0.158	0.110	0.143	0.102	0.048		
16	12629-12668	58	66	64	0.469	9.375	2.03	0.121	0.107	0.14	0.098	0.045		
17	16036-16046	59	65	61	0.586	5.278	2.86	0.114	0.110	0.159	0.110	0.042		
18	16840-16900	67	80	73	0.494	4.444	5.71	0.065	0.063	0.252	0.131	0.033		
19	16900-17000	58	58	56	0.463	4.167	3.24	0.121	0.132	0.258	0.136	0.056		
20	17040-17070	50	58	49	1.344	0.672	7.88	0.181	0.132	0.181	0.281	0.073		
21	18387-18424	47	67	60	0.556	5	1.87	0.208	0.104	0.162	0.121	0.047		
22	18535-18687	45	64	57	11.11	5.556	1.17	0.226	0.113	0.09	0.115	0.051		
23	19015-19054	39	56	54	0.494	4.444	0.47	0.288	0.138	0.252	0.126	0.057		
24	19054-19103	42	55	55	0.556	5	0.64	0.256	0.142	0.243	0.120	0.055		
25	19189-19700	47	63	61	0.679	9.167	1.32	0.208	0.116	0.227	0.091	0.045		
26	21230-21240		31	35	0.056	0.556	0.11		0.217	0.519	1.085	0.299	Moderate	
27	21240-21247		36	40	0.167	0.833	0.35		0.201	0.241	0.801	0.306	Mild	
28	22537-22648	36	52	50	0.122	2.444	0.13	0.323	0.151	0.4	0.138	0.055		
29	22795-22848	31	49	52	0.403	1.008	0.02	0.389	0.160	0.27	0.180	0.045		
30	23627-23785	36	48	49	0.25	3.75	0.68	0.323	0.164	0.172	0.134	0.068		
31	23894-23901		49	52	0.25	1.25	0.25		0.160	0.258	0.428	0.172	Mild	
32	25043-25080	40	41	44	0.75	15	1.6	0.276	0.186	0.12	0.072	0.086		
33	25951-25980	30	40	45	0.5	1.25	0.07	0.404	0.189	0.205	0.202	0.077		
34	26108-26112		14	12	0.033	0.167	0.04		0.271	0.458	0.886	4.393	Mild	SZ
35	26120-26126		23	23	0.003	0.052	0.001		0.242	1.651	3.481	0.293	High	SZ
36	26361-26398		15	18	0.005	0.055	0.001		0.267	1.438	1.694	0.420	Moderate	SZ
37	26415-26478		15	18	0.037	0.183	0.001		0.267	0.769	0.494	0.530	Mild	SZ
38	26478-26506		12	15	0.006	0.063	0.001		0.277	1.378	1.081	0.601	Moderate	SZ
39	26608-26620		15	15	0.001	0.019	0.001		0.267	2.952	5.578	0.558	High	SZ
40	26946-27346	48	61	61	0.469	9.375	0.68	0.198	0.123	0.14	0.081	0.040		

Table 5.22 Range of Classification Values for Classification Systems

S. No.	System	Category	All		Non-Squeezing		Squeezing	
		Ranges	Minimum	Maximum	Minimum	Maximum	Minimum	Maximum
1.	RSR	Value	25	76	25	76	-	-
		Chainage (in m)	13418-13424	2736-2739	13418-13424	2736-2739	-	-
2.	RMR	Value	8	81	17	81	8	51
		Chainage (in m)	26288-26343	16360-16400	13418, 6518	16360-16400	26288-26343	24574-24582
3.	Q	Value	0.001	58.08	0.017	58.08	0.001	0.667
		Chainage (in m)	26288-26343	16090-16110	6518-6523	16090-16110	26288-26343	24582-24595
4.	RMi	Value	0.001	42.5	0.01	42.5	0.001	0.9
		Chainage (in m)	26288-26343	4474-4478	14191-14200	4474-4478	26288-26343	966-987
5.	N	Value	0.013	90	0.042	90	0.013	3.333
		Chainage (in m)	26288-26343	16360-16400	6518-6523	16360-16400	26288-26343	24582-24595
6.	RCR	Value	11	77	15	77	11	54
		Chainage (in m)	26288-26343	16360-16400	6518-6523	16360-16400	26288-26343	24574-24582

The Project Authorities have categorized the support systems installed at the site as Class I, II, III, IV, V, VI and VI+ (APPENDIX – VIII). The total length of the tunnel covered is 20442 m having 648 sections where data is available. Table 5.23 indicates number and length of tunnel sections in each class of supports installed by the Project Authorities.

Based on the classification values obtained by different classification systems in all the tunnel sections, support types and/or ground conditions as obtained by these systems are shown in Figs. 5.1 through 5.4. Figure 5.5 shows tunnel length in different classes of supports installed by the Project Authorities.

Table 5.23 Number and Length of Tunnel Sections in Classes of Supports Installed by the Project Authorities

Project Class	Number of Sections	Length (in m)
II	122	3226
III	297	11065
IV	125	3160
V	75	2364
VI	17	238
VI+	12	389

Data for Class I supports is not available.

5.5 CORRELATIONS BETWEEN CLASSIFICATION SYSTEMS

A total 14 correlations between classification systems (RSR, RMR, Q, RMi, N and RCR) have been developed as shown in Figs. 5.6 through 5.19. A confidence limit of 90 percent for these correlations has also been indicated in these figures. All these correlations have been compiled in Table 5.24 also for concise presentation.

Due to new values proposed for SRF for moderately jointed rocks in the Q-system, Q-values obtained earlier have been changed. Therefore, revised correlations involving Q-values have been presented in Figs. 5.20 through 5.23 and a comparison with previous correlations (with un-revised SRF) and one presented by earlier researchers is indicated in Table 5.25. It may be seen that the coefficients of correlations have improved significantly.

The observed correlation between N and RCR is quite close to that of Goel (1995). The correlations between RSR and RMR or Q are also close to those found by Rutledge and Preston (1978).

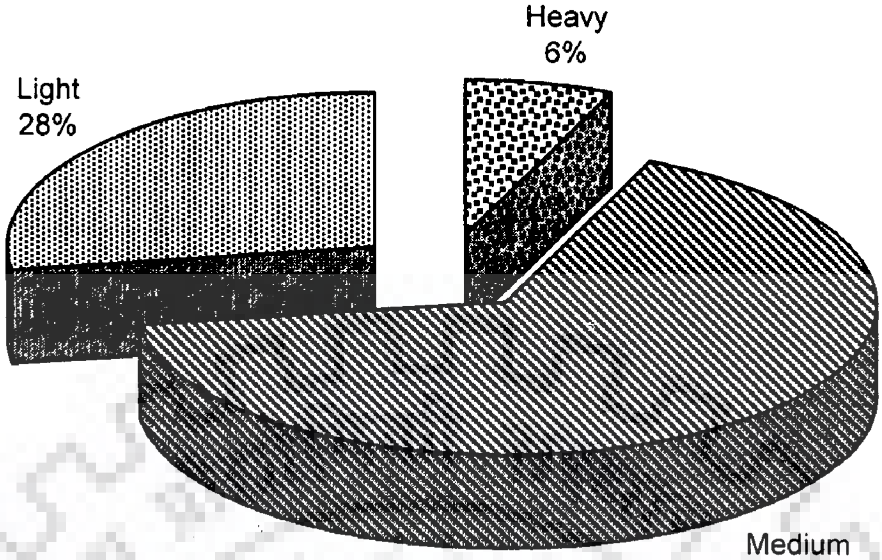


Fig. 5.1 Tunnel length in different support categories following RSR-system

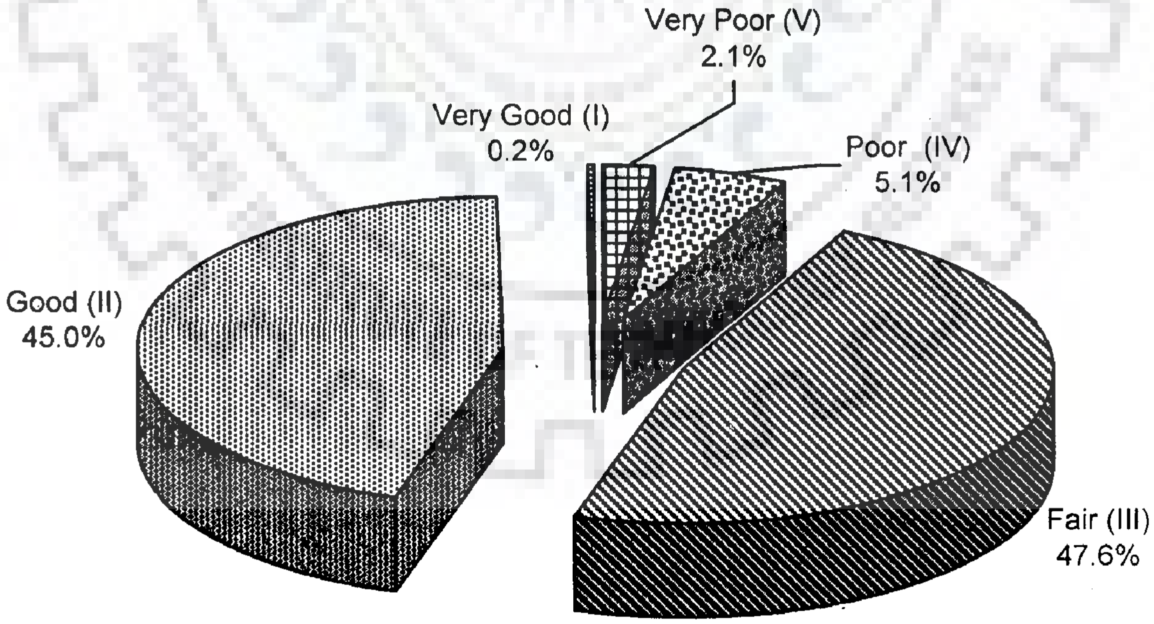
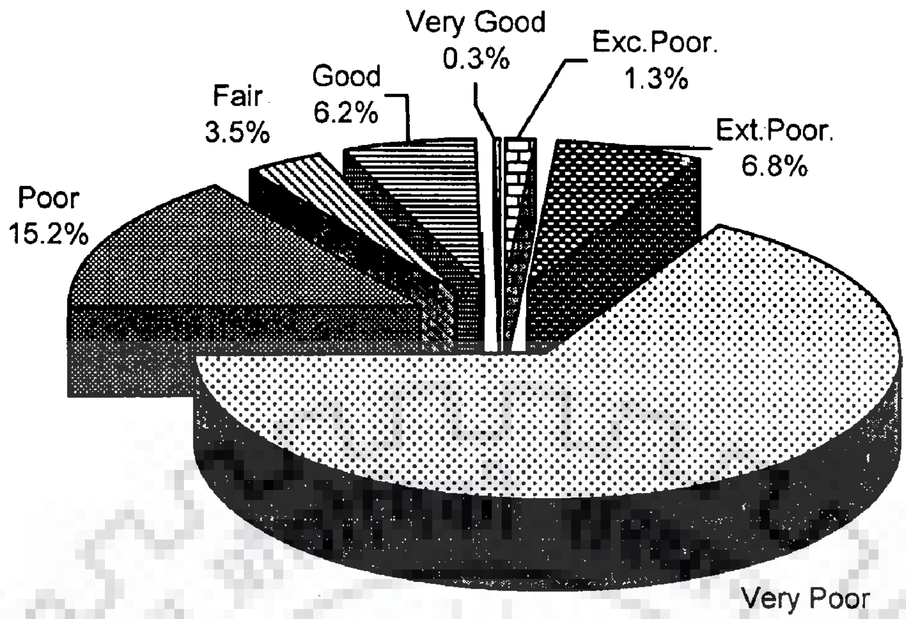
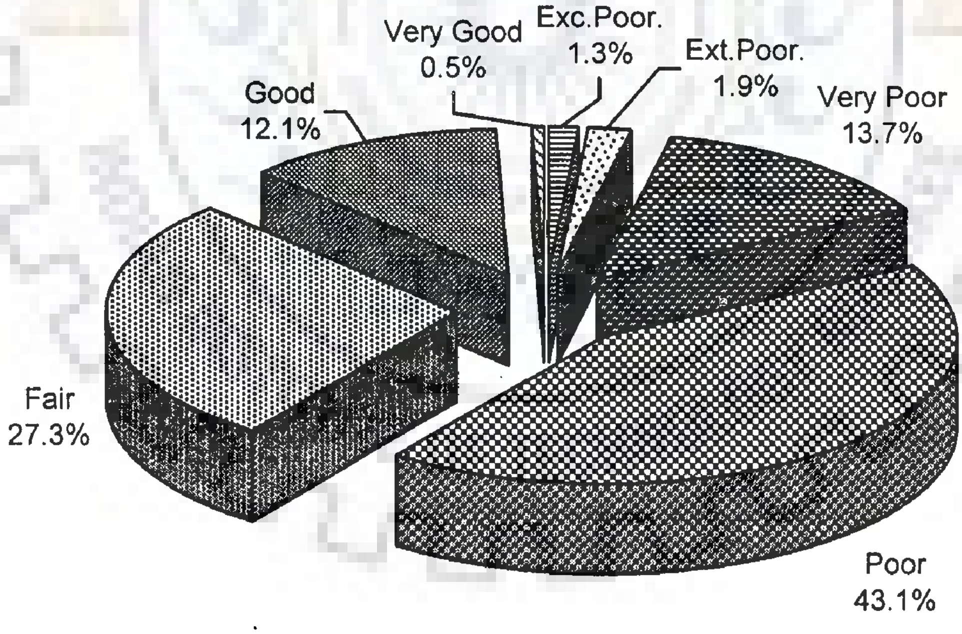


Fig. 5.2 Tunnel length in different types of grounds following RMR-system



(a) Q-system with original SRF-values



(b) Q-system with proposed new SRF-values

Fig. 5.3 Tunnel length in different types of grounds following Q-system

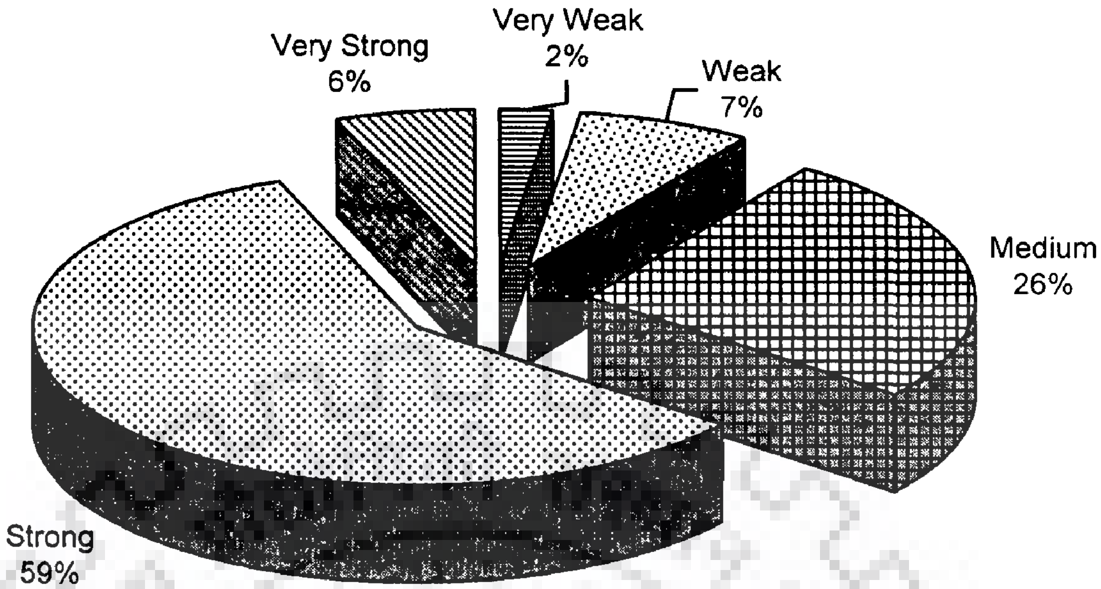


Fig. 5.4 Tunnel length in different types of grounds following RMi-system

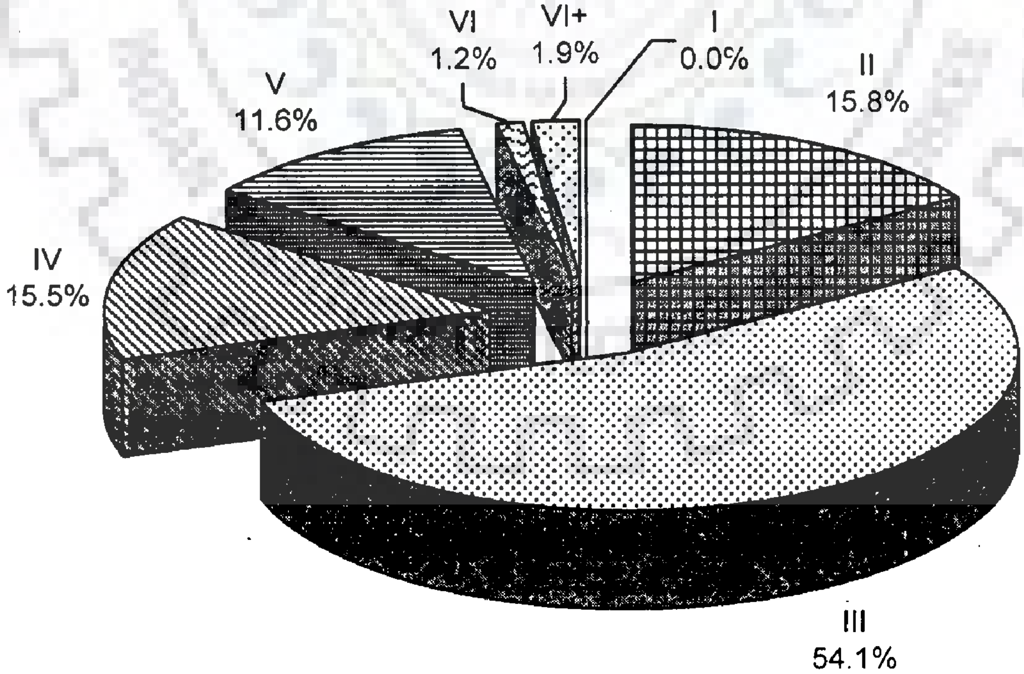


Fig. 5.5 Tunnel length in different classes of supports installed by the Project Authorities

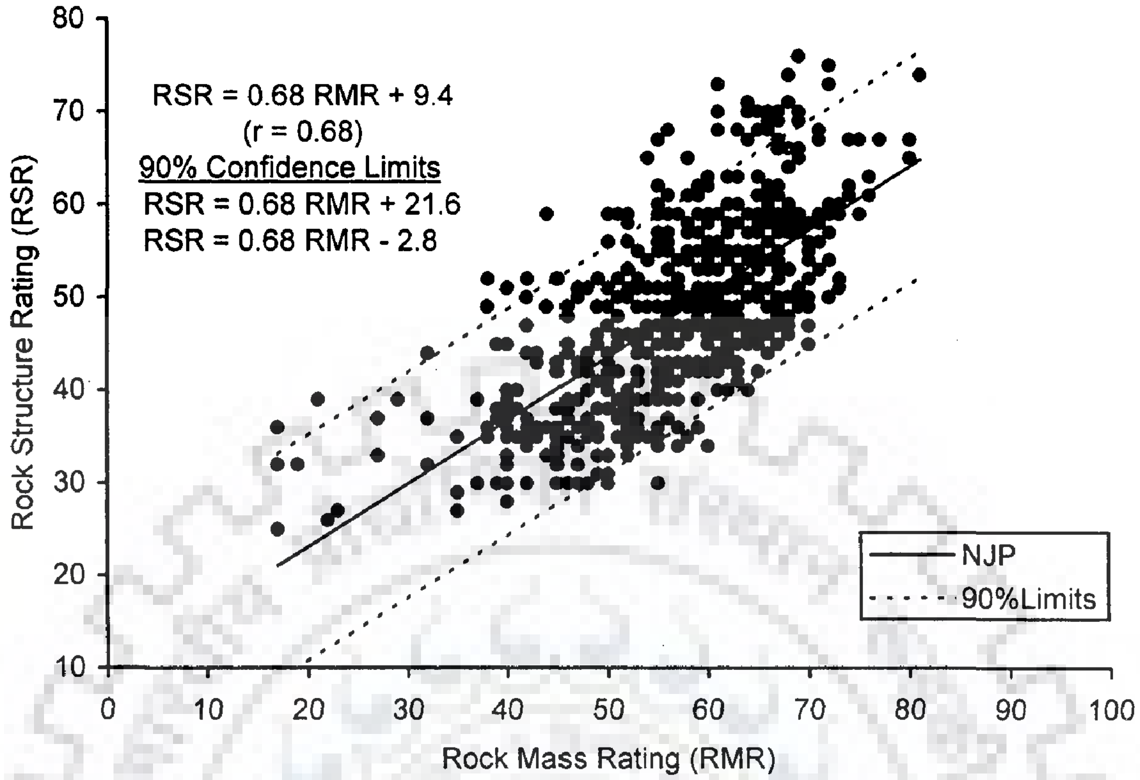


Fig. 5.6 Correlation between RSR and RMR

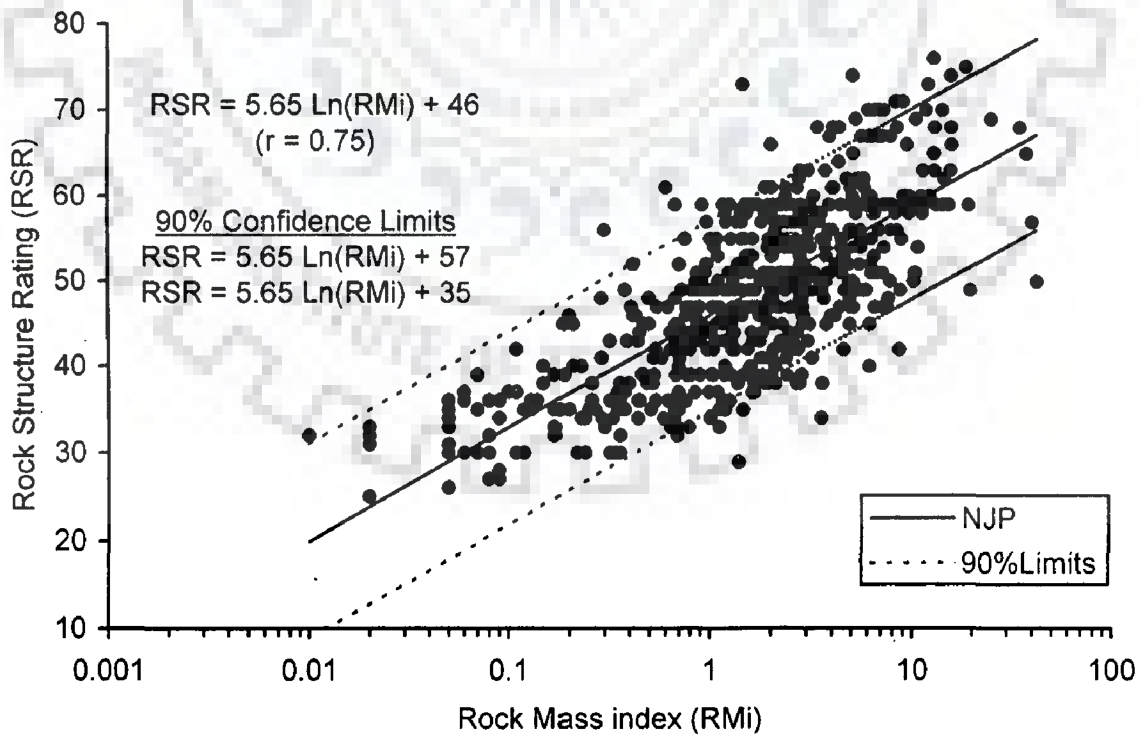


Fig. 5.7 Correlation between RSR and RMi

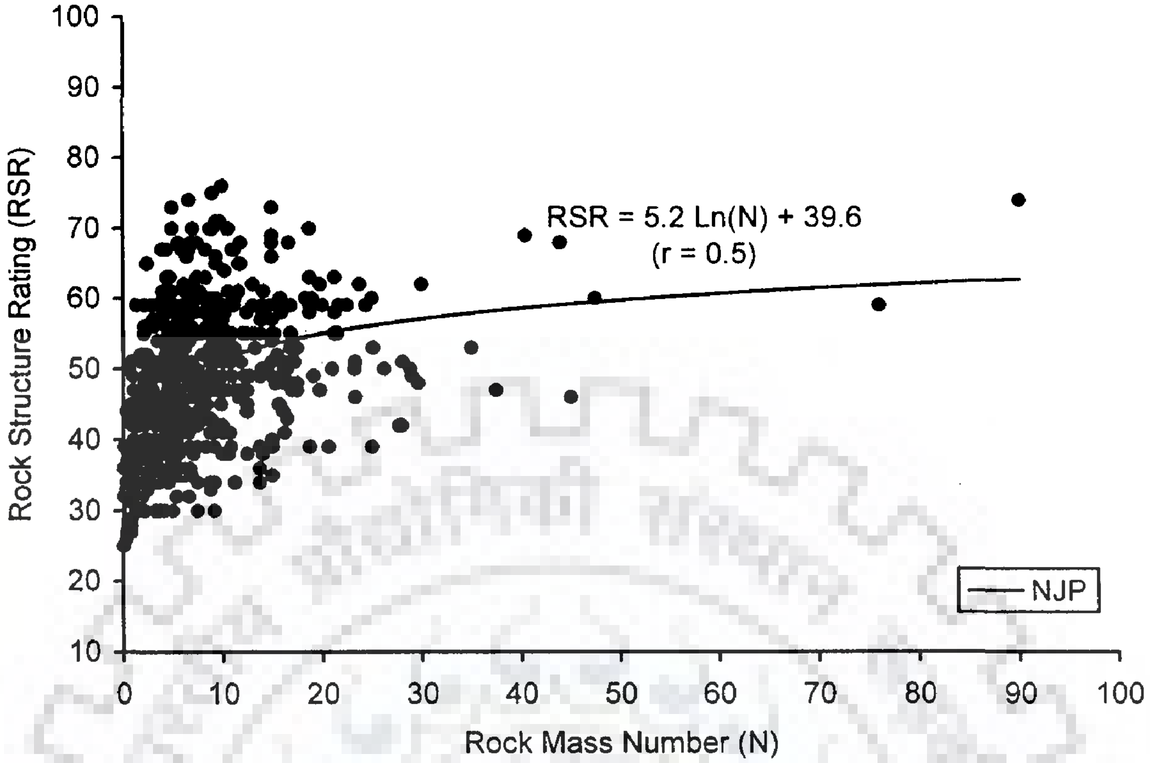


Fig. 5.8 Correlation between RSR and N

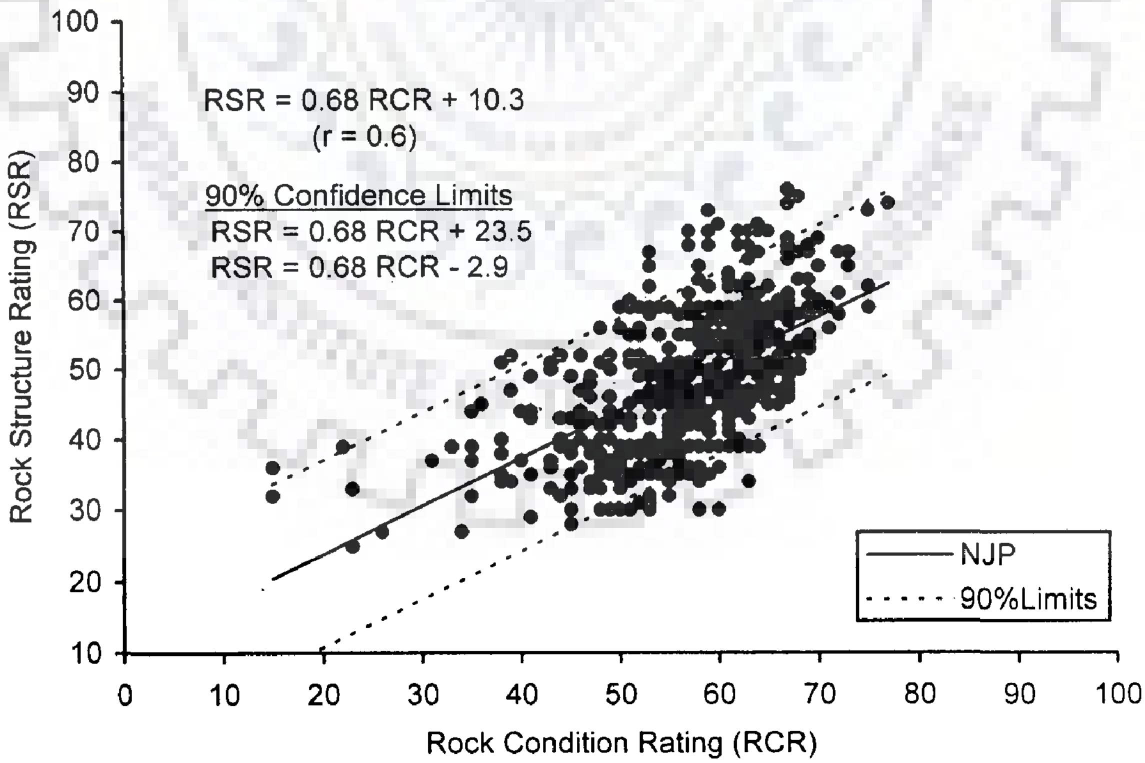


Fig. 5.9 Correlation between RSR and RCR

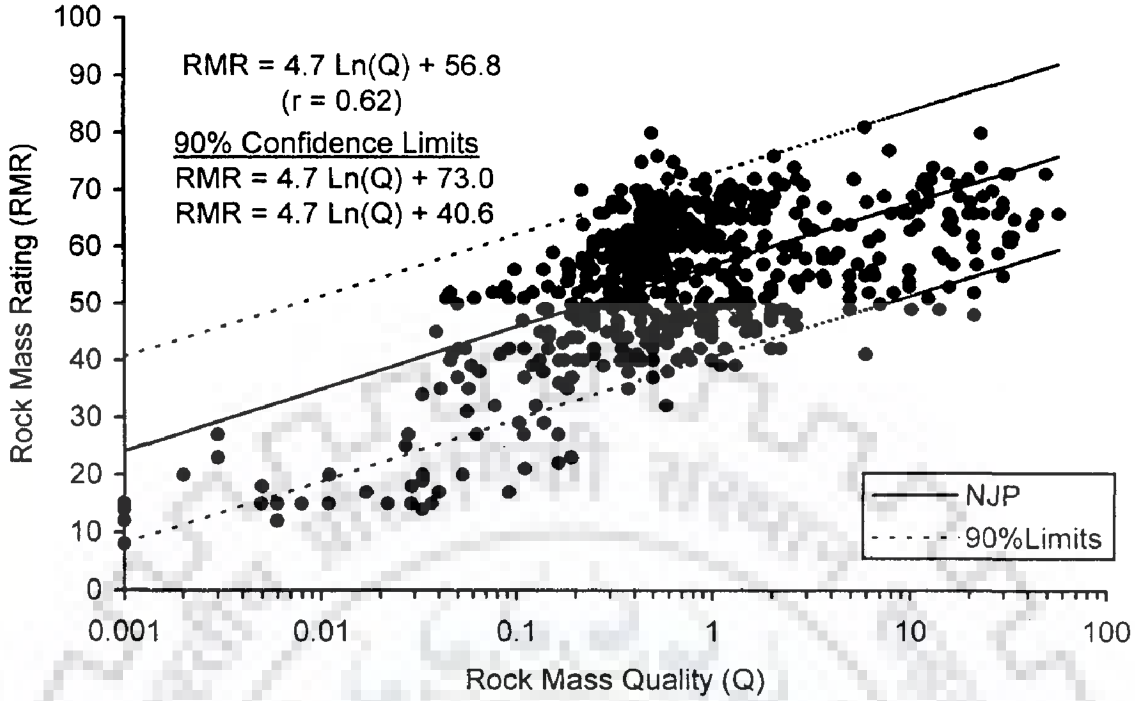


Fig. 5.10 Correlation between RMR and Q

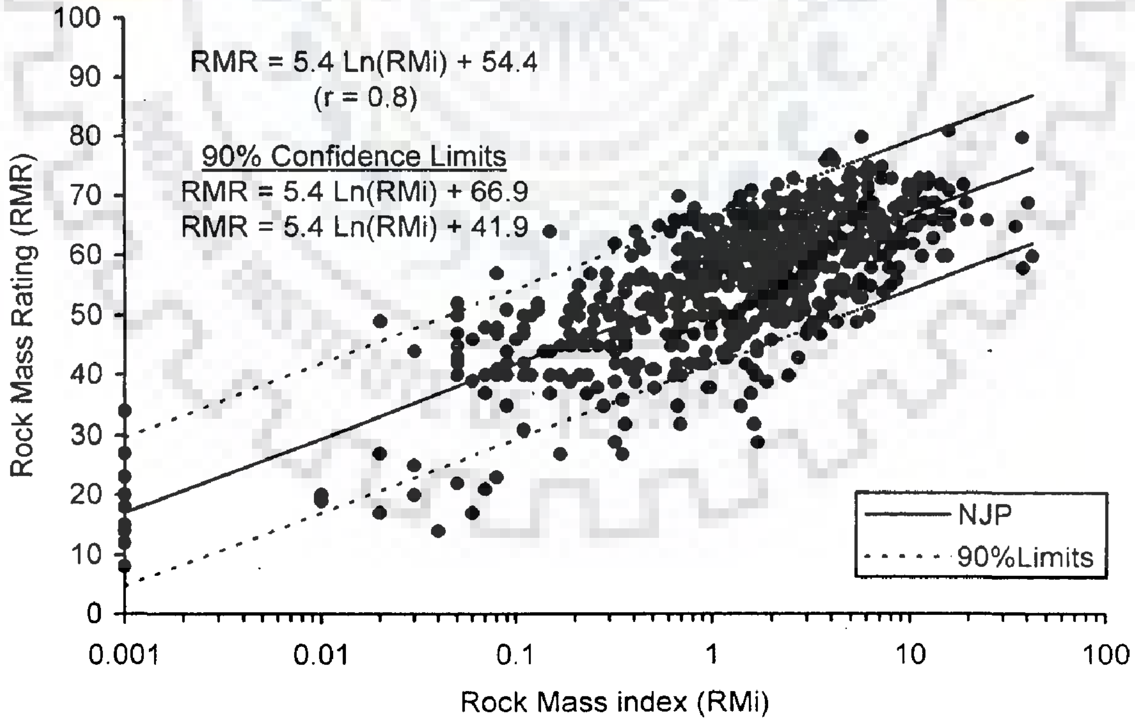


Fig. 5.11 Correlation between RMR and RMi

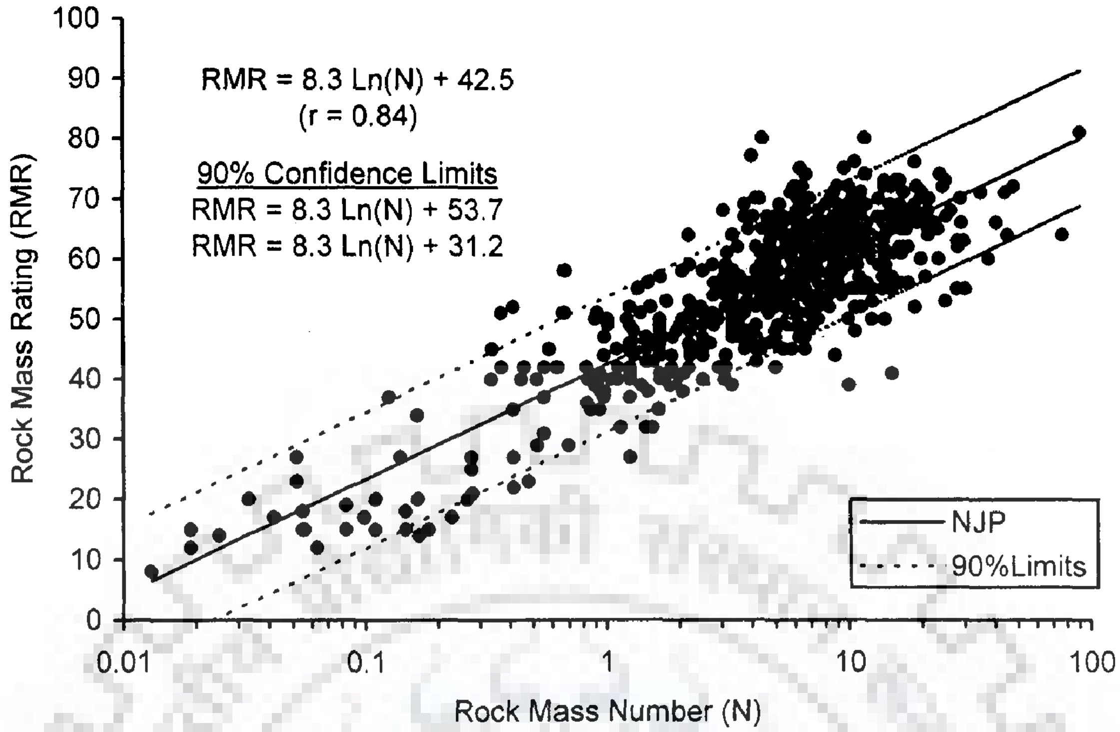


Fig. 5.12 Correlation between RMR and N

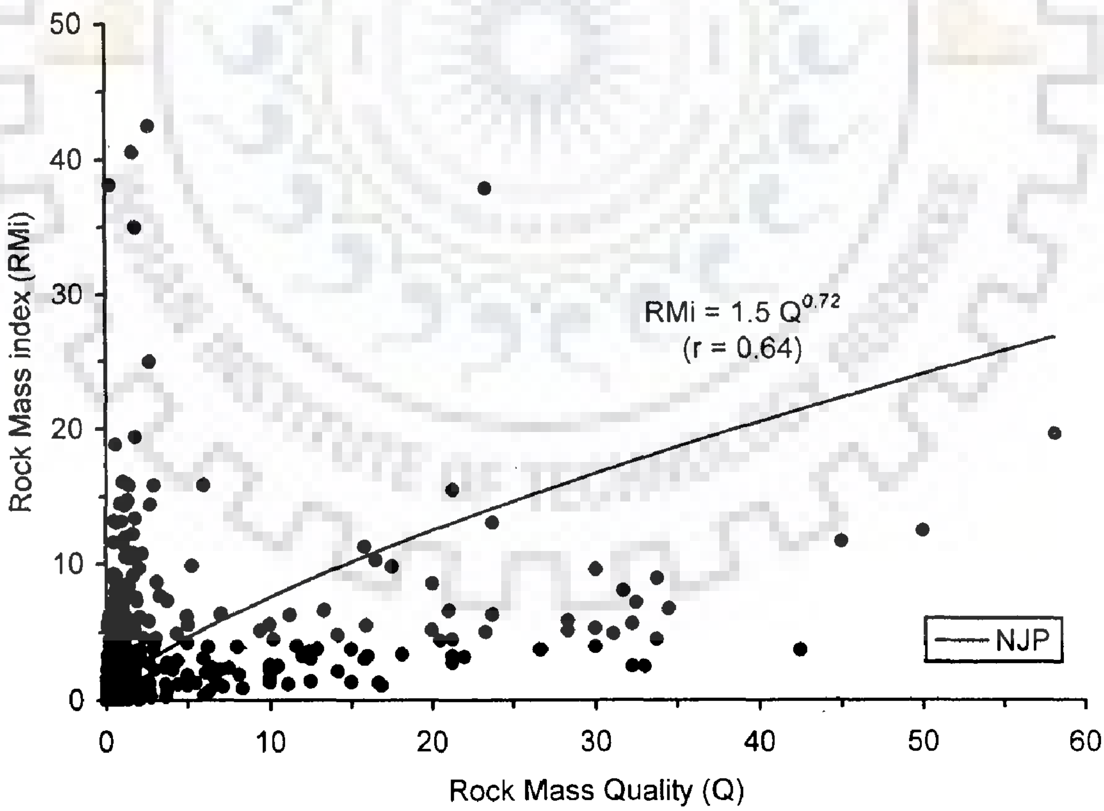


Fig. 5.13 Correlation between RMi and Q

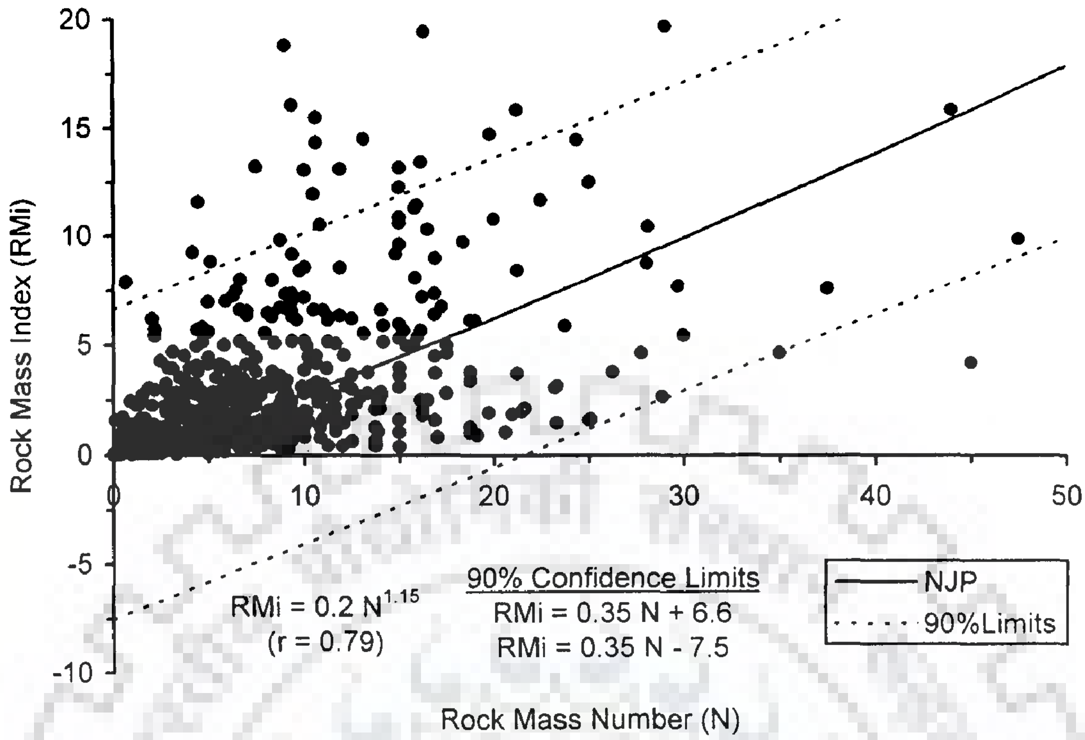


Fig. 5.14 Correlation between RMi and N

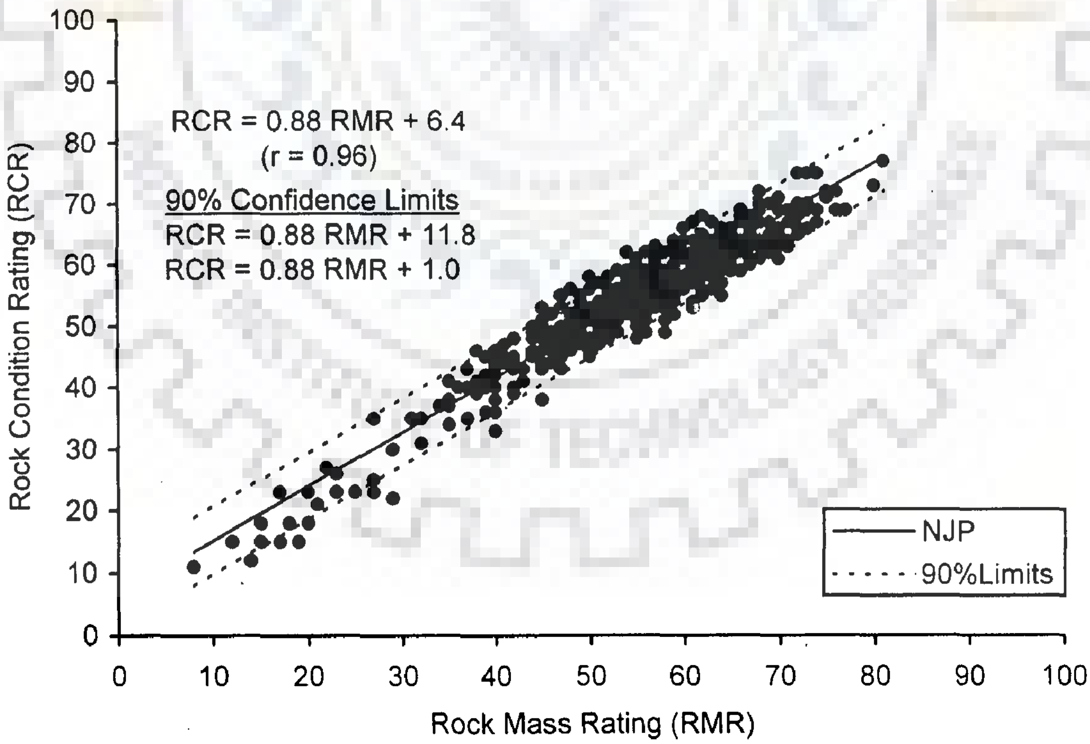


Fig. 5.15 Correlation between RCR and RMR

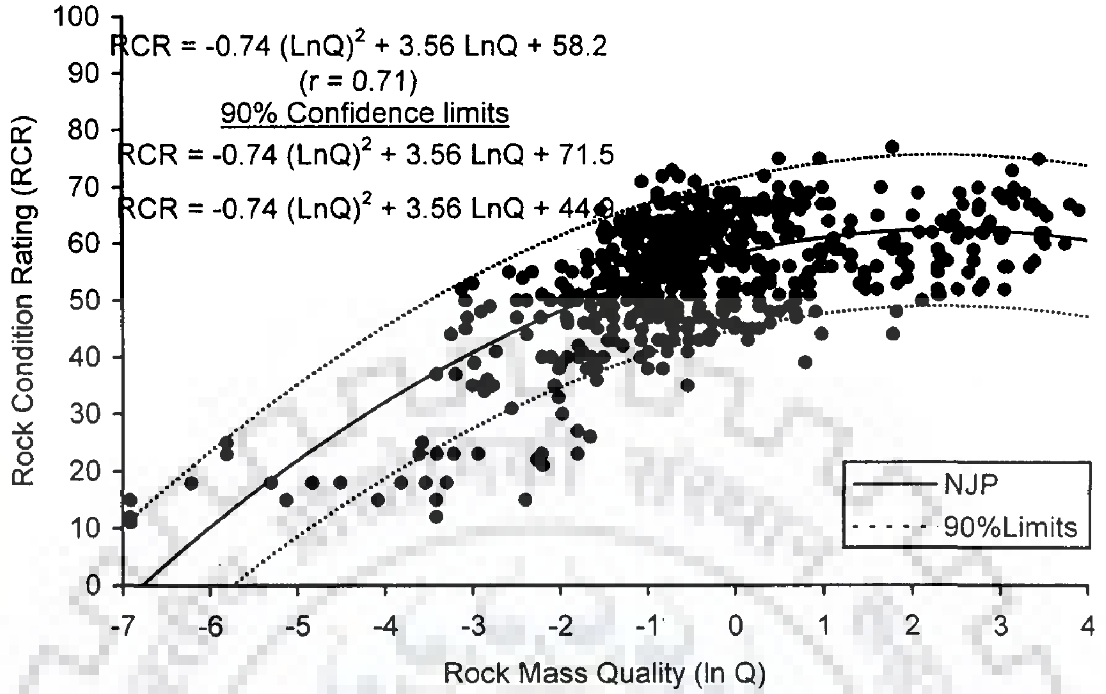


Fig. 5.16 Correlation between RCR and Q (polynomial)

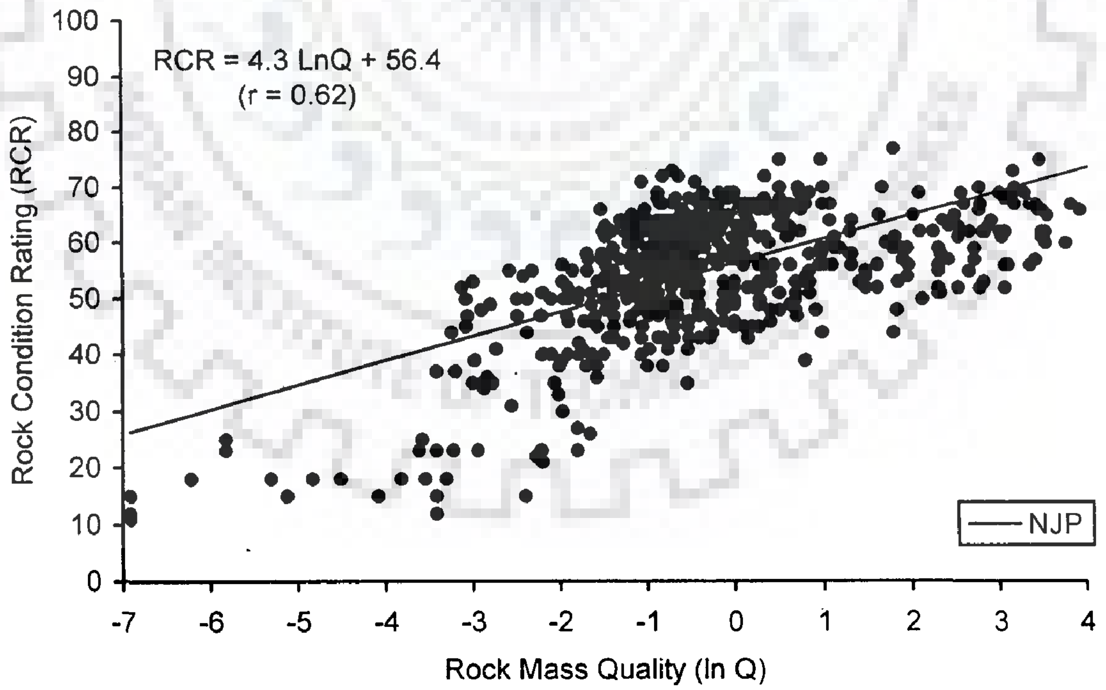


Fig. 5.17 Correlation between RCR and Q (linear)

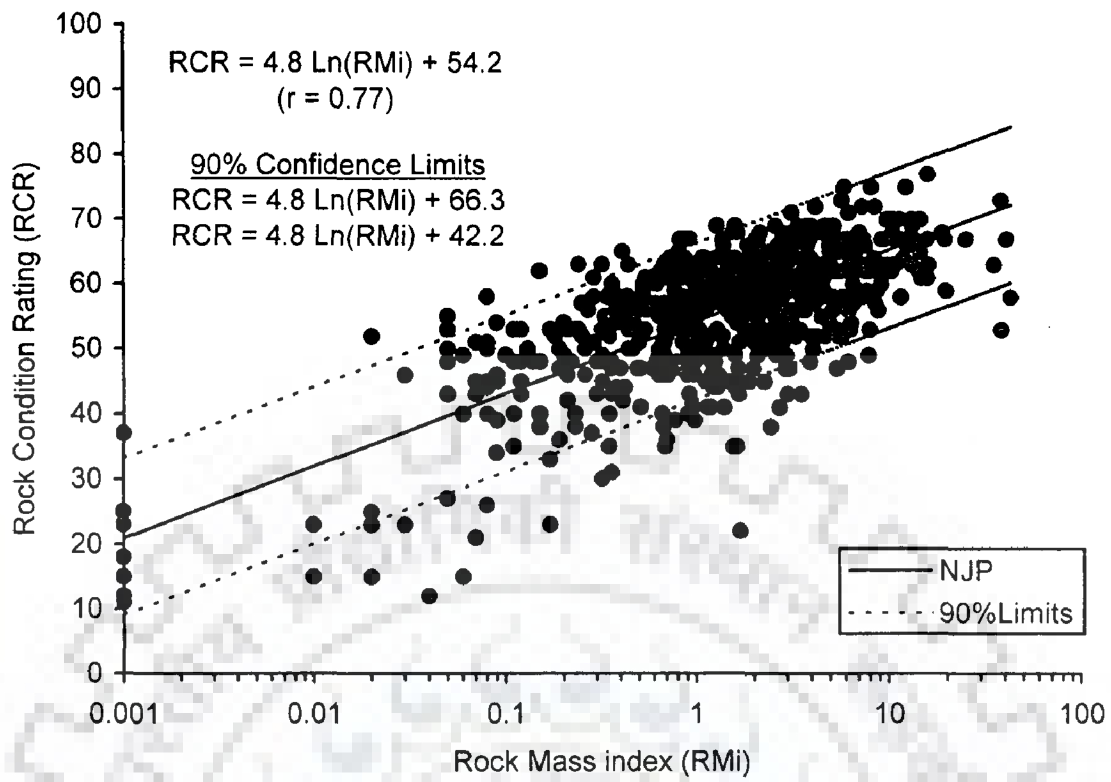


Fig. 5.18 Correlation between RCR and RMi

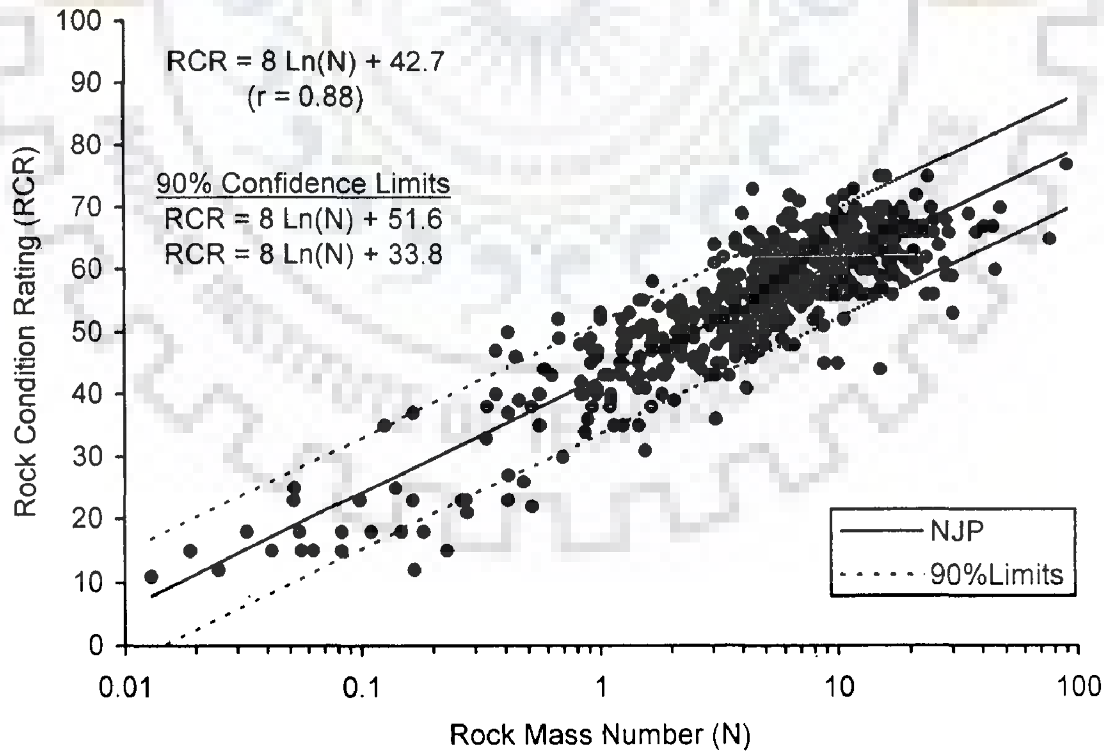


Fig. 5.19 Correlation between RCR and N

Table 5.24 Correlations between Classification Systems

<table>
<tr><th>S. No.</th><th>Correlations</th><th>Correlation Coefficient</th><th>90% Confidence Limits</th></tr>
<tr><td rowspan="2">1.</td><td rowspan="2">$RSR = 0.68\,RMR + 9.4$</td><td rowspan="2">0.68</td><td>$RSR = 0.68\,RMR + 21.6$</td></tr>
<tr><td>$RSR = 0.68\,RMR - 2.8$</td></tr>
<tr><td rowspan="2">2.</td><td rowspan="2">$RSR = 5.65\,Ln\,RMi + 46$</td><td rowspan="2">0.75</td><td>$RSR = 5.65\,Ln\,RMi + 57$</td></tr>
<tr><td>$RSR = 5.65\,Ln\,RMi + 35$</td></tr>
<tr><td>3.</td><td>$RSR = 5.2\,Ln\,N + 39.6$</td><td>0.50</td><td></td></tr>
<tr><td rowspan="2">4.</td><td rowspan="2">$RSR = 0.68\,RCR + 10.3$</td><td rowspan="2">0.60</td><td>$RSR = 0.68\,RCR + 23.5$</td></tr>
<tr><td>$RSR = 0.68\,RCR - 2.9$</td></tr>
<tr><td rowspan="2">5.</td><td rowspan="2">$RMR = 4.7\,Ln\,Q + 56.8$</td><td rowspan="2">0.62</td><td>$RMR = 4.7\,Ln\,Q + 73.0$</td></tr>
<tr><td>$RMR = 4.7\,Ln\,Q + 40.6$</td></tr>
<tr><td rowspan="2">6.</td><td rowspan="2">$RMR = 5.4\,Ln\,RMi + 54.4$</td><td rowspan="2">0.80</td><td>$RMR = 5.4\,Ln\,RMi + 66.9$</td></tr>
<tr><td>$RMR = 5.4\,Ln\,RMi + 41.9$</td></tr>
<tr><td rowspan="2">7.</td><td rowspan="2">$RMR = 8.3\,Ln\,N + 42.5$</td><td rowspan="2">0.84</td><td>$RMR = 8.3\,Ln\,N + 53.7$</td></tr>
<tr><td>$RMR = 8.3\,Ln\,N + 31.2$</td></tr>
<tr><td>8.</td><td>$RMi = 1.5\,Q^{0.72}$</td><td>0.64</td><td></td></tr>
<tr><td rowspan="2">9.</td><td rowspan="2">$RMi = 0.2\,N^{1.15}$</td><td rowspan="2">0.79</td><td>$RMi = 0.35N + 6.6$</td></tr>
<tr><td>$RMi = 0.35N - 7.5$</td></tr>
<tr><td rowspan="2">10.</td><td rowspan="2">$RCR = 0.88\,RMR + 6.4$</td><td rowspan="2">0.96</td><td>$RCR = 0.88\,RMR + 11.8$</td></tr>
<tr><td>$RCR = 0.88\,RMR + 1.0$</td></tr>
<tr><td rowspan="2">11.</td><td rowspan="2">$RCR = -0.74(Ln\,Q)^2 + 3.56\,Ln\,Q + 58.2$</td><td rowspan="2">0.71</td><td>$RCR = -0.74(Ln\,Q)^2 + 3.56\,Ln\,Q + 71.5$</td></tr>
<tr><td>$RCR = -0.74(Ln\,Q)^2 + 3.56\,L\,Q + 44.9$</td></tr>
<tr><td>12.</td><td>$RCR = 4.3\,Ln\,Q + 56.4$</td><td>0.62</td><td></td></tr>
<tr><td rowspan="2">13.</td><td rowspan="2">$RCR = 4.8\,Ln\,RMi + 54.2$</td><td rowspan="2">0.77</td><td>$RCR = 4.8\,Ln\,RMi + 66.3$</td></tr>
<tr><td>$RCR = 4.8\,Ln\,RMi + 42.2$</td></tr>
<tr><td rowspan="2">14.</td><td rowspan="2">$RCR = 8.0\,Ln\,N + 42.7$</td><td rowspan="2">0.88</td><td>$RCR = 8.0\,Ln\,N + 51.6$</td></tr>
<tr><td>$RCR = 8.0\,Ln\,N + 33.8$</td></tr>
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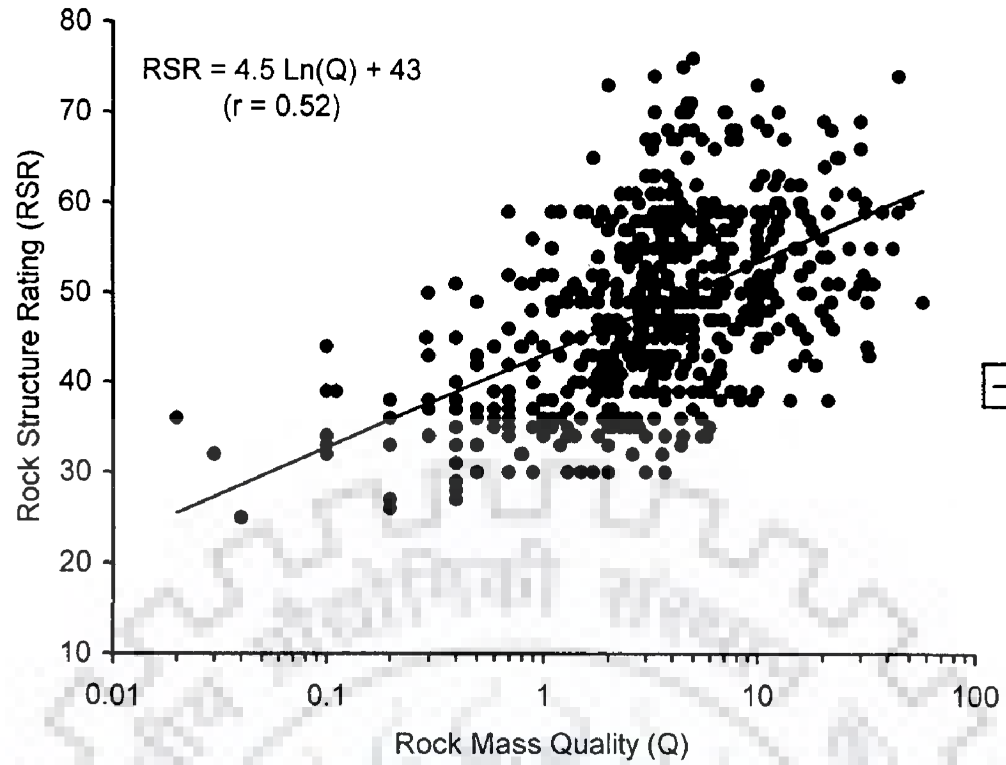


Fig. 5.20 Correlation between RSR and Q
(with proposed new SRF-values)

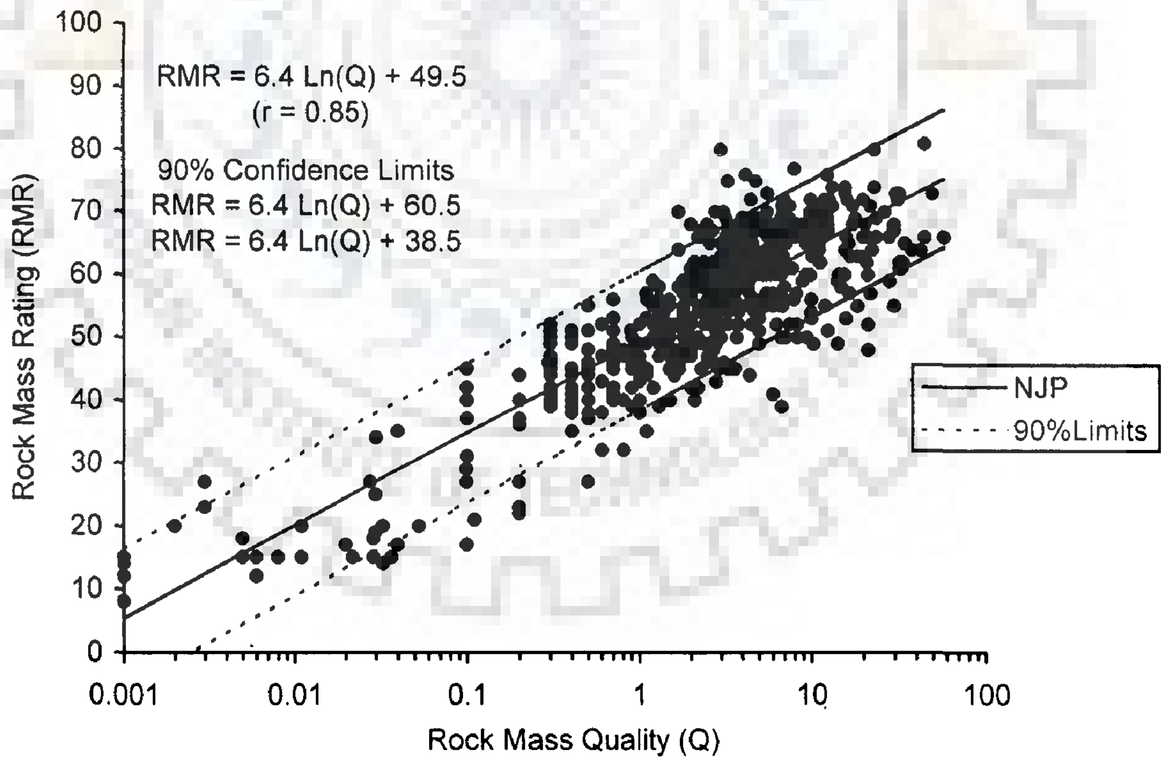


Fig. 5.21 Correlation between RMR and Q
(with proposed new SRF-values)

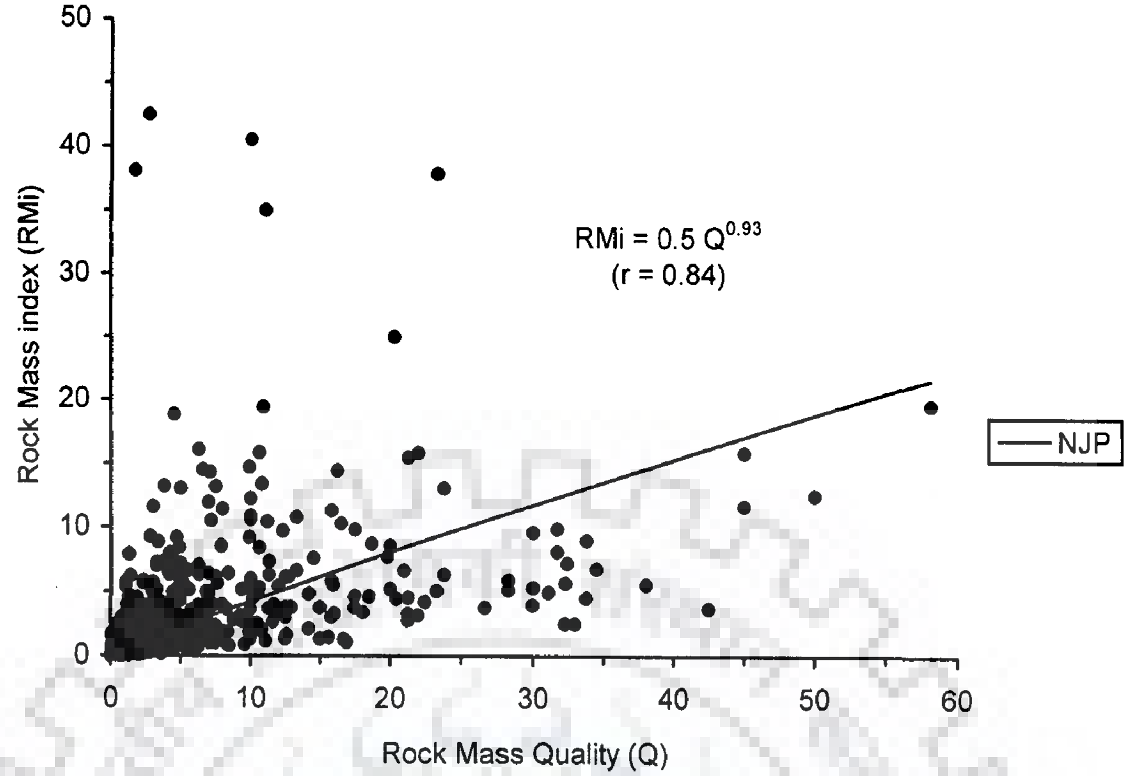


Fig. 5.22 Correlation between RMi and Q
(with proposed new SRF-values)

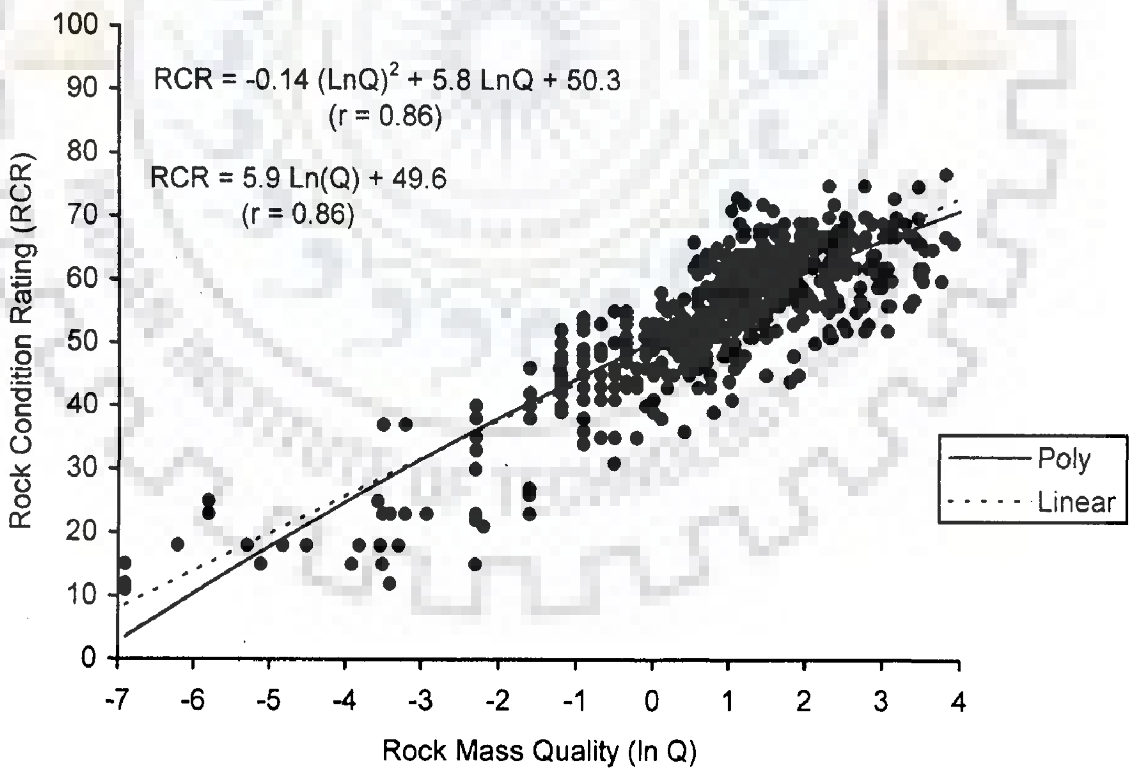


Fig. 5.23 Correlation between RCR and Q
(with proposed new SRF-values)

Table 5.25 Comparison of Correlations

Revised Correlations (Q with proposed new SRF-values)	Un-Revised Correlations (Q with original SRF-values)	Earlier Correlations
--	$RSR = 0.68\,RMR + 9.4$ $(r = 0.68)$	Rutledge and Preston (1978) $RSR = 0.77\,RMR + 12.4$
$RSR = 4.5\,Ln\,Q + 43$ $(r = 0.52)$	--	Rutledge and Preston (1978) $RSR = 5.8\,Ln\,Q + 46.5$
$RMR = 6.4\,Ln\,Q + 49.6$ $(r = 0.85)$	$RMR = 4.7\,Ln\,Q + 56.8$ $(r = 0.62)$	Bieniawski (1976) $RMR = 9.0\,Ln\,Q + 44.0$ Rutledge and Preston (1978) $RMR = 5.9\,Ln\,Q + 43.0$ $(r = 0.81)$ Moreno (1980) $RMR = 5.4\,Ln\,Q + 55.2$ $(r = 0.55)$ Cameron-Clarke and Budavari (1981) $RMR = 5.0\,Ln\,Q + 60.8$ (High Scatter) Abad et al. (1984) $RMR = 10.5\,Ln\,Q + 41.8$ $(r = 0.66)$
$RMi = 0.5\,Q^{0.93}$ $(r = 0.84)$	$RMi = 1.5\,Q^{0.72}$ $(r = 0.64)$	--
$RCR = -0.14(Ln\,Q)^2 + 5.3\,Ln\,Q + 50.3$ $(r = 0.86)$	$RCR = -0.74(Ln\,Q)^2 + 3.56\,Ln\,Q + 58.2$ $(r = 0.71)$	--
$RCR = 5.9\,Ln\,Q + 49.6$ $(r = 0.86)$	$RCR = 4.3\,Ln\,Q + 56.4$ $(r = 0.62)$	--
--	$RMR = 8.3\,Ln\,N + 42.5$ $(r = 0.84)$	Verman (1993) $RMR = 10\,Ln\,N + 36$ $(r = 0.91)$
--	$RCR = 8.0\,Ln\,N + 42.7$ $(r = 0.88)$	Goel (1995) $RCR = 8.0\,Ln\,N + 30.0$ $(r = 0.92)$

Limitations of these correlations are as follows:

(a) The RSR-system:

In the geological structure, massive rocks have not been included. The rocks encountered in all the tunnel sections are in the category of metamorphic rocks only.

(b) The RMR-system:

RMR = 8 to 81.

(c) The Q-system:

The parameters are in the following ranges:

J_n = 3 to 20, J_r = 0.5 to 3, J_a = 0.75 to 13, J_w = 0.15 to 1 and SRF = 0.5 to 20.

(d) The RMi-system:

The parameters are in the following ranges:

q_c = 9.5 to 125 MPa, jL = 0.75 to 1 and jR and jA same as J_r and J_a respectively of the Q-system.

DESIGN OF SUPPORT SYSTEMS

The chief aim of RMC is to estimate rock loads that are anticipated as well as type and amount of the support system required. RMC has been done using four methods of classification i.e. RSR, RMR, Q and RMi as covered in CHAPTER - 5. Various classification systems have suggested different types of support systems based on the classification of the rock mass. For example, the RSR-system recommended use of steel ribs as main support element, whereas RMR, Q and RMi have recommended shotcrete and rock bolt as the main supporting element. The RMR-system though recommends steel ribs also for poor rock masses apart from shotcrete and rock bolt. RMi is the recent RMC system proposed by Palmstrom (1995a), which has recommended use of SFRS in place of shotcrete and in the same way the Q-system has also updated its support methods by replacing shotcrete with SFRS.

6.1 ROCK STRUCTURE RATING (RSR) – SYSTEM

6.1.1 Design of Support

Wickham et al. (1972) used the concept of a standard datum support in order to develop a relationship between RSR and the tunnel support. A standard datum support, identified by a steel set size and spacing, is that which is capable of carrying a load equal to that imposed by loose, saturated, cohesionless sand (water table is below in Terzaghi's empirical equation). The height of the loose zone of sand is as follows:

$$H_p = 1.38\ (B + H_t) \tag{6.1}$$

For a given diameter of tunnel and a particular size of steel set, its spacing can be calculated which corresponds to the standard datum loading. A ratio called the rib ratio (RR) was then used to compare the actual set spacing with the set spacing for the standard datum support (for the steel section being used). Theoretical spacing of typical rib sizes for datum condition is indicated in Fig. III.1 and Table III.1 (APPENDIX – IIID).

$$RR = \frac{\text{Spacing of standard datum support}}{\text{Actual set spacing}} \times 100 \tag{6.2}$$

Wickham et al. (1972) developed the following empirical relationship between RR and RSR from a study of 53 projects:

$$(RR + 80)\,(RSR + 30) = 8800 \tag{6.3}$$

RR varying between 0 (no support) and 100 (heavy support) correspond to RSR-values of 80 and 19 respectively. Wickham et al. (1972) have recommended that for RSR ≥ 67, RR-value of 10 should be considered for the design of support. From RSR-value, Eq. 6.3 can be used to obtain RR and thereby determine the steel ribs required for a tunnel of any size. In the present work spacing of steel ribs, designated as ISMB 250 according to the Indian Standard Specifications, have been computed for 635 tunnel sections on the basis of RSR-values.

Although the RSR-system was developed primarily with respect to steel rib type of support, the correlation between RSR and rock loads was extended to show a general relationship between the RSR-system and shotcrete and rock bolt types of supports.

Wickham et al. (1972) had proposed relationships for computing thickness of shotcrete and spacing of rock bolts as presented below:

$$t_{sc} = 25.4 + 0.416\, p_v \quad \text{mm} \tag{6.4}$$

$$S_{bolt} = \sqrt{\frac{P_{bolt}}{p_v}} \quad \text{m} \tag{6.5}$$

where, p_v is in kPa and P_{bolt} in kN.

6.1.2 Estimation of Support Pressure

Since RR relate to a defined rock load, RSR-values can also be expressed in terms of unit loads. Equation 6.3 is developed to give the following equation for roof pressure (p_v):

$$p_v = 0.26(B + H_t)\left[\frac{8800}{(RSR + 30)} - 80\right] \quad \text{kPa} \tag{6.6}$$

where, B and H_t are in m. In the present work, B = H_t = 11.65 m.

Results of **ROMAC** as discussed in section 5.4, for the RSR-system include the RSR-value, rib ratio, roof pressure, thickness of shotcrete and spacing of rock bolts (APPENDIX – VIIB). Thickness of shotcrete and spacing of rock bolts have been computed as per Eqs.6.4 and 6.5 respectively. Rock bolts of 25 mm diameter with the capacity of 200 kN have been considered for the design.

6.2 ROCK MASS RATING (RMR) - SYSTEM

6.2.1 Design of Support

The RMR-system provides guidelines for the selection of rock reinforcement for tunnels in accordance with APPENDIX – IVF. These guidelines depend on depth below surface (in-situ stress), tunnel size and shape, and the method of excavation. These are permanent support measures applicable to rock masses excavated using conventional drilling and blasting procedures.

On the basis of these recommendations, linear interpolations have been performed for estimation of support details for a particular RMR-value, which lies in any of the rock mass class given by Bieniawski (1989) as follows:

$$t_{sc} = 200 - 2.5\ RMR;\ \text{and} \geq 50 \quad \text{mm} \quad (\text{for RMR} \leq 80) \tag{6.7}$$

$l_{bolt} = 6 - 0.05\ RMR;\ \text{and}\ \geq 3$ m (for RMR ≤ 80) (6.8)

$S_{bolt} = 0.5 + 0.025\ RMR;\ \text{and}\ \geq 1$ m (for RMR ≤ 80) (6.9)

$S_{rib} = 0.375 + 0.0375\ RMR$ m (for RMR ≤ 40) (6.10)

$S_{rib} = 0$ (for RMR ≥ 40) (6.11)

Results of **ROMAC**, for the RMR-system, include recommended support measures given in APPENDIX – IVF.

6.2.2 Estimation of Support Pressure

Unal (1983), on the basis of his studies in coal mines, proposed the following correlation for support pressure (p_v) using RMR for openings with flat roof:

$$p_v = \left(\frac{100 - RMR}{100}\right).\gamma\ B \qquad (6.12)$$

Goel and Jethwa (1991) proposed the following correlation:

$$p_v = \frac{7.5\ B^{0.1}\ H^{0.5} - RMR}{20\ RMR} \quad \text{MPa} \qquad (6.13)$$

where, B and H are in m.

Goel (1994) have proposed following empirical correlations for estimating support pressure for tunnel sections under non-squeezing and squeezing ground conditions using RCR:

<u>Non-Squeezing ground condition:</u>

$$p_v = \frac{46.6\ H^{0.15}.B^{0.1}}{RCR^{1.98}} \quad \text{MPa} \qquad (6.14)$$

<u>Squeezing ground condition:</u>

$$p_v = \frac{f(RCR)}{30}.10^{\frac{H^{0.6}.a^{0.1}}{RCR^{1.2}}} \quad \text{MPa} \quad [\text{for } f(RCR) \text{ refer Table 6.1}] \qquad (6.15)$$

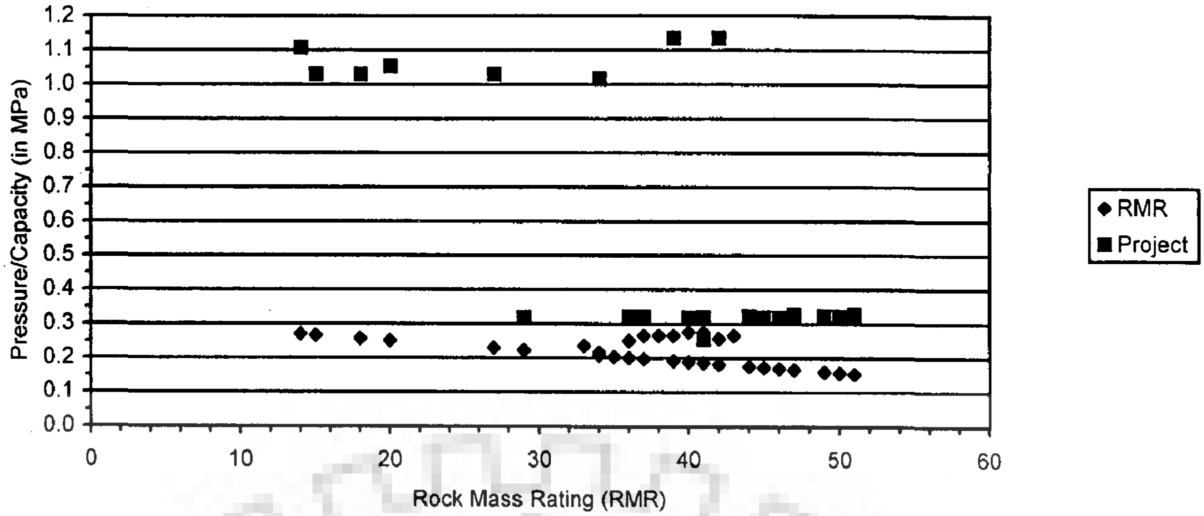
where, H, B and a are in m.

Table 6.1 Correction Factors for Tunnel Closure (for Eqs. 6.15 and 6.30)

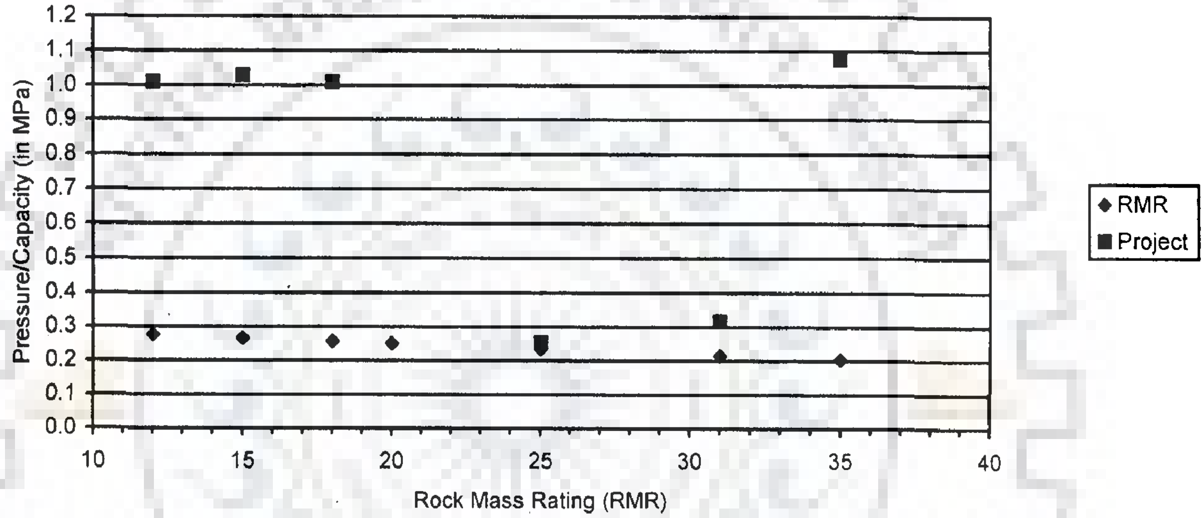
S. No.	Degree of Squeezing	Normalized Tunnel Closure, %	f(RCR)	f(N)
1.	Very mild squeezing ($270\ N^{0.33}.B^{-0.1} < H < 360\ N^{0.33}.B^{-0.1}$)	2	2.05	1.55
2.	Mild squeezing ($360\ N^{0.33}.B^{-0.1} < H < 450\ N^{0.33}.B^{-0.1}$)	3	1.43	1.20
3.	Mild to moderate squeezing ($450\ N^{0.33}.B^{-0.1} < H < 540\ N^{0.33}.B^{-0.1}$)	4	0.95	1.00
4.	Moderate squeezing ($540\ N^{0.33}.B^{-0.1} < H < 630\ N^{0.33}.B^{-0.1}$)	5	0.80	0.80
5.	High squeezing ($630\ N^{0.33}.B^{-0.1} < H < 800\ N^{0.33}.B^{-0.1}$)	6	1.20	1.05
6.	Very high squeezing ($800\ N^{0.33}.B^{-0.1} < H$)	7	1.80	1.50

Figure 6.1 compares pressure estimated from Eq. 6.12 proposed by Unal (1983) and the capacity of support installed by the Project Authorities for squeezing ground conditions. Capacity of support installed by the Project Authorities has been estimated by modified semi - empirical method of support design, which is discussed in detail in CHAPTER – 7. From Figs. 6.1 a, b and c it is quite clear that support pressure for mild, moderate and high squeezing ground conditions as per Unal (1983) are far lower than the capacity of installed support. Therefore, the conclusion by Goel and Jethwa (1991), that the estimated support pressures from Eq. 6.12 by Unal (1983) were unsafe under squeezing ground conditions, has been corroborated from the present study.

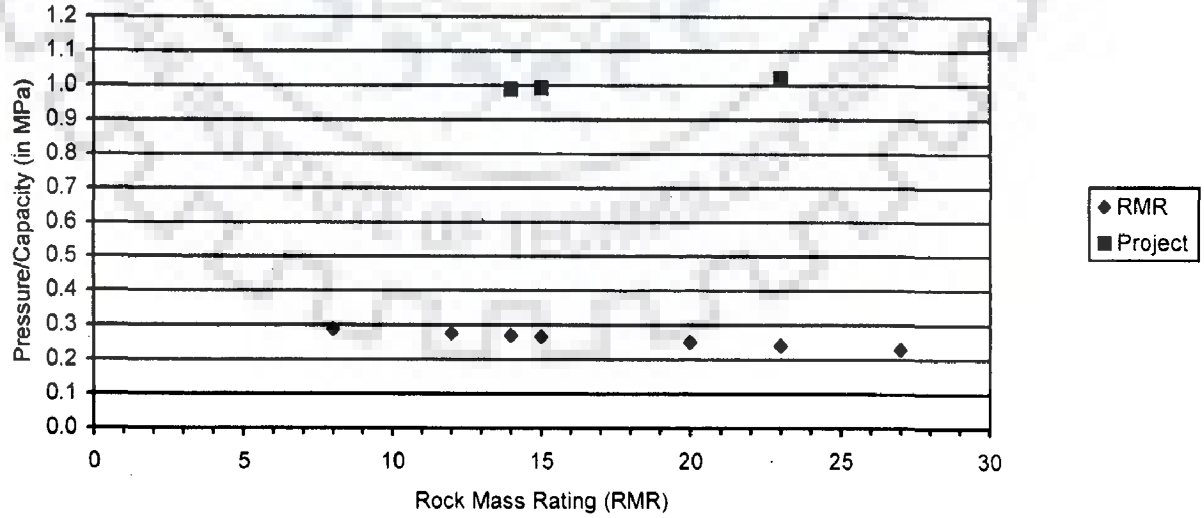
Equation 6.13 proposed by Goel and Jethwa (1991) has been applied in the present work for non-squeezing ground conditions and its results have been compared with that from N (Fig. 6.2). The basis for choosing N for the comparison is dealt in CHAPTER - 9. In Fig. 6.2 a, it is observed that pressures estimated from Eq. 6.13 are slightly on lower side as compared to that from N.



(a) mild squeezing

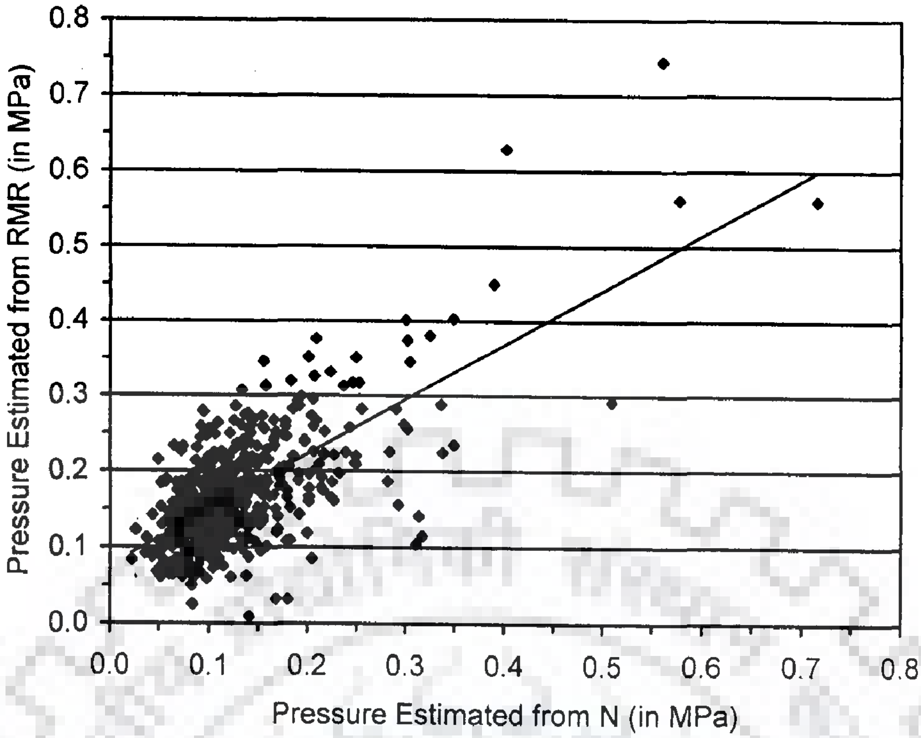


(b) moderate squeezing

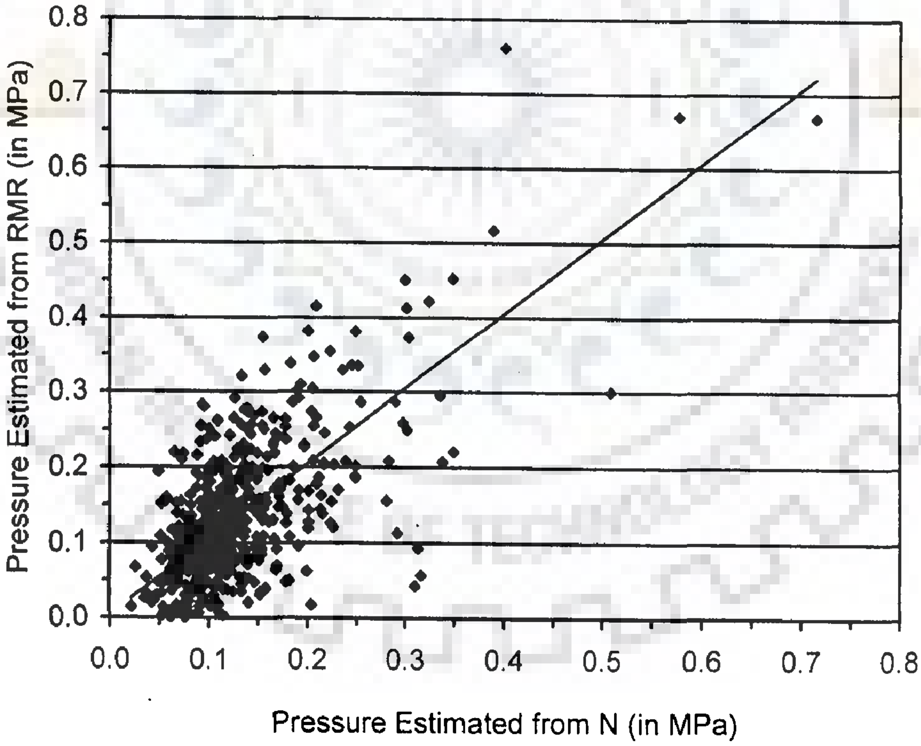


(c) high squeezing

Fig. 6.1 Comparison of pressure estimated from RMR (Unal, 1983) and capacity of support installed by the Project Authorities for squeezing ground conditions



(a) pressure estimated from RMR (Goel and Jethwa, 1991)



(b) pressure estimated from RMR (using Eq. 6.16 proposed in this study)

Fig. 6.2 Comparison of pressure estimated from RMR and N for non-squeezing ground conditions

Minor modification in Eq. 6.13 is proposed from this study as given in Eq. 6.16. Equation 6.16 gives a better correlation between the two (Fig. 6.2 b):

$$p_v = \frac{7.5\,B^{0.1}\,H^{0.5} - RMR}{14.6\,RMR} - 0.1 \quad \text{MPa} \tag{6.16}$$

where, B and H are in m.

Equation 6.14 given by Goel (1994) for non-squeezing ground condition, has been applied in the present work and its results have been compared with that from N (Fig. 6.3). From Fig. 6.3 a, it can be observed that pressures estimated with RCR from Eq. 6.14 are on lower side compared to that from N. Minor modification in Eq. 6.14 is proposed from this study as given in Eq. 6.17. Equation 6.17 gives a better correlation between the two (Fig. 6.3 b):

$$p_v = \frac{74\,H^{0.15}.B^{0.1}}{RCR^{1.98}} + 0.03 \quad \text{MPa} \tag{6.17}$$

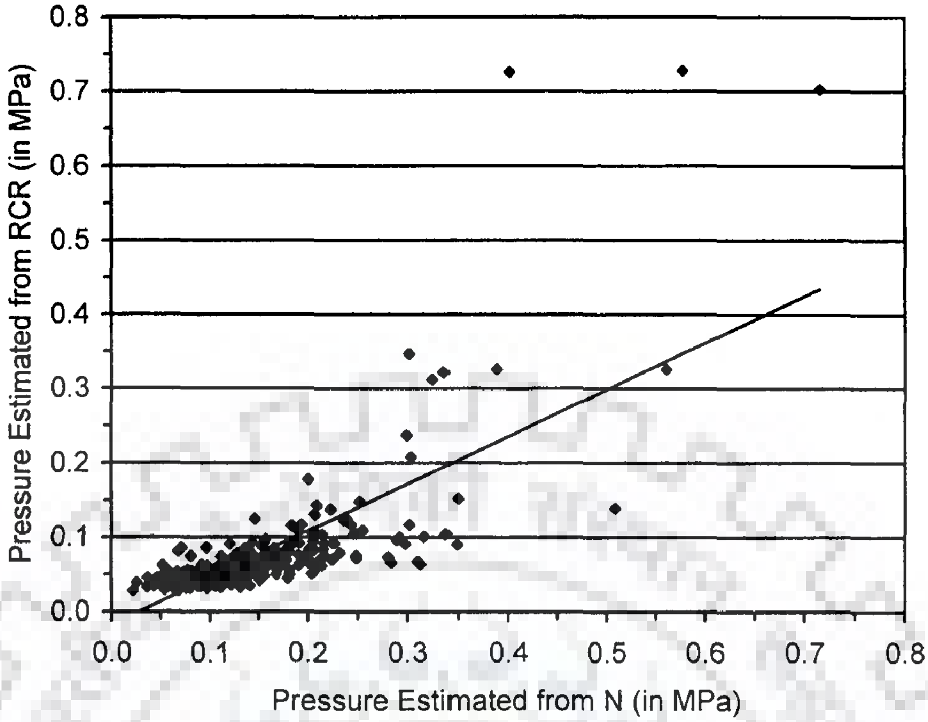
6.3 ROCK MASS QUALITY (Q) – SYSTEM

6.3.1 Design of Support

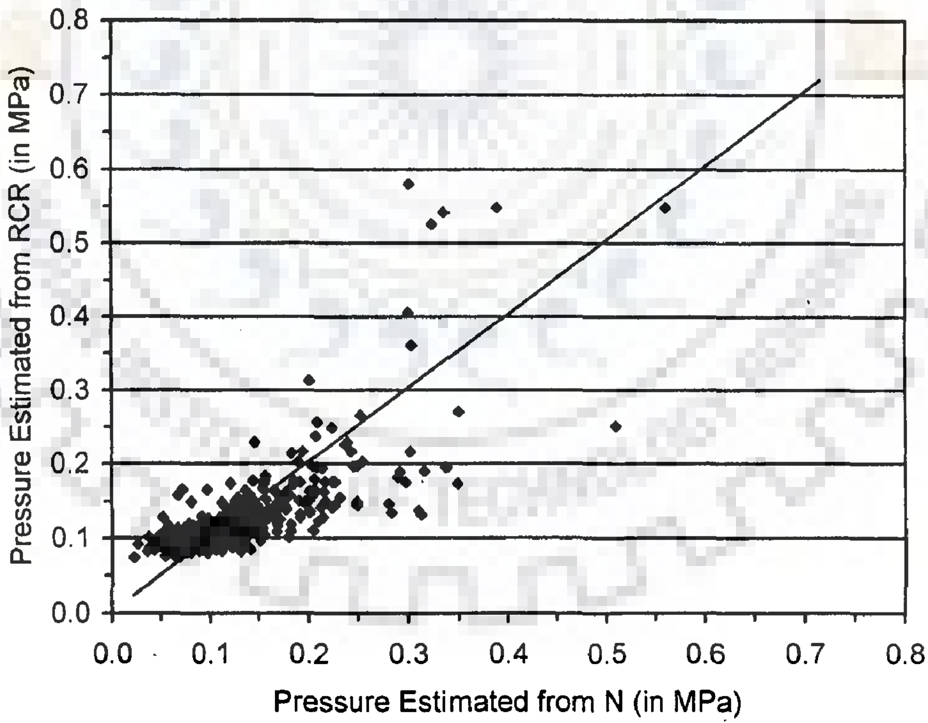
According to Barton et al. (1974), the chart between the index Q and the equivalent dimension of an excavation determines the appropriate support measures. This equivalent dimension, which is a function of both the size and the purpose of the excavation, is obtained by a parameter called ESR. Thus,

$$\text{Equivalent dimension} = \frac{\text{Excavation span, diameter or height (m)}}{\text{ESR}} \tag{6.18}$$

The ESR is related to the use for which the excavation is intended and the degree of safety demanded. For hydropower water tunnels, the ESR is 1.6. Barton et al. (1974) suggest 38 support categories, which give estimates of permanent support in the form of shotcrete, rock bolts, wire mesh and CCA.



(a) pressure estimated from RCR (Goel, 1994)



(b) pressure estimated from RCR (using Eq. 6.17 proposed in this study)

Fig. 6.3 Comparison of pressure estimated from RCR and N for non-squeezing ground conditions

Since the early 1980s, wet mix SFRS together with rock bolts has been the main components of a permanent rock support in underground openings in Norway. Based on the experience, Grimstad and Barton (1993) suggested a different chart for design of support using SFRS as shown in Fig. V.1 (APPENDIX – V). This chart has been used in the present work for the design of support according to the Q-system.

Length of Rock Bolt

Length of rock bolt depends on the dimensions of the excavations. Length of rock bolt has been determined as given by Bension (1971) and described by Barton et al. (1974). Lengths used in the roof arch are usually related to the span while lengths used in the walls are usually related to the height of the excavations as follows:

Roof: $$l_{bolt} = 2 + \frac{0.15\,B}{ESR} \quad \text{m} \qquad (6.19)$$

Walls: $$l_{bolt} = 2 + \frac{0.15\,H_t}{ESR} \quad \text{m} \qquad (6.20)$$

By putting $B = H_t = 11.65$ m and ESR = 1.6 in Eqs. 6.19 and 6.20, length of rock bolts in roof as well as walls comes out as 3.1 m.

6.3.2 Estimation of Support Pressure

Barton et al. (1974, 1975) suggested the following empirical correlation for ultimate support pressure:

$$p_v = \frac{0.2}{J_r}.Q^{-1/3} \quad \text{MPa} \qquad (6.21)$$

$$p_h = \frac{0.2}{J_r}.Q_w^{-1/3} \quad \text{MPa} \tag{6.22}$$

When the number of joint sets falls below three, the degree of freedom for block movement is greatly reduced since three joint sets (or two plus random) is the limiting case for three-dimensional rock blocks. Therefore, Barton et al. (1974) suggested an improved version of Eqs. 6.21 and 6.22 by incorporating J_n as follows:

$$p_v = \frac{0.2}{J_r}.\frac{\sqrt{J_n}}{3}.Q^{-1/3} \quad \text{MPa} \tag{6.23}$$

$$p_h = \frac{0.2}{J_r}.\frac{\sqrt{J_n}}{3}.Q_w^{-1/3} \quad \text{MPa} \tag{6.24}$$

Barton et al. (1974) thought it appropriate, in view of more favourable position of excavated walls as opposed to roof, to consider a hypothetically increased "wall quality" which will be some function of the general rock mass quality Q for a given excavation. They recommended that a hypothetical "wall quality" equal to 5.0 Q be regarded as the maximum for use in the better qualities of rock mass when Q > 10.

In intermediate qualities, i.e. 0.1 < Q < 10, in which the wall pressure is of more consequence, a value of 2.5 Q should be used. In the worst qualities, i.e. Q < 0.1, where the wall pressure can be almost equal to the vertical pressure, a minimum value 1.0 Q should be used.

In Eqs.6.21 through 6.24, size of opening does not figure in the support pressure prediction. Hence, this supposition made by Barton et al. (1974) is in contrast to the rock load estimates of Terzaghi (1946) who suggested doubling the support pressures when doubling the tunnel span. In rocks of good quality, support pressure is observed to be independent of B due to the restrained dilatancy (Singh et al., 1992).

Bhasin and Grimstad (1996) suggested the following correlation for predicting support pressures in tunnels through poor rock masses (Q < 4):

$$p_v = \frac{0.04\,B}{J_r}.Q^{-1/3} \quad \text{MPa} \tag{6.25}$$

where, B is in m.

The correlation between rock mass quality and tunnel support pressure proposed by Barton et al. (1974) has proved generally rewarding except in cases of squeezing ground conditions (Singh, 1988). Therefore, Singh et al. (1992) suggested modifications by correlating the support pressure with the rock mass quality, the overburden, the tunnel closure and the time after excavation as follows:

$$p_v = \frac{0.2}{J_r}.Q^{-1/3}.f.f'.f'' \quad \text{MPa} \tag{6.26}$$

$$f = 1 + \frac{H - 320}{800} \geq 1 \tag{6.27}$$

$$f'' = \log_{10}(9.5\,t^{0.25}) \tag{6.28}$$

where, t is in months.

Value of f' can be obtained from Table 6.2 on the basis of design value of tunnel closure. Value of f' is 1.0 for non-squeezing condition.

Goel et al. (1995a) proposed empirical correlations for estimating support pressure for tunnel sections under non-squeezing and squeezing ground conditions using N:

<u>Non-squeezing ground condition:</u>

$$p_v = \frac{0.12H^{0.1}.a^{0.1}}{N^{0.33}} - 0.038 \quad \text{MPa} \tag{6.29}$$

Table 6.2 Correction Factor f' for Tunnel Closure (Singh et al., 1992)

S.No.	Rock Condition	Support System	Tunnel Closure (u_a/a), %	Correction Factor, f'
1.	Non-squeezing	-	< 1	1.0
2.	Squeezing	Very stiff	< 2	> 1.8
3.	-do-	Stiff	2 - 4	0.85
4.	-do-	Flexible	4 - 6	0.70
5.	-do-	Very flexible	6 - 8	1.15
6.	-do-	Extremely flexible	> 8	1.8

Squeezing ground condition:

$$p_v = \frac{f(N)}{30} \cdot 10^{\frac{H^{0.6} \cdot a^{0.1}}{50 \cdot N^{0.33}}} \quad \text{MPa} \quad \text{(for f(N), refer Table 6.1)} \qquad (6.30)$$

where, H and a are in m.

Figure 6.4 compares pressures estimated from Q with N for squeezing ground conditions. Pressures from Q have been estimated both before and after applying corrections following Eq. 6.26. It is clear from Fig. 6.4 that pressures from Q before corrections are on lower side as compared to N and an excellent correlation is obtained between Q and N, once corrections are applied in Q as suggested by Singh et al. (1992). Therefore, it may be concluded that support pressures estimated from Q after corrections as per Singh et al. (1992) and N are almost same for big tunnels in squeezing ground conditions.

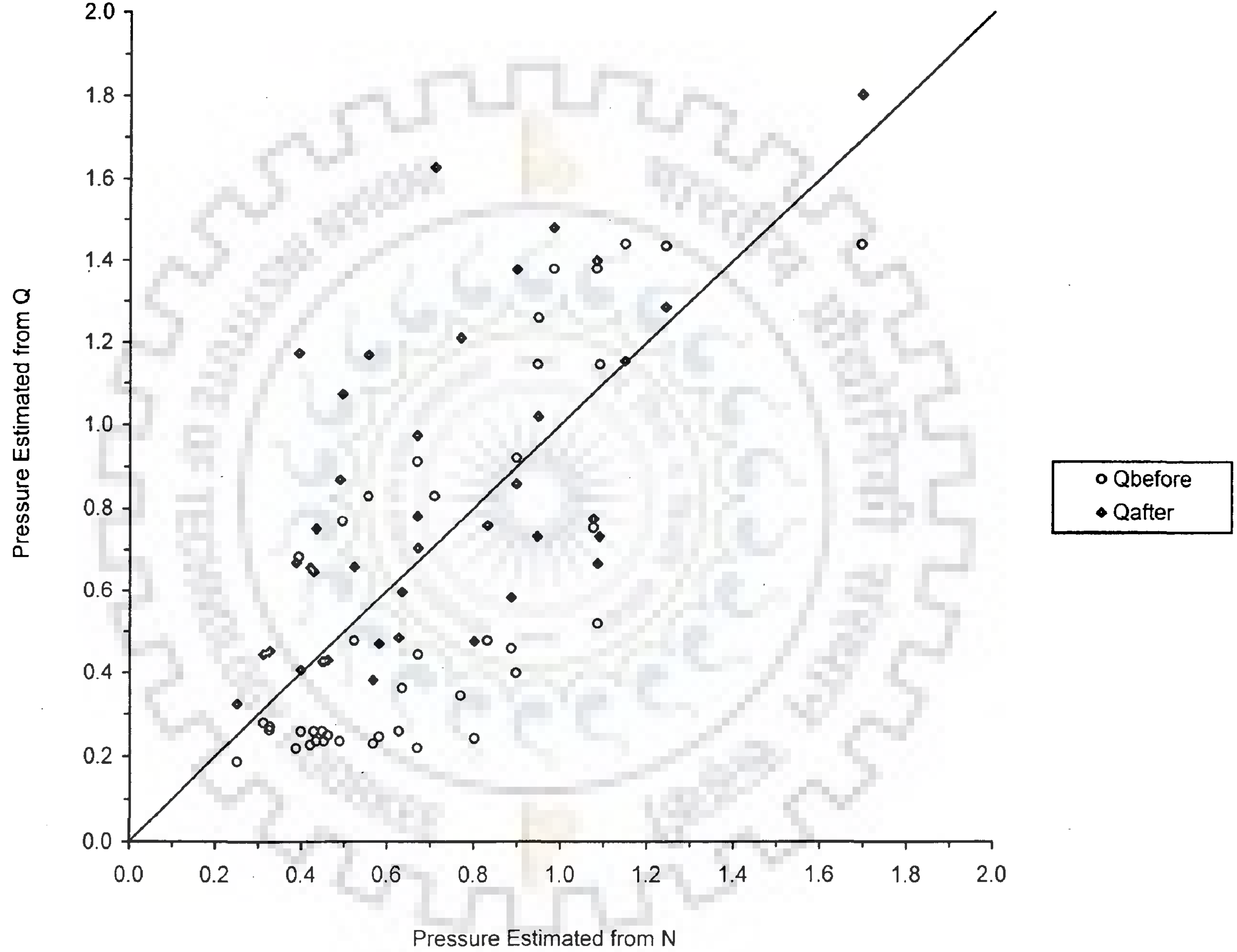


Fig. 6.4 Comparison of pressure estimated from Q before and after corrections and N

6.4 ROCK MASS INDEX (RMi) – SYSTEM

6.4.1 Design of Support

The number of blocks on the periphery of an underground opening will largely determine whether the surrounding ground will behave

(a) as a continuous, bulk material where the magnitude of the rock stresses is important; or

(b) as a blocky material, dominated by the individual blocks and the character of the joints.

This can be assessed from the ratio known as the continuity factor CF = tunnel diameter/block diameter. The RMi support method applies different calculations and support charts for continuous and blocky ground:

I Blocky ground

The stability in blocky (jointed) ground is mainly influenced by the block size and shape, the shear strength of the joints delineating the blocks, and the orientation of the same joints relative to the opening. The following two support parameters, which include all these features, are used in the support chart in Fig. VI.1 (APPENDIX – VI).

- The ground quality given as the ground condition factor

$$G_c = RMi.(SL.C) = q_c.JP.(SL.C) \tag{6.31}$$

- The scale factor expressed as the size ratio

$$S_r = CF.\frac{C_o}{N_j} = \frac{B}{D_b}.\frac{C_o}{N_j} \tag{6.32}$$

Ratings for SL, C, C_o and N_j can be found from APPENDIX - VIB. Ratings for C and N_j can also be computed from Eqs. 6.33 and 6.34 respectively:

$$C = 5 - 4\cos\delta \tag{6.33}$$

$$N_j = \frac{3}{n_j} \tag{6.34}$$

In cases where a seam or filled joint (with thickness $T_s < 1$ m) occurs in the tunnel, the following adjustment of the size ratio is made:

$$S_{rs} = S_r (1 + T_s) C_{os} \tag{6.35}$$

Support assessments for crushed zones with blocky materials (where CF = approximately 1 to 600) may be carried out using Fig. VI.1. In small and medium sized zones (thickness between 1 and 20 m approximately), stability is influenced by the interplay between the zone and the adjacent rock masses. Therefore, the stresses in such zones are generally lower than in the adjacent ground, which will reduce the effect of squeezing. In this case, G_c is the same as for blocky ground, while the size ratio for weakness zones is as follows:

$$S_r = \frac{T_z}{D_b} \cdot \frac{C_o}{N_j} \quad (\text{for } T_z < B) \tag{6.36}$$

$$S_r = \frac{B}{D_b} \cdot \frac{C_o}{N_j} \quad (\text{for } T_z > B) \tag{6.37}$$

Large zones ($T_z >$ approximately 20 m) will often behave like continuous ground, as there will be little or no arching effect.

II Continuous ground

Continuous ground occurs when CF < approximately 5 (massive rock), in which the properties of intact rock dominate, and when CF > approximately 100 (particulate or highly jointed rock), the ground behaves as a bulk material. In these types of ground the main influence on the behaviour in an underground opening comes from the stresses. Therefore, a competency factor (C_g = strength of rock mass/stresses acting) is used. It is expressed as follows:

Massive ground:

$$C_g = \frac{RMi}{\sigma_\theta} = \frac{q_c . f_\sigma}{\sigma_\theta} \approx 0.5 \frac{q_c}{\sigma_\theta} \tag{6.38}$$

Particulate ground:

$$C_g = \frac{RMi}{\sigma_\theta} = \frac{q_c.JP}{\sigma_\theta} \tag{6.39}$$

Competent ground occurs where $C_g > 1$; else the ground is overstressed (incompetent). C_g is applied in the ground support chart (Fig. VI.2).

Massive, competent ground is generally stable and does not need any support, except for some scaling work in drill and blast tunnels. Massive, incompetent (overstressed) ground, however, requires support because the time-dependent type of deformation and/or failures may take place.

Particulate materials (highly jointed rocks) require immediate support. Their initial behaviour is similar to that of blocky ground i.e. the support chart in Fig VI.1 can be used for CF = 1 to 600. In overstressed (incompetent) ground time-dependent squeezing will, in addition to the initial instability, take place.

Length of Rock Bolt

Palmstrom (2000a) suggested following expressions for calculating the length of rock bolts from tunnel dimensions and the block size:

Roof: $$l_{bolt} = 1.4 + 0.16\,B\left(1 + \frac{0.1}{D_b}\right) \quad m \tag{6.40}$$

Walls: $$l_{bolt} = 1.4 + 0.08\,(B + H_t)\left(1 + \frac{0.1}{D_b}\right) \quad m \tag{6.41}$$

where, B, H_t and D_b are in m.

Putting B = H_t = 11.65 m in Eqs. 6.40 and 6.41 and simplifying:

Roof: $$l_{bolt} = 3.264 + 0.016 * CF \tag{6.42}$$

Walls: $$l_{bolt} = 2.798 + 0.012 * CF \tag{6.43}$$

Eqs. 6.42 and 6.43 have been used in the present work for computing lengths of rock bolts in roof and walls of the tunnel following the RMi-system.

6.5 SUPPORT PRESSURES AND SUPPORT SYSTEMS

Support pressures have been estimated from RSR (Eq. 6.6), RMR (Eq. 6.12), RCR (Eqs. 6.14 and 6.15), Q (Eqs. 6.23 and 6.24) and N (Eqs. 6.29 and 6.30) and support systems have been selected as recommended by RSR, RMR, Q and RMi systems. Support pressures and support systems for typical sections have been presented in Table 5.21 and Table 6.3.

6.6 DISCUSSION OF RESULTS

6.6.1 Non-Squeezing Ground Conditions

Number of tunnel sections (in percentage) for different pressure ranges and different classification ranges of RSR, RMR, Q, N and RCR systems for non-squeezing ground conditions have been depicted in Figs. 6.5 through 6.9 and compared in Fig. 6.10. From these figures following observations are made:

(a) According to RSR (Fig. 6.5), pressure varies from 0.065 MPa to 0.484 MPa corresponding to RSR-value of 76 and 25 respectively and in majority of the sections (about 92 percent) pressure is less than 0.34 MPa. From Fig. 6.5, it is clear that up to 0.34 MPa pressure, number of sections are almost uniformly distributed in each pressure range with a maximum of 20.3 percent sections lying in range 0.18 - 0.22 MPa (RSR value range 50 - 40).

(b) According to RMR (Fig. 6.6), pressure varies from 0.06 MPa to 0.26 MPa corresponding to RMR-value of 81 and 17 respectively. It is interesting to note that majority of the sections (about 81 percent) are lying in range 0.10 - 0.18 MPa (RMR value range 70 - 40) with a maximum of 48.5 percent sections in range 0.10 - 0.14 MPa (RMR value range 70 - 50).

Table 6.3 Support Recommendations by Classification Systems

S. No.	Chainage (in m)	RSR	RMR							Q				RMi					
		Spacing of ISMB 250 (mm)	Roof				Walls			Roof		Walls		Roof			Walls		
			Rock Bolt		Shotcrete Thickness (in mm)	Spacing of ISMB 250 (in mm)	Rock Bolt		Shotcrete Thickness (in mm)	SFRS Thickness (in mm)	Rockbolt Spacing (in m)	SFRS Thickness (in mm)	Rockbolt Spacing (in m)	SFRS Thickness (in mm)	Rockbolt		SFRS Thickness (in mm)	Rockbolt	
			Length (in m)	Spacing (in m)			Length (in m)	Spacing (in m)							Length (in m)	Spacing (in m)		Length (in m)	Spacing (in m)
1	0-50	3050	3.00	2.40	50					100	2.45	70	2.5	55	3.50	1.6	50	3.00	2
2	966-987	650	3.90	1.55	95				30	135	2.05	135	2.05	200	4.00	1	120	3.40	1.25
3	1375-1587	1450	3.00	2.05	50					85	2.5	50	2.5	60	3.50	1.6	40	3.00	3
4	2057-2112	700	3.30	1.85	65				30	80	2.5	50 (P)	1	90	3.80	1.3	80	3.20	1.35
5	2155-2202	450	4.75	1.15	140	1350	4.75	1.15	100	180	1.65	180	1.65	SDSC			SDSC		
6	3631-3644	650	3.45	1.80	75				30	150	1.7	150	1.7	SDSC			155	3.90	1.1
7	6208-6338	1750	3.00	3.00	50				30	Unsupported		Unsupported			3.40	2.5		3.90	spot
8	9687-9697	1050	3.05	2.00	50				30	120	2.2	100	2.45	150	3.80	1.2	100	3.20	1.25
9	11435-11446	950	3.25	1.90	65				30	90	2.5	60	2.5	85	3.80	1.35	85	3.20	1.35
10	11634-11643	1250	3.00	2.10	50					100	2.45	65	2.5	140	4.20	1.2	100	3.50	1.25
11	11932-12044	2250	3.00	2.05	50					100	2.45	70	2.5	60	3.60	1.5	60	3.10	1.5
12	12044-12070	950	3.20	1.90	60				30	90	2.5	60	2.5	100	3.90	1.25	100	3.30	1.25
13	12087-12223	950	3.25	1.90	65				30	100	2.45	70	2.5	85	3.70	1.35	60	3.20	1.5
14	12359-12428	1750	3.00	2.00	50				30	90	2.5	65	2.5	80	3.70	1.35	60	3.20	1.5
15	12544-12608	1250	3.00	2.00	50				30	100	2.45	65	2.5	125	4.00	1.2	90	3.30	1.3
16	12629-12668	1650	3.00	2.15	50					95	2.45	65	2.5	95	3.80	1.3	70	3.20	1.4
17	16036-16046	1750	3.00	2.15	50					90	2.5	60	2.5	90	3.60	1.3	60	3.00	1.6
18	16840-16900	3050								90	2.5	65	2.5	60	3.50	1.5	50	3.00	2
19	16900-17000	1650	3.10	1.95	55				30	95	2.45	65	2.5	100	3.70	1.25	60	3.10	1.5
20	17040-17070	1100	3.10	2.00	55				30	60	2.5	45 (P)	1.3	80	3.70	1.4	60	3.20	1.7
21	18387-18424	950	3.00	2.20	50					90	2.5	60	2.5	150	4.00	1.2	100	3.40	1.25
22	18535-18687	850	3.00	2.10	50					Unsupported		Unsupported		SDSC			SDSC		
23	19015-19054	650	3.20	1.90	60				30	90	2.5	65	2.5	SDSC			SDSC		
24	19054-19103	750	3.25	1.90	65				30	90	2.5	60	2.5	SDSC			SDSC		
25	19189-19700	950	3.00	2.10	50					85	2.5	50	2.5	150	4.50	1.15	125	3.70	1.25
26	21230-21240	800	4.45	1.30	125	1550	4.45	1.30	100	150	1.7	150	1.7	200	3.80	1	200	3.20	1
27	21240-21247	1000	4.20	1.40	110	1750	4.20	1.40	100	120	2.2	100	2.45	150	3.80	1.2	150	3.20	1.2
28	22537-22648	600	3.40	1.80	70				30	135	2	110	2.4	SDSC			200	3.40	1
29	22795-22848	500	3.55	1.75	75				30	100	2.45	70	2.5	SDSC			SDSC		
30	23627-23785	600	3.60	1.75	80				30	110	2.35	90	2.5	180	4.10	1.15	100	3.40	1.25
31	23894-23901	600	3.55	1.75	80				30	110	2.35	90	2.5	180	3.80	1.15	125	3.30	1.2
32	25043-25080	700	3.95	1.55	100				30	85	2.5	50	2.5	125	3.90	1.2	100	3.30	1.25
33	25951-25980	500	4.00	1.50	100	1900	4.00	1.50	100	90	2.5	65	2.5	SDSC			SDSC		
34	26108-26112	350	5.30	1.00	165	930	5.30	1.00	150	170	1.65	170	1.65	SDSC			SDSC		
35	26120-26126	650	4.85	1.10	145	1250	4.85	1.10	100	250	1.5	250	1.5	SDSC			SDSC		
36	26361-26398	600	5.25	1.00	165	950	5.25	1.00	150	240	1.5	240	1.5	SDSC			SDSC		
37	26415-26478	600	5.25	1.00	165	950	5.25	1.00	150	170	1.65	170	1.65	SDSC			SDSC		
38	26478-26506	600	5.40	1.00	170	850	5.40	1.00	150	230	1.5	230	1.5	SDSC			SDSC		
39	26608-26620	600	5.25	1.00	165	950	5.25	1.00	150	C C A		C C A		SDSC			SDSC		
40	26946-27346	1000	3.00	2.05	50					90	2.5	65	2.5	140	3.70	1.2	100	3.10	1.3

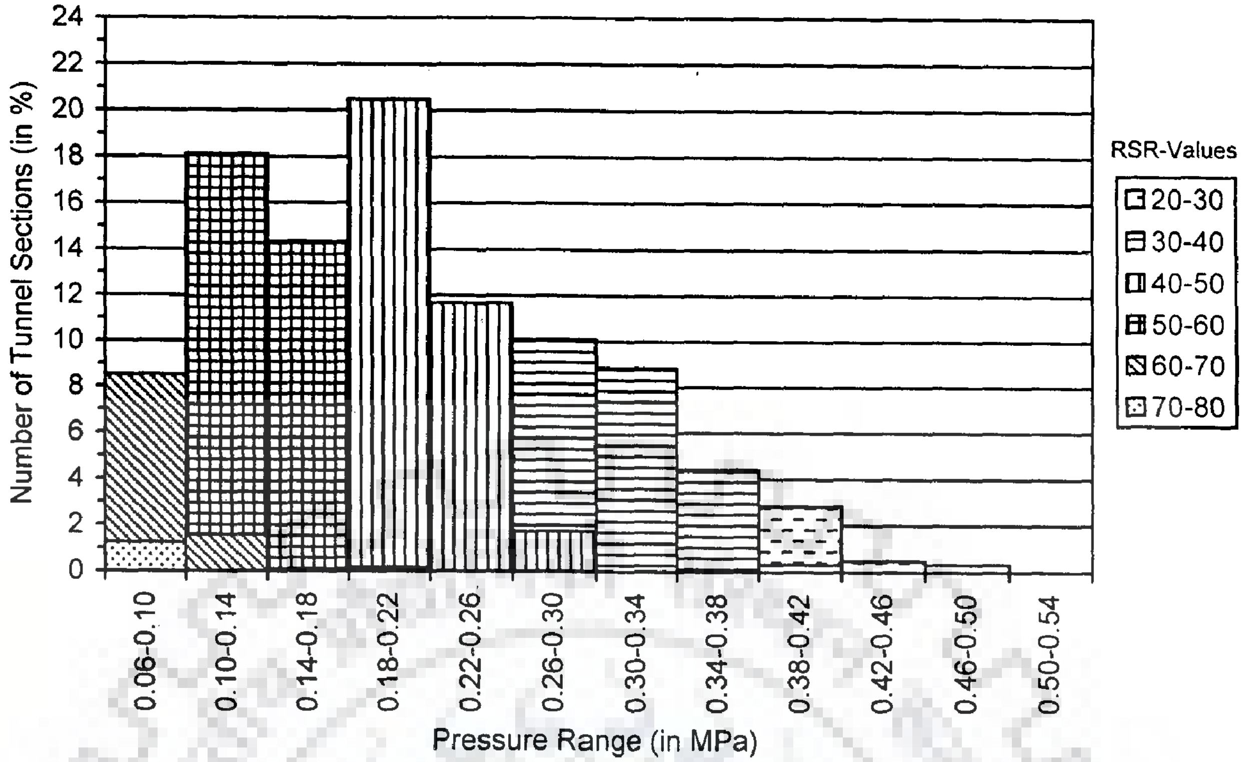


Fig. 6.5 Number of tunnel sections in different pressure ranges for RSR-values in non-squeezing ground conditions

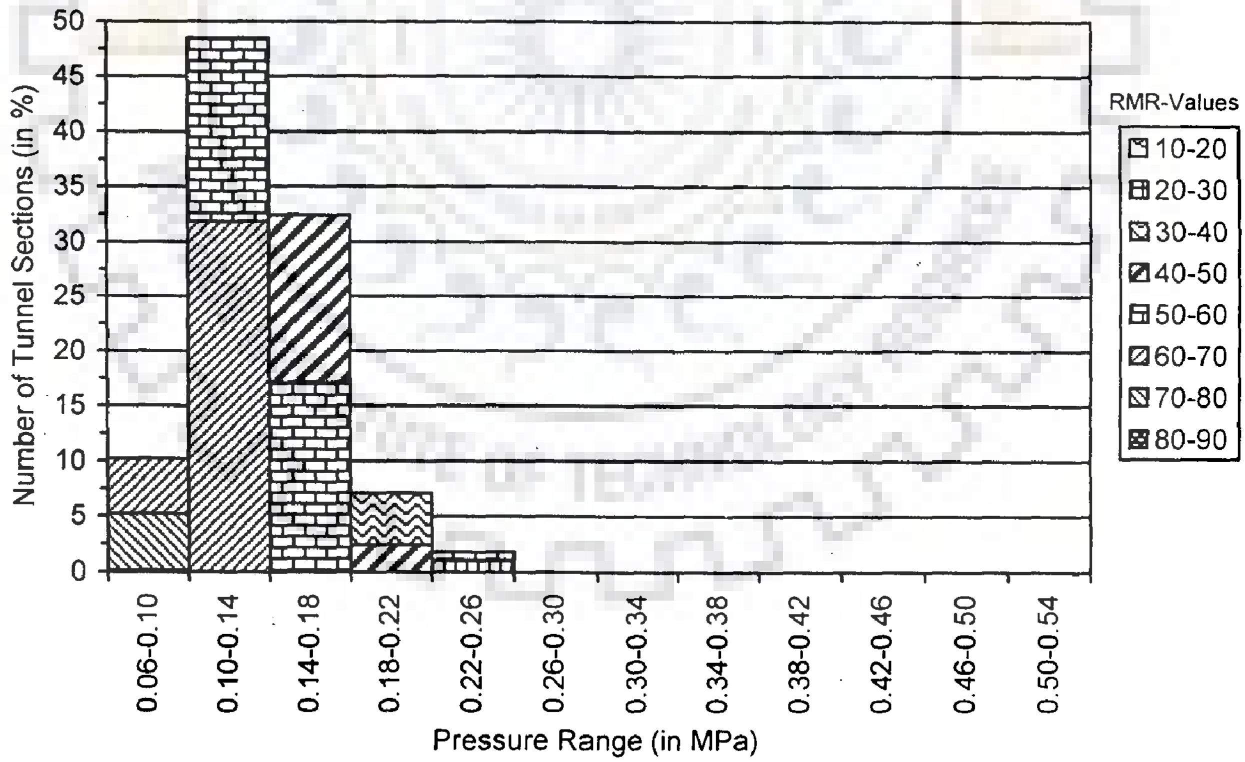


Fig. 6.6 Number of tunnel sections in different pressure ranges for RMR-values in non-squeezing ground conditions

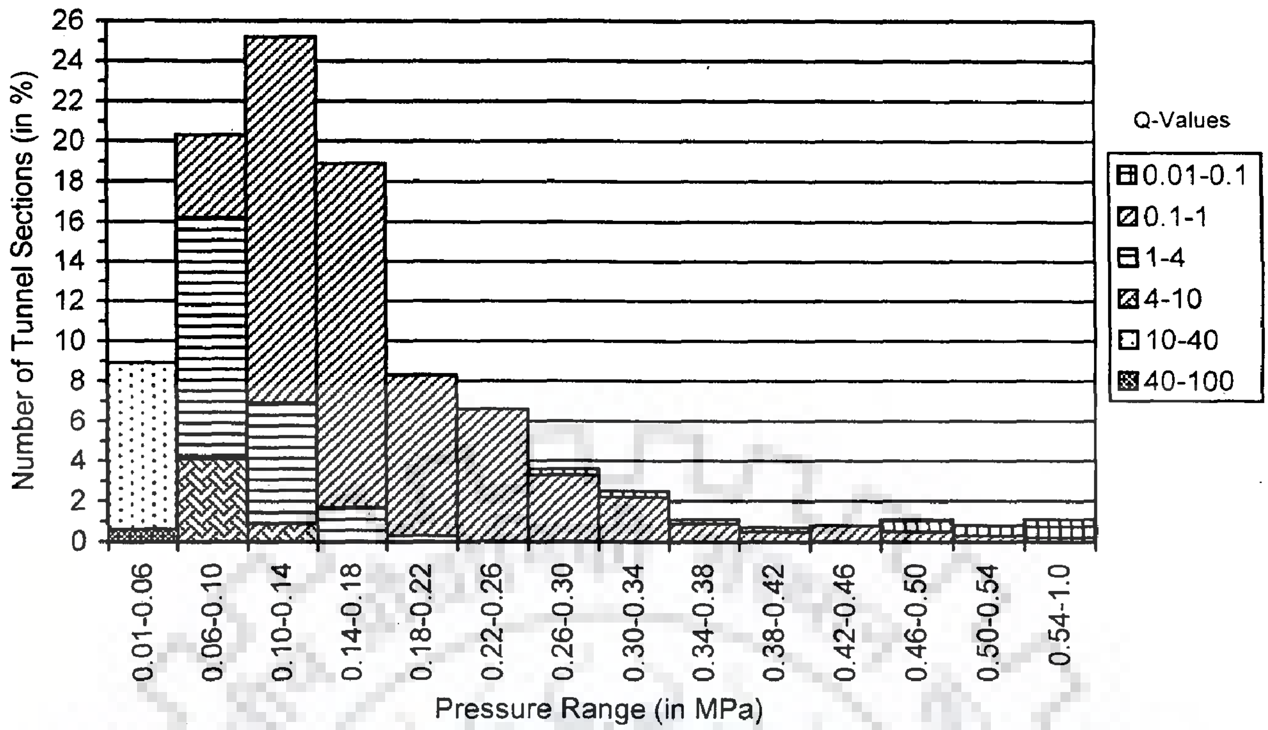


Fig. 6.7 Number of tunnel sections in different pressure ranges for Q-values in non-squeezing ground conditions

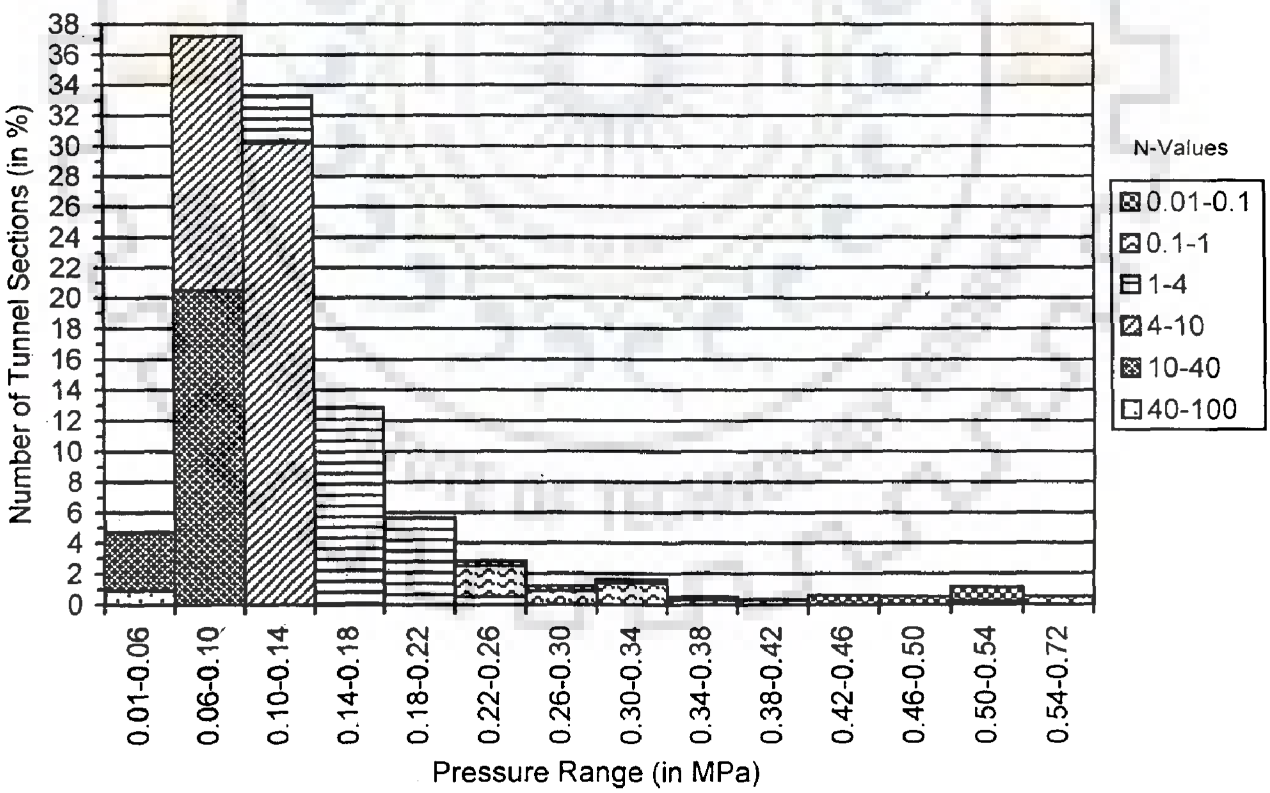


Fig. 6.8 Number of tunnel sections in different pressure ranges for N-values in non-squeezing ground conditions

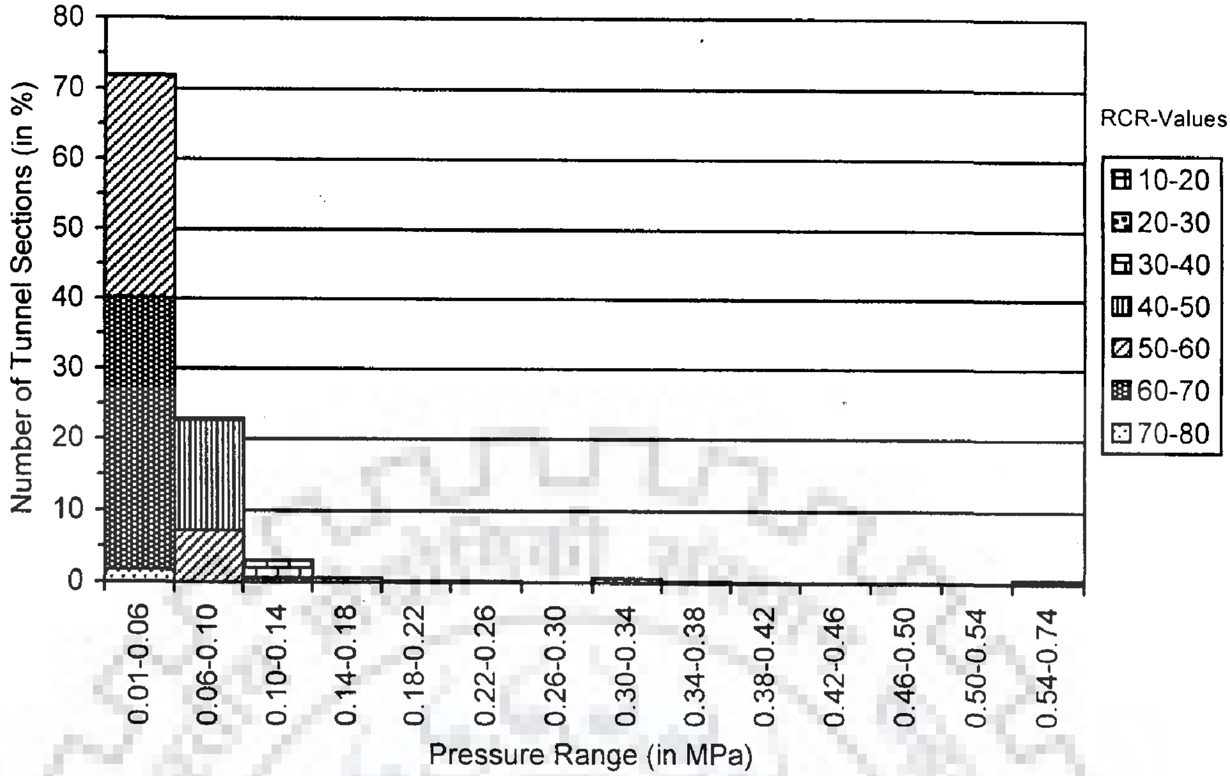


Fig. 6.9 Number of tunnel sections in different pressure ranges for RCR-values in non-squeezing ground conditions

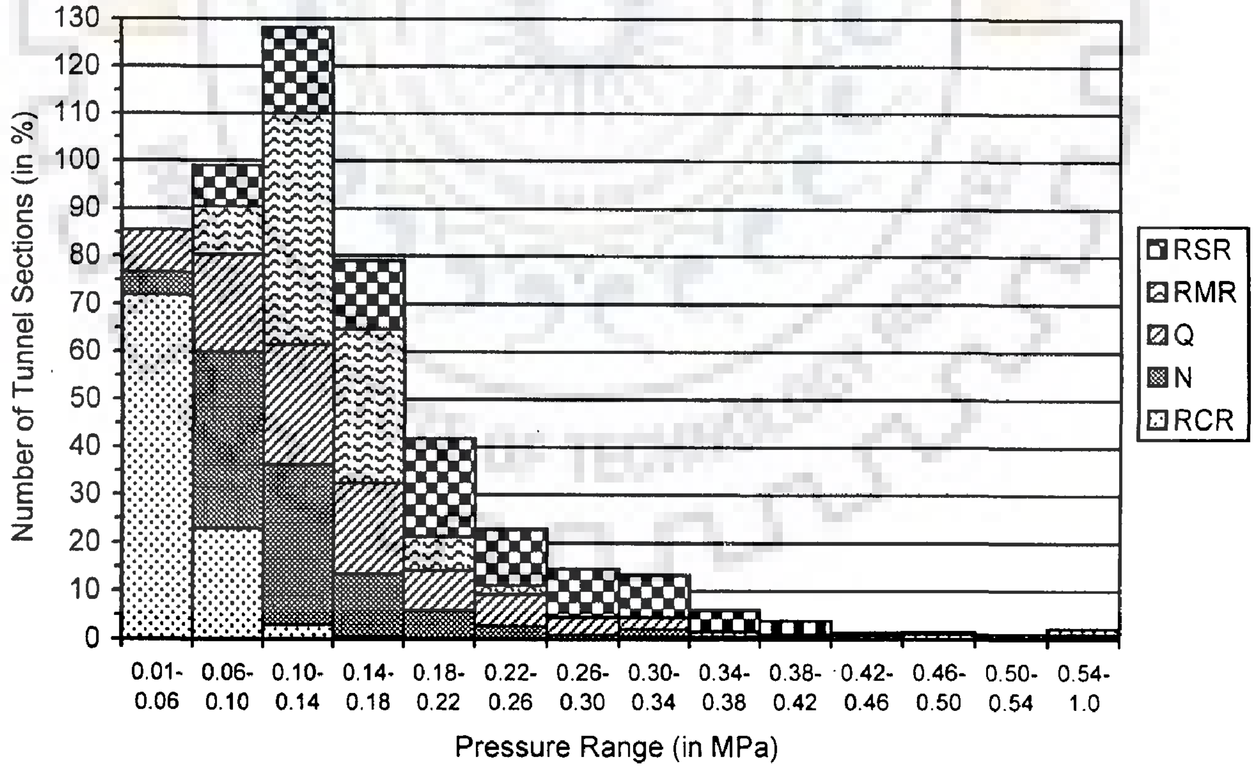


Fig. 6.10 Percent tunnel sections in different pressure ranges for all classification systems in non-squeezing ground conditions

(c) According to Q (Fig. 6.7), pressure varies from 0.01 MPa to 1.0 MPa corresponding to Q-value of 58 and 0.017 respectively and in majority of the sections (about 94 percent) pressure is less than 0.34 MPa. From Fig. 6.7, it is clear that between 0.06 MPa and 0.18 MPa, number of sections are almost equally distributed in the three pressure ranges totaling to about 65 percent with a maximum of 25.2 percent sections lying in range 0.10 - 0.14 MPa (Q value range 10 - 0.1). An interesting aspect of this figure is that for Q range of 0.1 - 1, pressure spreads in all the pressure ranges excepting 0.01 - 0.06 MPa which is an indicator of the fact that there are other parameters too apart from the Q-value which influence the support pressure following the Q-system.

(d) According to N (Fig. 6.8), pressure varies from 0.02 MPa to 0.72 MPa corresponding to N-value of 90 and 0.042 respectively and in majority of the sections (about 94 percent) pressure is less than 0.22 MPa. From Fig. 6.8, it is clear that as high as 70.5 percent sections are lying between 0.06 MPa and 0.14 MPa with a maximum of 37.2 percent sections lying in range 0.06 - 0.10 MPa (N value range 40 - 4).

(e) According to RCR (Fig. 6.9), pressure varies from 0.028 MPa to 0.728 MPa corresponding to RCR-value of 77 and 15 respectively. It is interesting to note that almost all the sections (about 95 percent) lie below 0.10 MPa and 72 percent below 0.06 MPa.

Figure 6.10 shows comparison of number of sections in different pressure ranges for all classification systems. From this figure following observations are made:

(i) In range 0.01 - 0.06 MPa, 71.9 percent sections for RCR are lying whereas others are very few. This indicates that the RCR grossly underestimates the support pressures.

(ii) In range 0.06 - 0.10 MPa, N has got the largest share with 37.2 percent followed by those of RCR, Q, RMR and RSR. The lowest share (8.5 percent) of the RSR-system and equally low share (10.2 percent) of the RMR-system in this range indicate that pressures estimated from RSR and RMR are of higher order. The higher percentage of the N (37.2 percent) compared to Q (20.3 percent) is attributed to the fact that in a number of sections, Q-values for 'competent rock, rock stress problem' category have been computed by adopting SRF as per Barton's table, which is not appropriate to use for moderately jointed rocks. SRF in fact should be very low for moderately jointed rocks in Himalaya as has been established in the present study (section 9.1.1) thereby increasing Q-values and decreasing support pressures. Therefore, it is not difficult to perceive that once proposed new SRF-values are adopted, number of sections for N and Q would be almost same in this range.

(iii) In range 0.10 - 0.14 MPa, RMR has got the highest share of 48.5 percent followed by those of N, Q and RSR. It is a clear indication of the fact that number of sections increases in higher pressure ranges for RMR. Therefore, conclusion of Goel and Jethwa (1991) that for tunnel diameter > 9 m, pressure estimation by Unal (1983) is over-safe for non-squeezing ground conditions, has been corroborated in the present study. Slightly higher percentage of N (33.3 percent) compared to that of Q (25.2 percent) is attributed to the reason cited in (ii) above.

(iv) In range 0.14 - 0.18 MPa also, RMR has got the highest share of 32.4 percent followed by those of Q, N and RSR. This again indicates that for quite a good number of sections, pressures estimated by Unal (1983) are very high for tunnels of larger diameter. It is interesting to note that in this range, share of Q (18.9 percent) has now slightly increased in comparison to that of N (12.9 percent) as against the previous ranges. It is quite obvious due to the reason cited in (ii) above that in higher pressure ranges contribution of lower Q-values would be more as compared to that of N.

(v) In range 0.18 - 0.22 MPa, effects of RSR begin to show in the form of there being the largest percentage (20.5 percent) of sections in comparison to the others. Percentage of that of Q (8.3 percent) is again slightly higher compared to that of N (5.7 percent), which is quite understandable.

(vi) In the remaining ranges from 0.22 MPa onwards, largest share of RSR is clearly visible indicating again that pressure estimation from RSR is highest among all the classification systems.

Table 6.4 compares classification systems in estimation of support pressure in non-squeezing ground conditions. Two systems have been taken at a time and number of sections falling in each group of inequality has been indicated, on the basis of which a conclusion has been drawn in the last column of Table 6.4.

Summarizing above and from Table 6.4 a general trend, which emerges for pressure estimation in non-squeezing ground conditions, may be given as follows:

Support Pressure (Non-Squeezing): $\mathbf{P_{RCR} < P_Q \leq P_N < P_{RMR} < P_{RSR}}$

Support pressures have also been estimated from Q (with proposed new SRF-values) and the same have been compared with others as indicated in Table 6.5.

Table 6.5 clearly establishes that $P_Q < P_{RSR}$ and $P_Q < P_{RMR}$. In Table 6.4 sections with $P_Q > P_N$ are 55 percent which reduce to 8 percent only after considering proposed new SRF and sections with $P_Q < P_N$ are as high as 70 percent. But considering maximum deviation of 0.07 MPa only, it may be concluded that $P_Q \leq P_N$. In Table 6.5, $P_{RCR} < P_Q$ even after considering proposed new SRF.

Table 6.5, therefore, clearly establishes that the general trend presented earlier is quite appropriate.

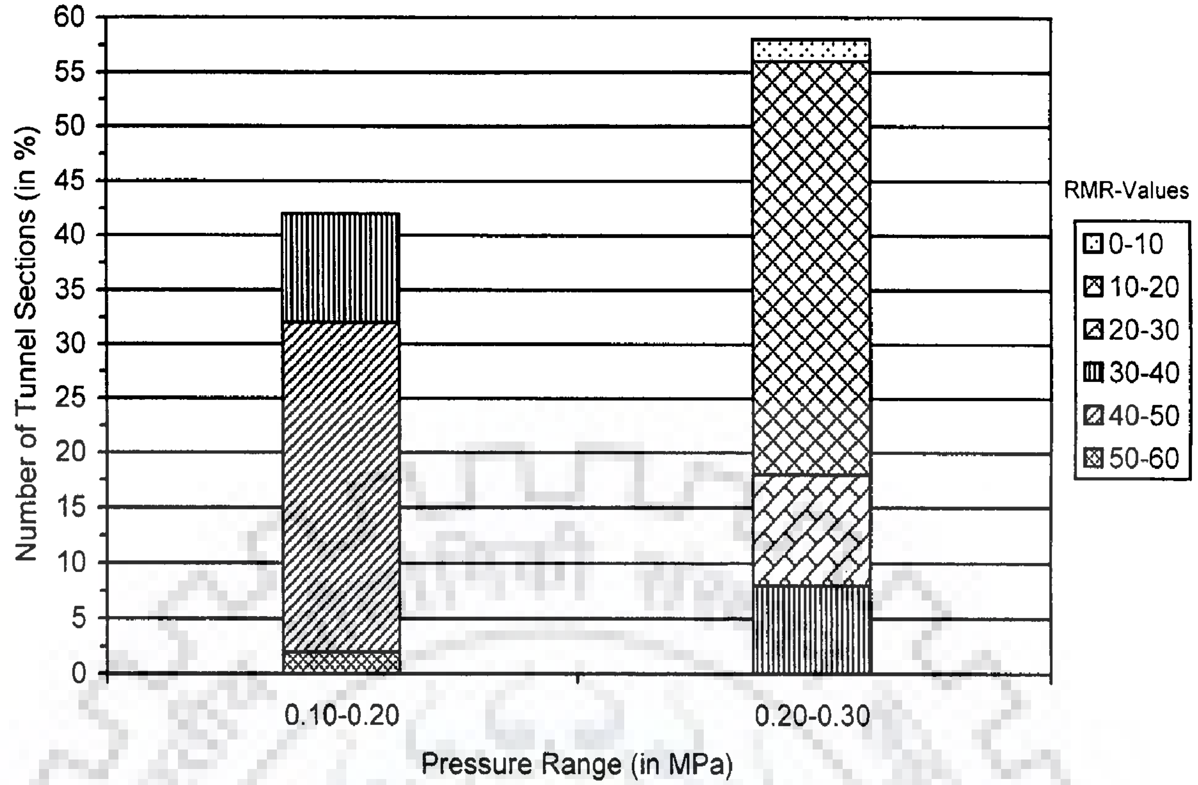


Fig. 6.11 Number of tunnel sections in different pressure ranges for RMR-values in squeezing ground conditions

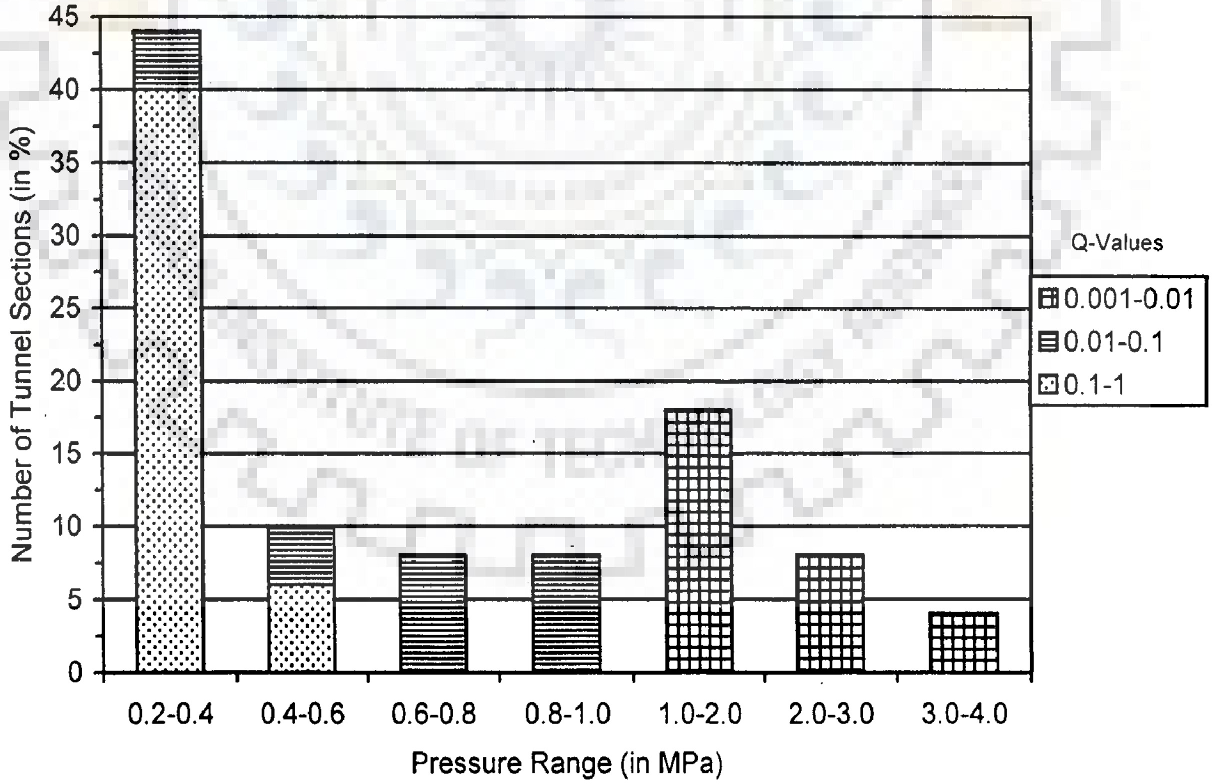


Fig. 6.12 Number of tunnel sections in different pressure ranges for Q-values in squeezing ground conditions

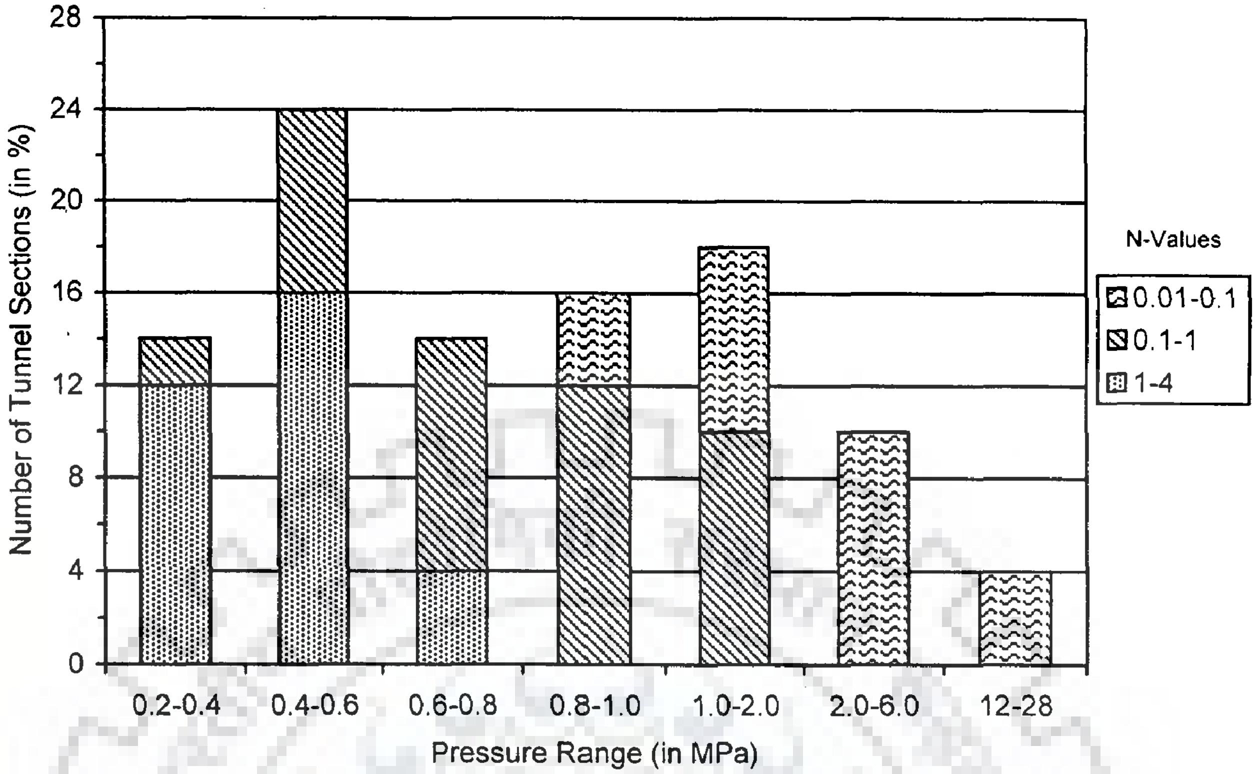


Fig. 6.13 Number of tunnel sections in different pressure ranges for N-values in squeezing ground conditions

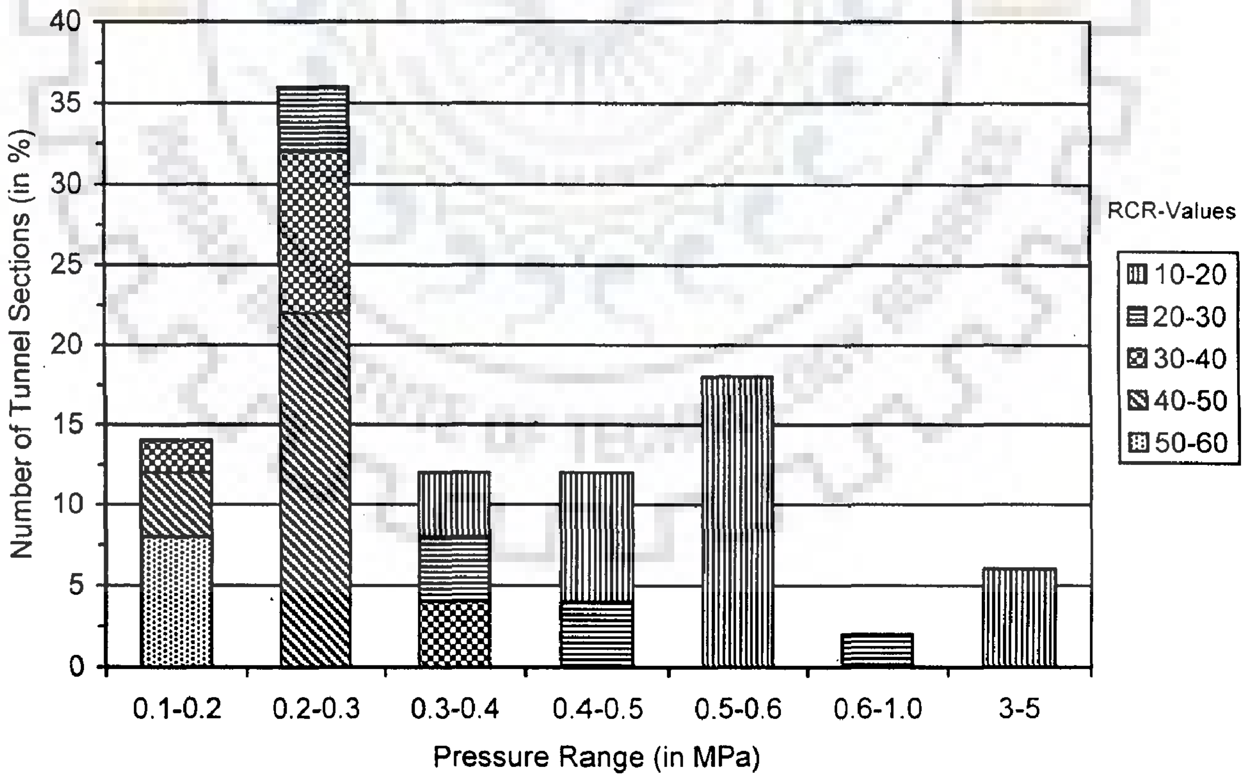


Fig. 6.14 Number of tunnel sections in different pressure ranges for RCR-values in squeezing ground conditions

fact that pressures estimated by Unal (1983) for squeezing ground conditions are very low and unsafe.

(b) According to Q (Fig. 6.12), pressure varies from 0.2 MPa to 3.4 MPa corresponding to Q-value of 0.667 and 0.001 respectively. From Fig. 6.12, it is clear that only about 12 percent sections lie above 2.0 MPa and about 44 percent sections are lying in range 0.2 - 0.4 MPa alone. Sections are almost uniformly distributed in all the ranges except the first one.

(c) According to N (Fig. 6.13), pressure varies from 0.25 MPa to as high as 27.5 MPa corresponding to N-value of 3.33 and 0.013 respectively. From Fig. 6.13, it is clear that only about 14 percent sections lie above 2.0 MPa and sections are almost evenly distributed in all the pressure ranges. A comparison of P_N (Fig. 6.13) with P_Q (Fig. 6.12) indicates that in all ranges barring 0.2 - 0.4 MPa, percentage sections for P_N is higher than for P_Q. From this it may be concluded that Q underestimates pressures compared to N in squeezing ground conditions, as corrections are not applied in Q.

(d) According to RCR (Fig. 6.14), pressure varies from 0.17 MPa to 3.4 MPa corresponding to RCR-value of 54 and 11 respectively. A maximum of about 36 percent sections lie in range 0.2 - 0.3 MPa and in rest of the ranges, sections are almost uniformly distributed. From Fig. 6.14, it is clear that about 92 percent sections lie below 0.6 MPa indicating that pressures estimated from RCR are lower in comparison with P_Q and P_N but higher than P_{RMR}.

Figures 6.15, 6.16 and 6.17 show comparison of support pressures estimated from RMR, RCR, Q and N for mild, moderate and high squeezing ground conditions respectively. Capacities of supports installed by the Project Authorities, which have been estimated from modified semi - empirical method of support design discussed in CHAPTER – 7, have also been included in these figures. From Figs. 6.15, 6.16 and 6.17, it is clear that RMR gives least and RCR second least pressures in squeezing ground conditions. N gives highest pressures in mild and high squeezing

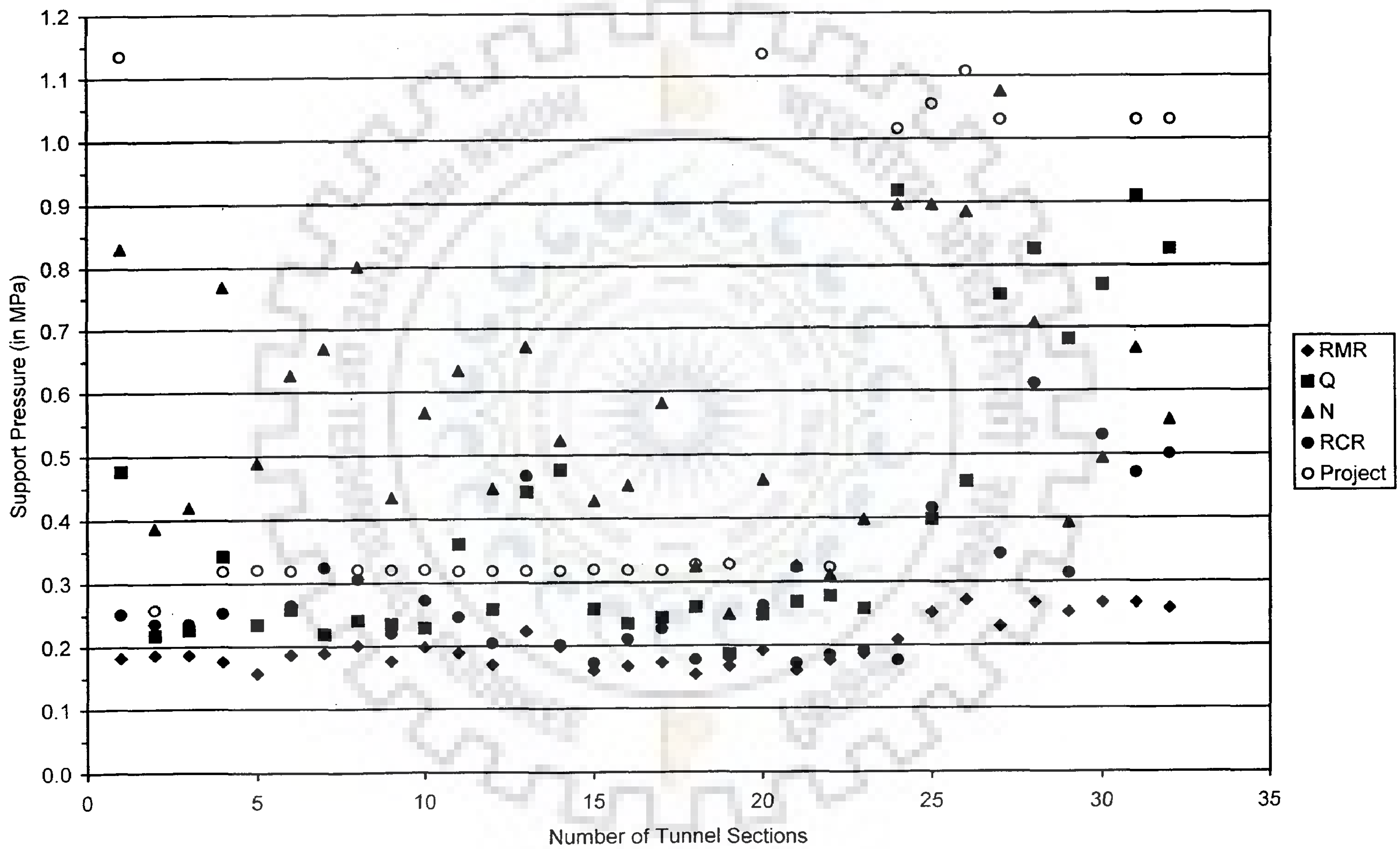


Fig. 6.15 Comparison of support pressure for mild squeezing ground conditions

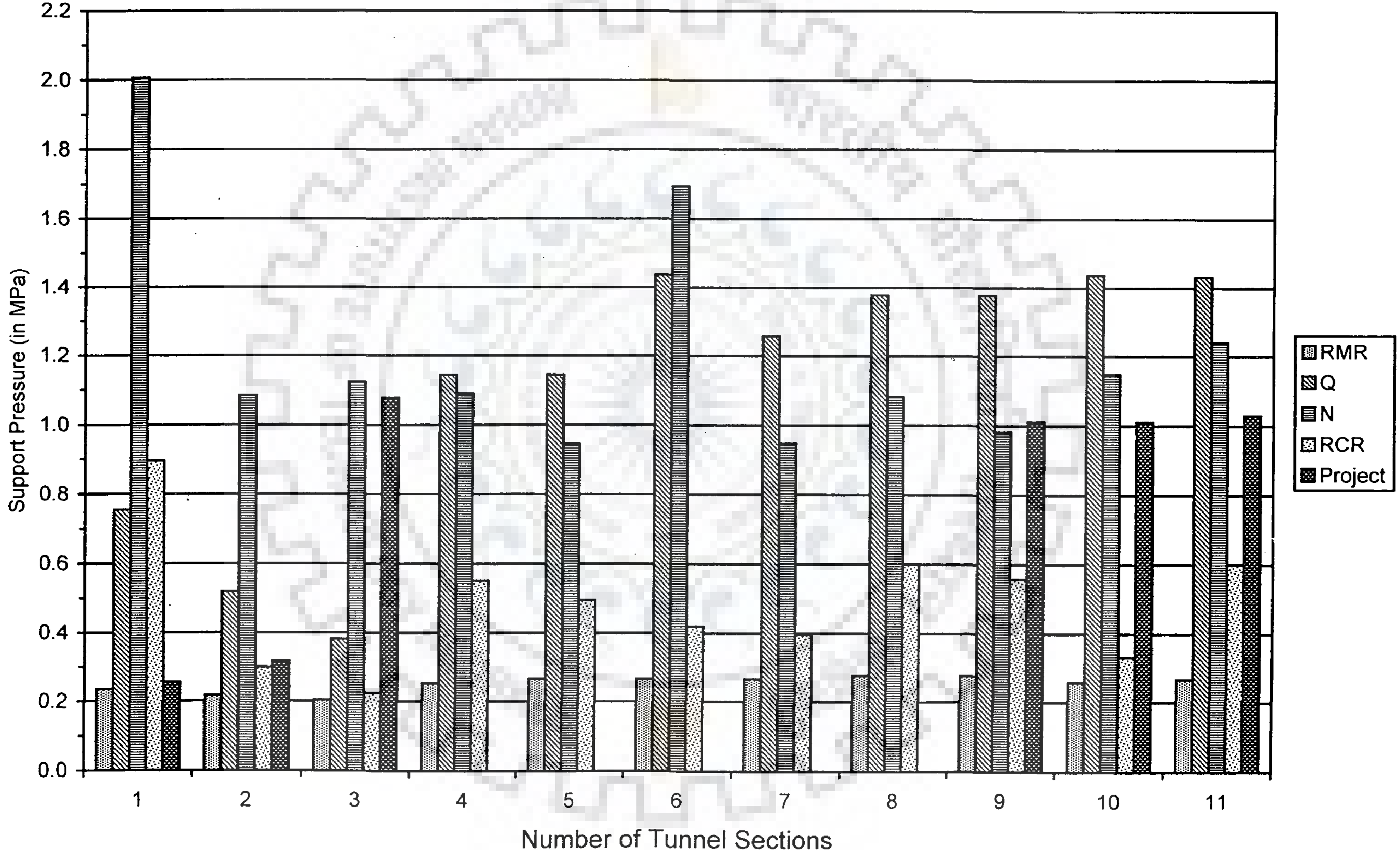


Fig. 6.16 Comparison of support pressure for moderate squeezing ground conditions

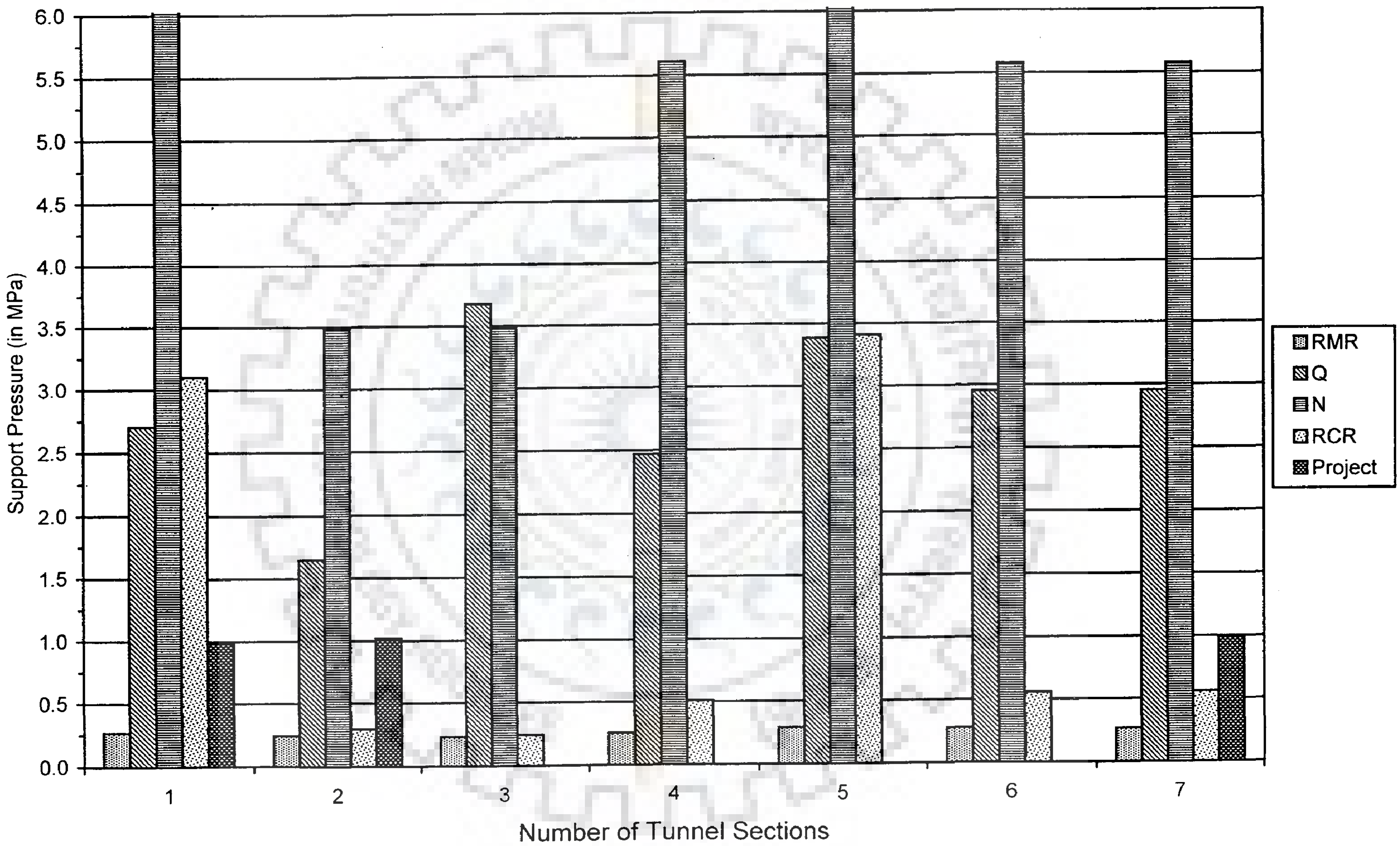


Fig. 6.17 Comparison of support pressure for high squeezing ground conditions

conditions whereas Q gives highest pressure in moderate squeezing conditions with minor exceptions.

Table 6.6 compares classification systems in estimation of support pressure in squeezing ground conditions involving a total of 50 sections. Two systems have been taken at a time and number of sections falling in each group of inequality has been indicated, on the basis of which a conclusion has been drawn in the last column of Table 6.6.

Table 6.6 Comparisons of Classification Systems in Estimation of Pressure in Squeezing Ground Conditions

S. No.	Classification Systems	Inequality	Number of Sections (in percent)	Conclusion
1.	Q and N	$P_Q < P_N$	72	$P_Q < P_N$
		$P_Q > P_N$	28	
2.	RCR and N	$P_{RCR} < P_N$	96	$P_{RCR} < P_N$
		$P_{RCR} > P_N$	4	
3.	RMR and N	$P_{RMR} < P_N$	100	$P_{RMR} < P_N$
		$P_{RMR} > P_N$	0	
4.	RCR and Q	$P_{RCR} < P_Q$	74	$P_{RCR} < P_Q$
		$P_{RCR} > P_Q$	26	
5.	RMR and Q	$P_{RMR} < P_Q$	100	$P_{RMR} < P_Q$
		$P_{RMR} > P_Q$	0	
6.	RMR and RCR	$P_{RMR} < P_{RCR}$	100	$P_{RMR} < P_{RCR}$
		$P_{RMR} > P_{RCR}$	0	

Summarizing above and from Table 6.6, a general trend, which emerges for pressure estimation in squeezing ground conditions, may be given as follows:

Support Pressure (Squeezing): $\mathbf{P_{RMR} < P_{RCR} < P_Q < P_N}$

COMPARISON OF SUPPORTS WITH SEMI - EMPIRICAL METHOD

The support systems recommended by RSR, RMR, Q and RMi systems have been discussed in CHAPTER – 6. The supports recommended by the RSR-system are in the form of steel ribs, which are primarily used for the roof supports. Therefore, no wall supports have been listed in Table 6.3. On the other hand, rest of the classification systems i.e. RMR, Q and RMi recommended supports for roof as well as walls. Table 6.3 indicates that all the four systems have recommended different types of supports viz. (i) RSR steel ribs, (ii) RMR shotcrete, rock bolts and steel ribs, (iii) Q SFRS/shotcrete, rock bolts, RRS and CCA and (iv) RMi SFRS and rock bolts. It is difficult to compare four classification systems from support point of view since different types of supports have been recommended by them. Partially, RSR and RMR may be compared approximately by way of spacing of ribs recommended by the two systems. RMR suggested shotcrete and rock bolts also. Similarly, Q and RMi may also be compared up to some extent by thickness of SFRS and spacing of rock bolts. But both the systems have a great deal of dissimilarities, such as RMi unlike Q suggests specially designed shotcrete or concrete (SDSC) or immediate shotcrete (ISC) also, which is difficult to quantify. Moreover, only RSR and Q have given formulation for estimation of support pressure on the basis of which a comparison could have been made among all the systems.

In view of above, a need was felt to estimate capacity of the supports recommended by classification systems so that a comparison may be made easily.

Singh et al. (1995a, b) proposed a semi - empirical method for the design of supports consisting of shotcrete/SFRS, rock bolts, steel ribs and grouted arches to support tunnels and caverns. Supports are designed to resist certain amount of pressures exerted on them by the rock mass. Conversely, if a support system successfully resists rock loads, pressure or its capacity can be estimated from the semi - empirical method suggested by Singh et al. (1995). This method is discussed below.

7.1 THE SEMI - EMPIRICAL METHOD FOR THE DESIGN OF SUPPORTS

The aim of supporting underground openings is to reinforce the rock mass so that it can act as a self-stabilizing structural system to support unstable zones in the rock mass. This objective may be achieved by constructing a reinforced rock arch, i.e. an array of (perfo/resin) rock anchors in both the roof and the side walls, based on the overall ground conditions.

According to Singh et al. (1995), the aim of an integrated approach to support is to construct a self-stabilizing structural system to support a wide variety of ground conditions and weak zones, keeping in mind basic tunnel mechanics and the inherent uncertainties in the exploration, testing and behaviour of geological materials.

7.1.1 The Semi - Empirical Method

Barton et al. (1974) presented five tables for the design of shotcrete and rock bolt support systems for both tunnels and caverns. Subsequently, Grimstad and Barton (1993) presented a chart (Fig. V.1) for the design of support systems consisting of SFRS and rock bolts.

Considering the usefulness of SFRS in squeezing as well as rock burst conditions apart from conventional support systems in the form of steel rib, shotcrete and rock bolts as suggested by RSR and RMR, Singh et al. (1995a, b) proposed a semi - empirical method of design of supports.

They suggested empirical relations for mobilizing factors of shotcrete and rock bolts from the extensive data of the Norwegian Geotechnical Institute (NGI) tables and charts for support systems presented by Barton et al. (1974).

Figure 7.1 shows the support system, which consists of shotcrete and rock bolts in the roof. The dotted line shows the effective width of the reinforced rock arch. The load carrying capacity (T) of the reinforced rock arch is dependent on the minimum UCS of the reinforced rock arch. The equation is as follows:

Ultimate support pressure = Total capacity of support system roof

$$(U + p_v) = P_{sc} + P_{bolt} + P_{gt} + P_{rib}$$

or $$(U + p_v) = \frac{2q_{sc}t_{sc}}{F_{sc}.B} + \frac{2q_{crm}.l'.\sin\theta}{F_s.B} + \frac{2q_{gt}l_{gt}\sin\theta}{F_{gt}.B} + \frac{P'_{rib}}{S_{rib}B} \tag{7.1}$$

$$\sin\theta \cong 1.3\,B^{-0.16} \leq 1 \tag{7.2}$$

$$F_{sc} = 0.55(100t_{sc})^{0.05} < 1 \tag{7.3}$$

$$F_s = 3.25.p_v^{0.10} \quad \text{(for pretensioned bolts)} \tag{7.4}$$

$$= 9.5p_v^{-0.35} \quad \text{(for grouted anchor)} \tag{7.5}$$

$F_{gt} \cong F_s$ for anchor

$$q_{crm} = \left[\frac{P_{bolt}}{S^2_{bolt}} - U\right]\frac{1+\sin\varphi_j}{1-\sin\varphi_j} \geq 0 \tag{7.6}$$

$$\tan\varphi_j = \frac{J_r}{J_a} \tag{7.7}$$

$$l' = l_{bolt} - \frac{FAL}{2} - \frac{S_{bolt}}{2} + 2\ S_{rock} \tag{7.8}$$

$$= l_{bolt} - \frac{FAL}{2} - \frac{S_{bolt}}{4} + S_{rock} \tag{7.9}$$

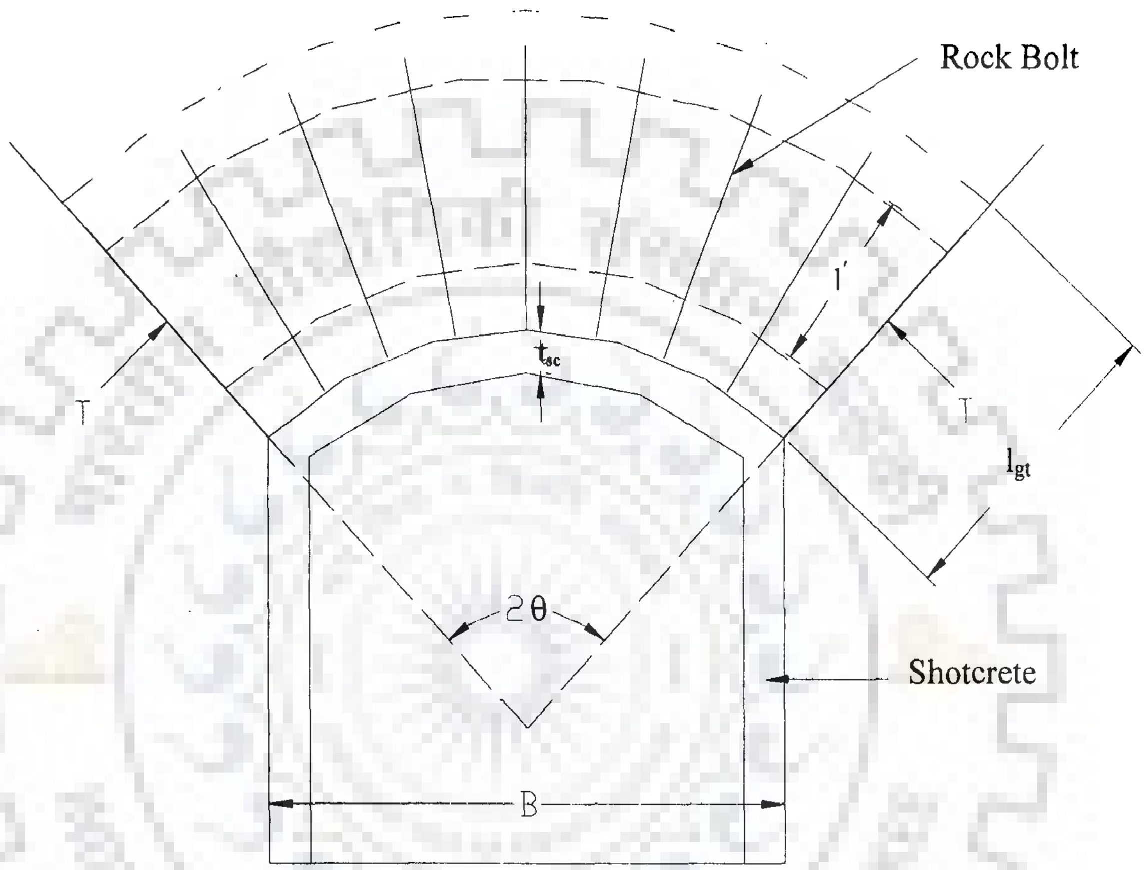


Fig. 7.1 Capacity of reinforced rock arch

(in case of mesh and / or shotcrete)

$$\leq l_{bolt} - \frac{FAL}{2}$$

$$FAL = 100 \text{ x diameter of anchor bars} \tag{7.10}$$

where, all parameters are in tonnes and/or metres. P'_{rib} is the load on the rib.

Singh et al. (1995b) analyzed 67 ranges of Q-values ranging from 0.001 to 100 and span ranging from 2 m to 46 m taken from Barton et al. (1974) and 8 ranges of Q-values ranging from 0.1 to 10 and span ranging from 5 m to 30 m from Grimstad and Barton (1993). Singh et al. (1995b) compared empirically suggested support capacity and the one by semi - empirical estimate of the support capacity proposed by them. The comparison indicated that the estimated support capacity matched the empirical capacity for 60 percent cases only. According to them, the comparison was quite encouraging for low pressures and the high scatter was for unusually large spans and unusually thick shotcrete only.

7.1.2 Modified Semi - Empirical Method

The semi - empirical method has been utilized in the present work to ascertain its practical value by estimating capacities of support systems recommended by all the systems. Equations 7.1 through 7.10 have been used in computing the support capacities. The input data chosen in the computations are indicated in Table 7.1.

The support capacities estimated from the semi - empirical method have been plotted against the support pressures estimated from the Q-system in Fig. 7.2. In this figure, if support pressures are greater than 0.25 MPa or so, support capacities estimated by the semi - empirical method are not equal to that of the pressures (Eq. 6.23).

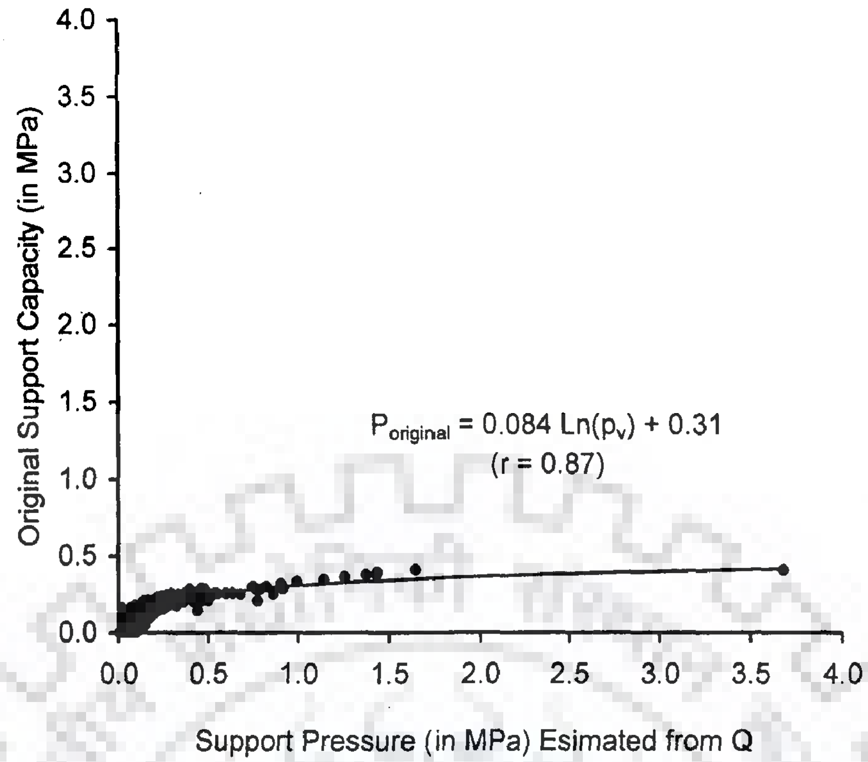


Fig. 7.2 Comparison of support pressure estimated from Q and capacity estimated from semi - empirical method

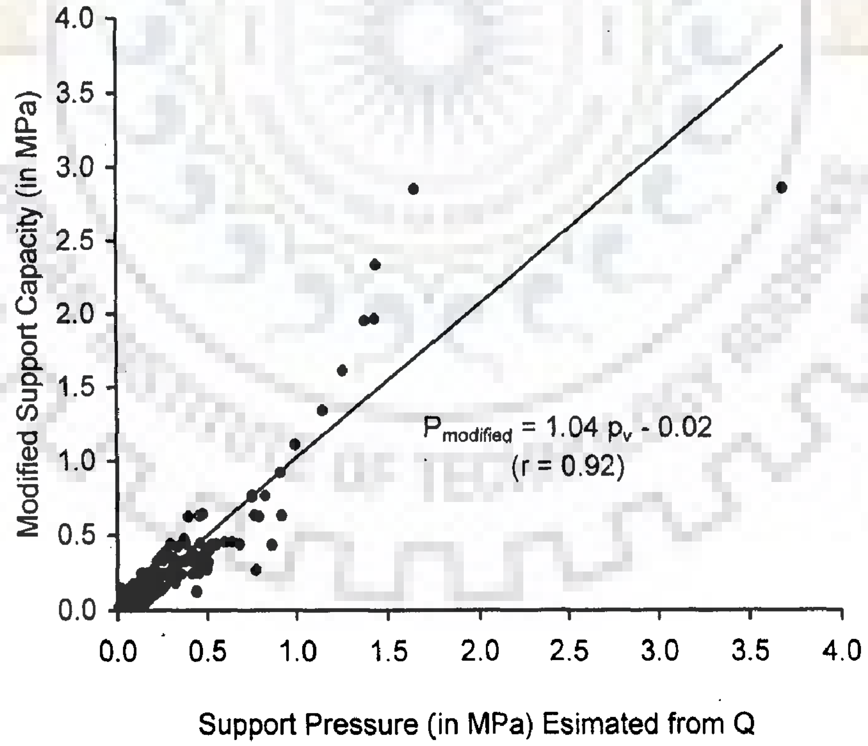


Fig. 7.3 Comparison of support pressure estimated from Q and capacity estimated from modified semi - empirical method

Table 7.1 Input Values for Estimating Support Capacity by Semi - Empirical Method

Parameter	Value / Source
q_{sc}	3 MPa (for shotcrete) 6 MPa (for SFRS)
Diameter of rock bolt, d_b	25 mm
P_{bolt}	200 kN
U	0
P_{gt}	0
p_v	Table 5.21 for the Q-system

Therefore, this method may not be applied successfully beyond 0.25 MPa pressure range, as concluded by Singh et al. (1995b) also. They mentioned that high scatter was obtained for larger span or higher shotcrete thickness indicating that support pressures in those situations were very high. In Fig. 7.2, support capacity becomes almost constant for pressure greater than 0.5 MPa.

A correlation is developed between support capacity ($P_{original}$) by Eq. 7.1 and support pressure (p_v) by Eq. 6.23 as:

$$P_{original} = 0.084\, L_n\,(p_v) + 0.31 \quad (7.11)$$

Coefficient of correlation for Eq. 7.11 is 0.87.

Equation 7.11 is utilized to incorporate modification in Eq. 7.1 so that it can be applied for higher support pressures as well. The correlation established between modified support capacity ($P_{modified}$) and original support capacity ($P_{original}$) is as follows:

$$P_{modified} = e^{\left(\frac{P_{orifinal} - 0.32}{0.084}\right)} \quad (7.12)$$

The graph plotted between modified support capacity ($P_{modified}$) and support pressure (p_v) (Fig. 7.3, placed below Fig. 7.2) indicates an excellent correlation between the two, having a coefficient of correlation as 0.92. This clearly demonstrates that the modification suggested in this study as per Eq. 7.12 (called as modified semi – empirical method in this thesis) may well render original method applicable for higher pressures as well.

7.2 COMPARISON OF SUPPORTS

For making a comparison of support systems recommended by RSR, RMR, Q and RMi, a numerical value in the form of modified support capacity for each support system, has been obtained by Eq. 7.1 and 7.12 as presented in Table 7.2. Support charts for estimating capacity of shotcrete/SFRS and rock bolts systems have been prepared for the database contained in this thesis (Fig. 7.4). Capacities for the supports installed by the Project Authorities have also been estimated on the similar lines as for the RMR, Q and RMi systems (Table 7.2).

7.2.1 Capacity Ranges and General Trend

7.2.1.1 Non-squeezing ground conditions

Figure 7.5 compares support capacities involving 592 sections in non-squeezing ground conditions where RSR, RMR and Q have suggested support measures. However, in case of RMi only 432 sections have been included since RMi has not recommended any quantifiable support measure in rest of the sections. In these sections, RMi recommended either ISC or SDSC.

From Fig. 7.5, capacity range is: (i) for RSR 0.064 - 0.467 MPa, (ii) for RMR 0.022 - 0.302 MPa (for about 97 percent sections, capacity < 0.065 MPa), (iii) for Q 0.01 - 0.639 MPa (for about 98 percent sections, capacity < 0.4 MPa) and (iv) for RMi 0.022 - 1.08 MPa (for about 77 percent sections, capacity < 0.4 MPa). From maximum capacity point of view, $C_{RMR} < C_{RSR} < C_Q < C_{RMi}$,

Table 7.2 Capacity of Supports by Classification Systems and Installed by the Project Authorities

S. No.	Chainage (in m)	Support Capacity (in MPa)					Squeezing Condition	Shear Zone
		RSR	RMR	Q	RMi	Project		
1	0-50	0.065	0.039	0.177	0.079	0.073		
2	966-987		0.062	0.477		1.135	Mild	SZ
3	1375-1587	0.135	0.039	0.120	0.084	0.069		
4	2057-2112	0.288	0.045	0.116	0.177	0.037		
5	2155-2202		0.261	0.756		0.255	Moderate	SZ
6	3631-3644	0.299	0.051	0.373		0.069		
7	6208-6338	0.114	0.038	0.049	0.023	0.069		
8	9687-9697	0.190	0.038	0.246	0.533	0.262		
9	11435-11446	0.208	0.045	0.098	0.177	0.076		
10	11634-11643	0.158	0.038	0.143	0.599	0.069		
11	11932-12044	0.088	0.038	0.109	0.086	0.070		
12	12044-12070	0.208	0.042	0.098	0.272	0.075		
13	12087-12223	0.208	0.045	0.145	0.154	0.070		
14	12359-12428	0.114	0.038	0.140	0.139	0.070		
15	12544-12608	0.158	0.038	0.143	0.415	0.070		
16	12629-12668	0.121	0.038	0.140	0.197	0.070		
17	16036-16046	0.114	0.040	0.159	0.18	0.283		
18	16840-16900	0.065	0.022	0.252	0.084	0.068		
19	16900-17000	0.121	0.040	0.258	0.198	0.066		
20	17040-17070	0.181	0.040	0.181	0.122	0.324		
21	18387-18424	0.208	0.038	0.162	0.756	0.070		
22	18535-18687	0.226	0.023	0.090		0.073		
23	19015-19054	0.288	0.042	0.252		0.074		
24	19054-19103	0.256	0.044	0.243		0.074		
25	19189-19700	0.208	0.038	0.227		0.074		
26	21230-21240		0.222	0.519		0.317	Moderate	
27	21240-21247		0.192	0.241	0.502	0.321	Mild	
28	22537-22648	0.323	0.047	0.400		0.277		
29	22795-22848	0.389	0.050	0.270		0.320		
30	23627-23785	0.323	0.052	0.172		0.265		
31	23894-23901		0.052	0.258	0.928	0.320	Mild	
32	25043-25080	0.276	0.065	0.120		0.095		
33	25951-25980	0.404	0.172	0.205		0.319		
34	26108-26112		0.382	0.458		1.107	Mild	SZ
35	26120-26126		0.287	1.651		1.021	High	SZ
36	26361-26398		0.382	1.438			Moderate	SZ
37	26415-26478		0.382	0.769			Mild	SZ
38	26478-26506		0.421	1.378			Moderate	SZ
39	26608-26620		0.382	2.952		0.992	High	SZ
40	26946-27346	0.198	0.038	0.140	0.553			

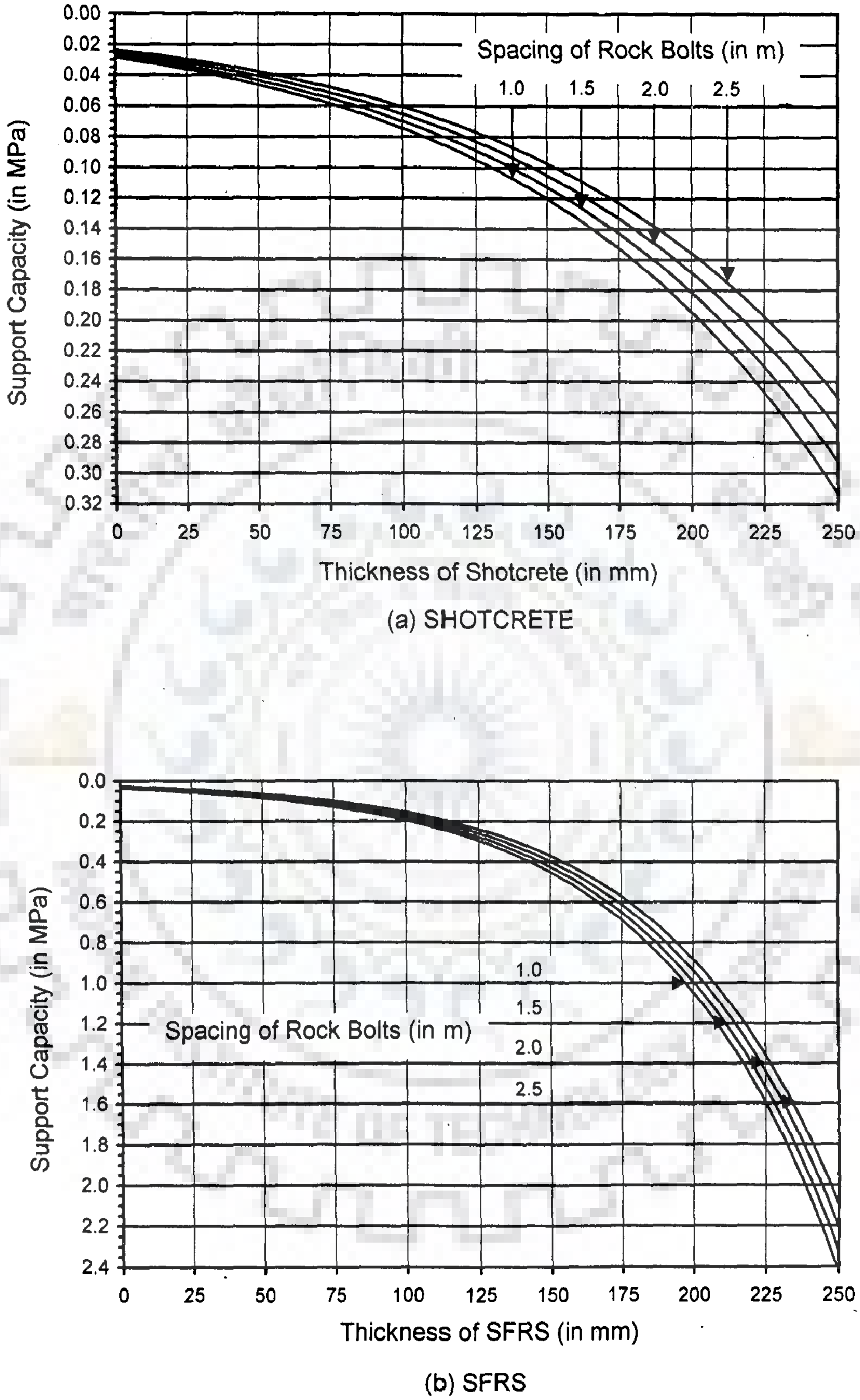


Fig. 7.4 Chart for estimating support capacity from modified semi - empirical method

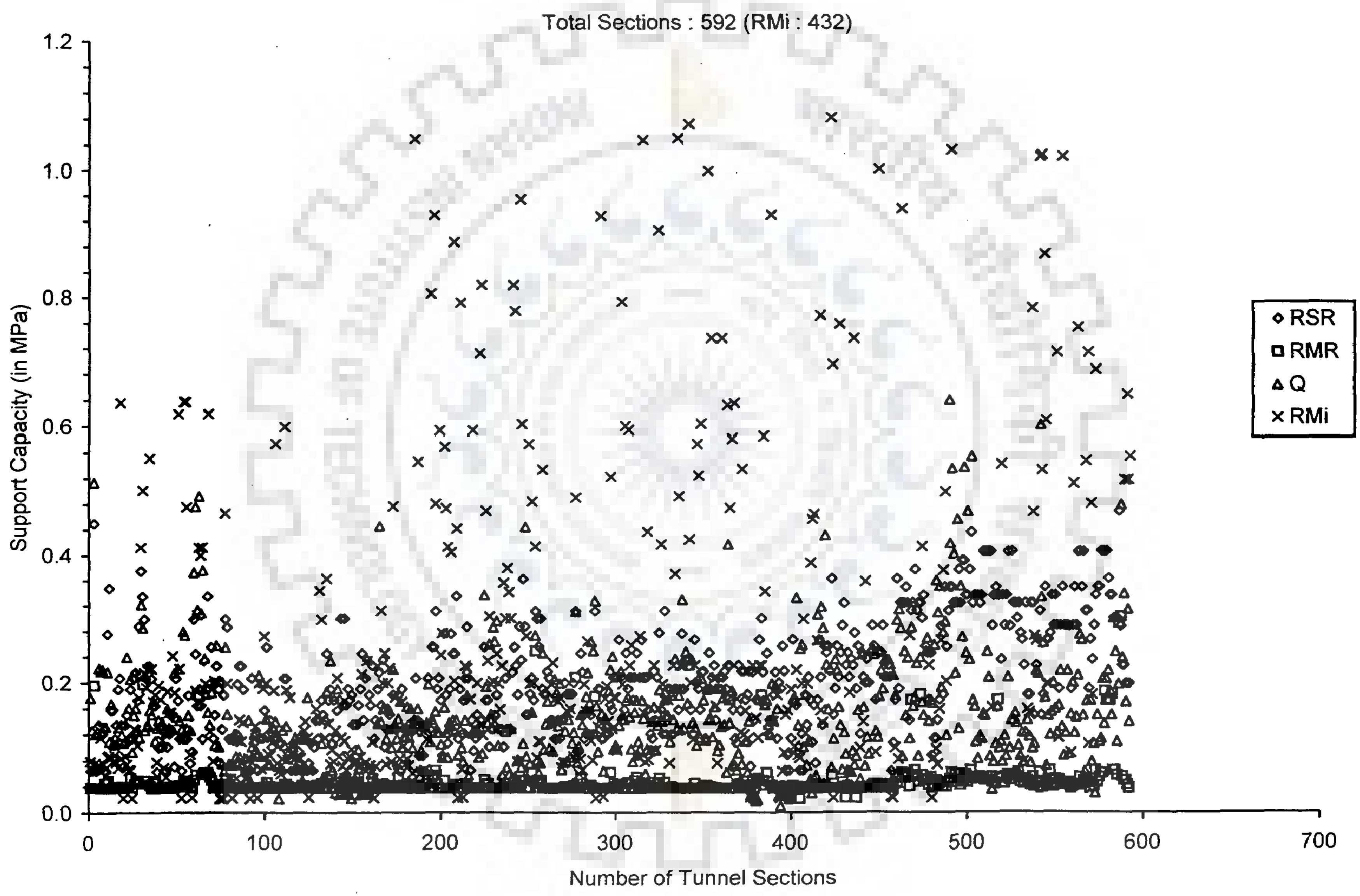


Fig. 7.5 Comparison of support Capacity for non-squeezing ground conditions

but considering that for Q, about 98 percent sections are below 0.4 MPa, RSR and Q swap their positions in the order and new order becomes $C_{RMR} < C_Q < C_{RSR} < C_{RMi}$. However for RMi also, about 77 percent sections are below 0.4 MPa and brings it close to Q and forces a new order of $C_{RMR} < C_Q = C_{RMi} < C_{RSR}$. Rest of the sections i.e. about 23 percent are of very high order of capacity thereby not convincing enough to keep the order just mentioned. Therefore a general trend may be given as follows:

Support Capacity (Non-Squeezing): $C_{RMR} < C_Q < C_{RSR} << C_{RMi}$

Elaborating Further:

(a) RMR suggested less support than RSR for all the sections and RMi for about 95 percent sections. This clearly indicates that RMR suggested lighter supports in non-squeezing ground conditions as compared to RSR and RMi.

(b) RSR suggested lighter supports than RMi above 0.2 MPa range for about 93 percent of sections and heavier supports than RMi below 0.2 MPa range for about 75 percent of sections. This indicates that for higher pressures RMi recommended supports heavier than even those by RSR, which were earlier considered as the heaviest ones.

(c) Q suggested lighter supports than RSR for about 87 and 65 percent sections for above and below 0.2 MPa respectively. Hence, Q suggested lighter supports than RSR for about 75 percent sections, which is consistent with the general trend.

Figure 7.6 shows number of tunnel sections in different support capacity ranges for non-squeezing ground conditions. From this figure, it is clear that all classification systems except RMR suggested supports in almost all the capacity ranges more or less uniformly. RMR recommended support capacity in the range of 0 - 0.05 MPa for about 82.6 percent sections whereas about 97 percent sections fall below 0.1 MPa (Fig. 7.7). Another interesting feature to note from Fig. 7.6 and 7.7 is that RMi recommended supports almost uniformly up to 0.25 MPa,

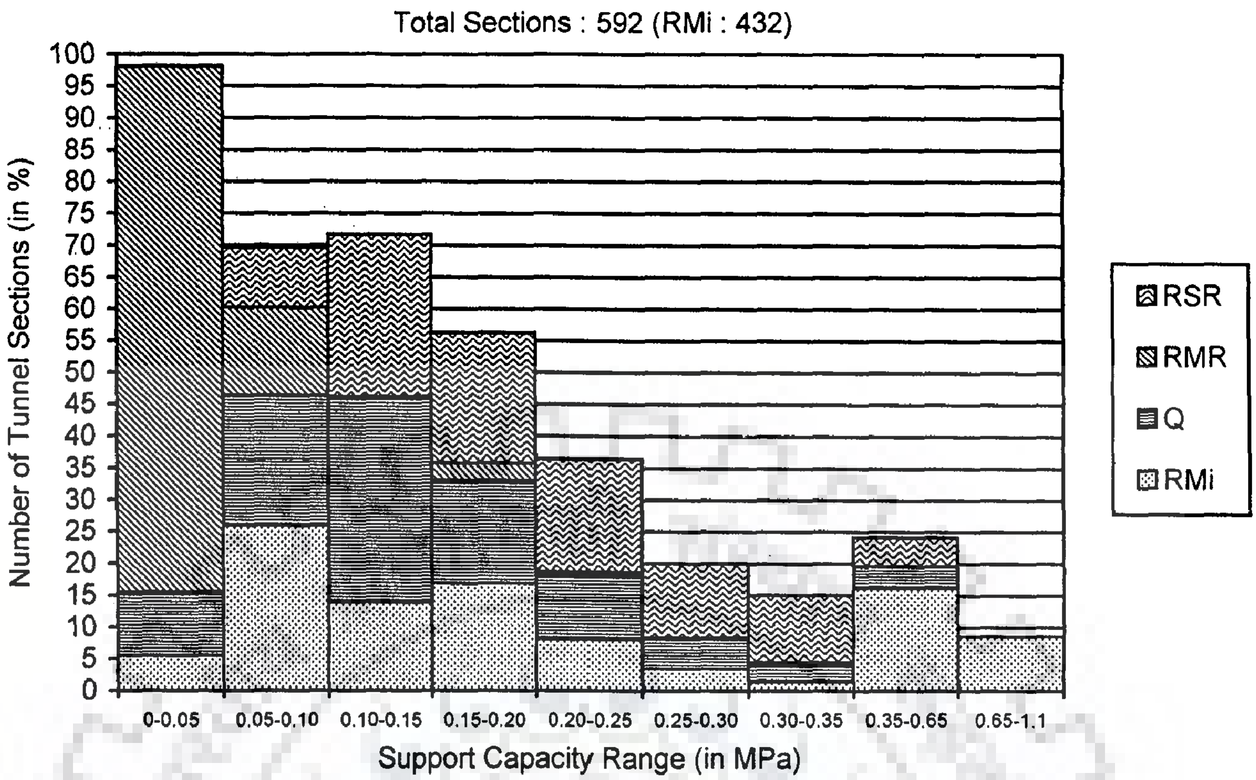


Fig. 7.6 Number of tunnel sections in different support capacity ranges for non-squeezing ground conditions

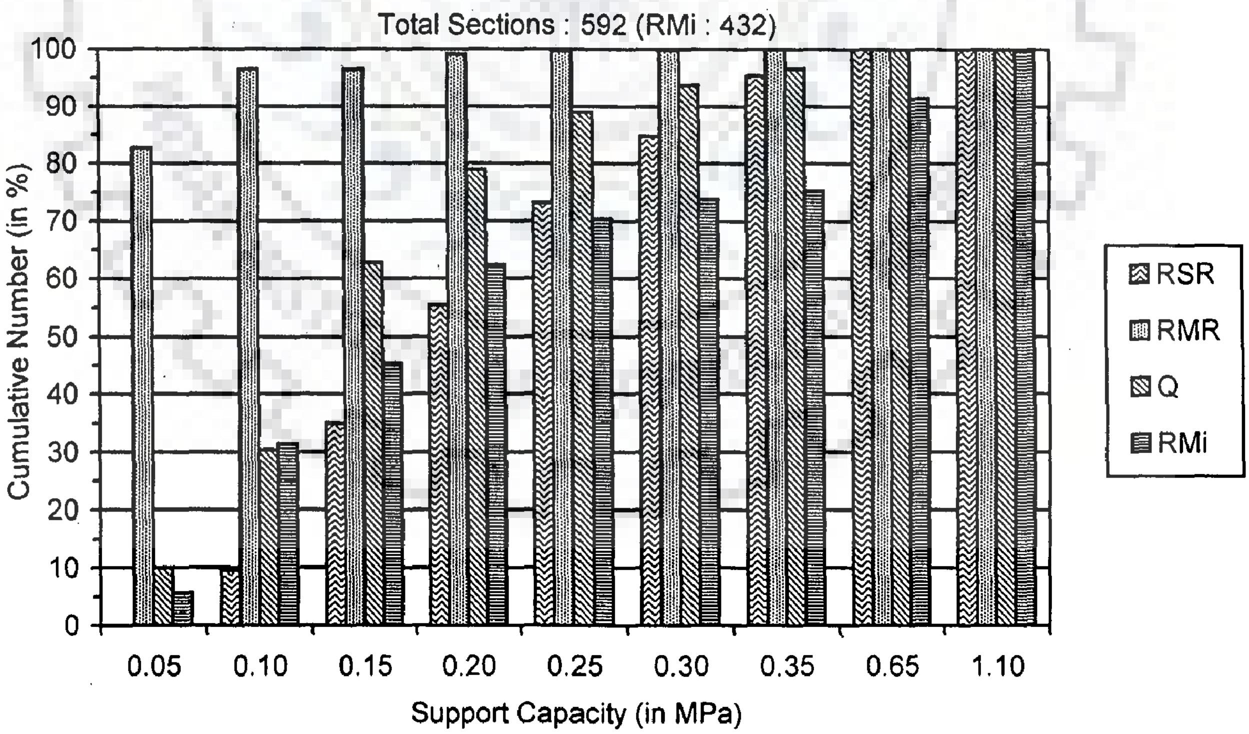


Fig. 7.7 Cumulative number of tunnel sections below support capacity for non-squeezing ground conditions

decreased thereafter up to 0.35 MPa and above 0.35 MPa percent sections increased to about 25 percent.

Table 7.3 compares classification systems in terms of capacity of support recommended by them in non-squeezing ground conditions involving a total of 592 sections. Two systems have been taken at a time and number of sections falling in each group of inequality has been indicated, on the basis of which a conclusion has been drawn in the last column of Table 7.3.

Table 7.3 Comparisons of Classification Systems in Terms of Capacity of Support in Non-Squeezing Ground Conditions

S. No.	Classification Systems	Inequality	Maximum Deviation (in MPa)	Number of Sections (in percent)	Conclusion
1.	RSR and RMi	$C_{RSR} < C_{RMi}$	0.9	63*	$C_{RSR} < C_{RMi}$
		$C_{RSR} = C_{RMi}$	±0.015	7	
		$C_{RSR} > C_{RMi}$	0.26	30	
2.	Q and RMi	$C_Q < C_{RMi}$	0.9	71*	$C_Q < C_{RMi}$
		$C_Q = C_{RMi}$	±0.015	7	
		$C_Q > C_{RMi}$	0.3	22	
3.	RMR and RMi	$C_{RMR} < C_{RMi}$	1.0	95	$C_{RMR} < C_{RMi}$
		$C_{RMR} = C_{RMi}$	±0.015	2.5	
		$C_{RMR} > C_{RMi}$	0.02	2.5	
4.	Q and RSR	$C_Q < C_{RSR}$	0.3	66	$C_Q < C_{RSR}$
		$C_Q = C_{RSR}$	±0.015	9.5	
		$C_Q > C_{RSR}$	0.3	24.5	
5.	RMR and RSR	$C_{RMR} < C_{RSR}$	0.35	100	$C_{RMR} < C_{RSR}$
6.	RMR and Q	$C_{RMR} < C_Q$	0.5	87	$C_{RMR} < C_Q$
		$C_{RMR} = C_Q$	±0.015	11	
		$C_{RMR} > C_Q$	0.1	2	

* Including SDSC

Table 7.3 further establishes that the general trend: $C_{RMR} < C_Q < C_{RSR} << C_{RMi}$ mentioned earlier is quite appropriate.

7.2.1.2 Squeezing ground conditions

Figure 7.8 shows comparison of support capacities involving 50 sections in squeezing ground conditions where RMR and Q have suggested support measures whereas for RMi, just 8 sections were available since in rest of the sections RMi has not recommended any quantifiable support measure. In these sections, RMi recommended either ISC or SDSC.

From Fig. 7.8, it may be noted that capacity range is: (i) for RMR 0.049 - 0.501 MPa, (ii) for Q 0.187 - 3.68 MPa (for about 86 percent sections, capacity < 1.5 MPa; for about 70 percent sections, capacity < 1.0 MPa and for about 52 percent sections, capacity < 0.5 MPa) and (iii) for RMi 0.326 - 0.928 MPa. From maximum capacity point of view, $C_{RMR} < C_{RMi} < C_Q$, but considering that for Q, about 86 percent sections are below 1.5 MPa and only 8 sections have been considered in RMi with the likelihood that in rest of the sections capacity might be much more than 1.5 MPa, it is quite clear that supports suggested by Q are lighter than those by RMi. For all 50 sections, Q suggested heavier supports than those by RMR. For all 8 sections, RMi recommended heavier supports as compared to those by RMR. Therefore, a general trend may be given as follows:

Support Capacity (Squeezing): $\mathbf{C_{RMR} < C_Q < C_{RMi}}$

The RSR-system is not applicable in squeezing ground conditions.

The trends observed in both non-squeezing and squeezing ground conditions are similar. Therefore, it may be concluded that irrespective of ground conditions, support capacities suggested by all classification systems barring RSR may be given in the following order:

Support Capacity: $\mathbf{C_{RMR} < C_Q < C_{RMi}}$

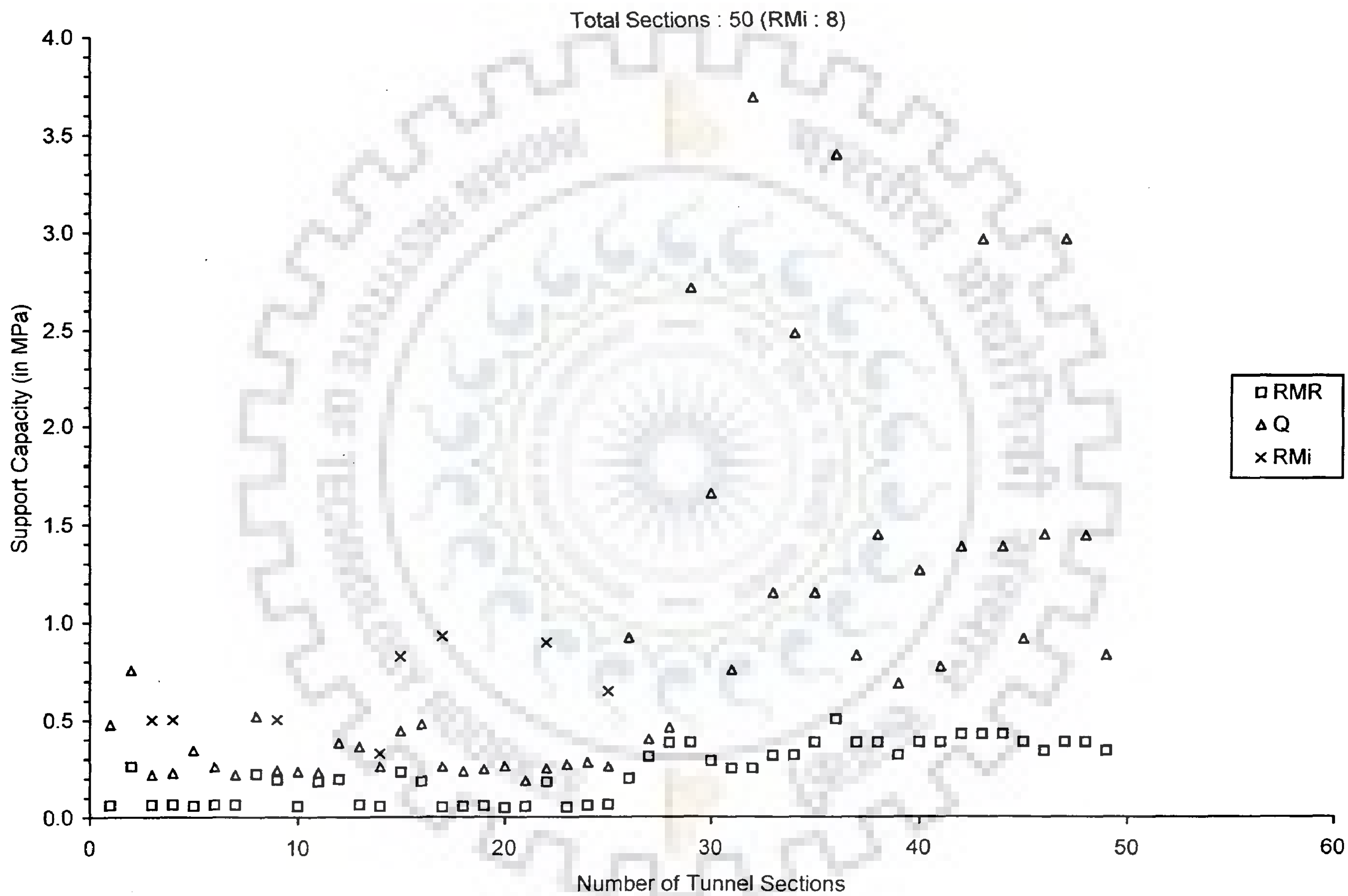


Fig. 7.8 Comparison of support capacity for squeezing ground conditions

Figure 7.9 shows number of tunnel sections for different support capacity ranges. Fig. 7.10 shows cumulative number of sections below support capacity for squeezing ground conditions.

In Figs. 7.9 and 7.10, almost half the sections are below 0.2 MPa for RMR whereas for Q, about 52 percent sections are below 0.6 MPa, a high value. But interestingly, between 0.2 MPa and 0.5 MPa, both Q and RMR have same number of sections i.e. about 52 percent.

Table 7.4 compares classification systems in terms of capacity of support recommended by them in squeezing ground conditions involving a total of 50 sections. Two systems have been taken at a time and number of sections falling in each group of inequality has been indicated, on the basis of which a conclusion has been drawn in the last column of Table 7.4.

Table 7.4 Comparisons of Classification Systems in Terms of Capacity of Support in Squeezing Ground Conditions

S. No.	Classification Systems	Inequality	Number of Sections (in percent)	Conclusion
1.	Q and RMi	$C_Q < C_{RMi}$	75	$C_Q < C_{RMi}$
		$C_Q > C_{RMi}$	25*	
2.	RMR and RMi	$C_{RMR} < C_{RMi}$	100	$C_{RMR} < C_{RMi}$
		$C_{RMR} > C_{RMi}$	0	
3.	RMR and Q	$C_{RMR} < C_Q$	100	$C_{RMR} < C_Q$
		$C_{RMR} > C_Q$	0	

*Q: SFRS > 230 mm thick and CCA
RMi: SDSC

Table 7.4 further establishes that the general trend: $C_{RMR} < C_Q << C_{RMi}$ mentioned earlier is quite appropriate.

7.2.2 Evaluation of Supports

Number and length of tunnel sections in Class I, II, III, IV, V, VI and VI + supports installed by the Project Authorities (Figs. VIII.1 a, b, c and 4.1) has been presented in Table 5.23. Average capacity of each class of supports installed by the Project Authorities estimated as per the modified semi - empirical method is presented in Table 7.5.

Table 7.5: Average Capacity of Support Installed by the Project Authorities

Project Class	Number of Sections (in %)	Support Capacity (in MPa)
II	17.8	0.037
III	43.4	0.07
IV	18.2	0.27
V	11.0	0.325
VI	2.5	1.135
VI+	1.7	-

Data of Class I is not available and Class VI+ cannot be quantified.

7.2.2.1 RSR v/s Project

Figure 7.11 indicates number of tunnel sections in support classes by the Project Authorities for RSR-values. Most of the tunnel sections fall in RSR range of 30 - 70, and in almost all ranges, the Project Authorities have not used RSR-system for all classes. However, in 40 - 50 range, majority of supports by the Project Authorities are in Class II and III (about 24.3 percent out of 32 percent), in 50 - 60 range also, majority supports are also in Class II and III (about 25.1 percent out of 29 percent) and all supports by the Project Authorities in 60 - 70 range are again in Class II

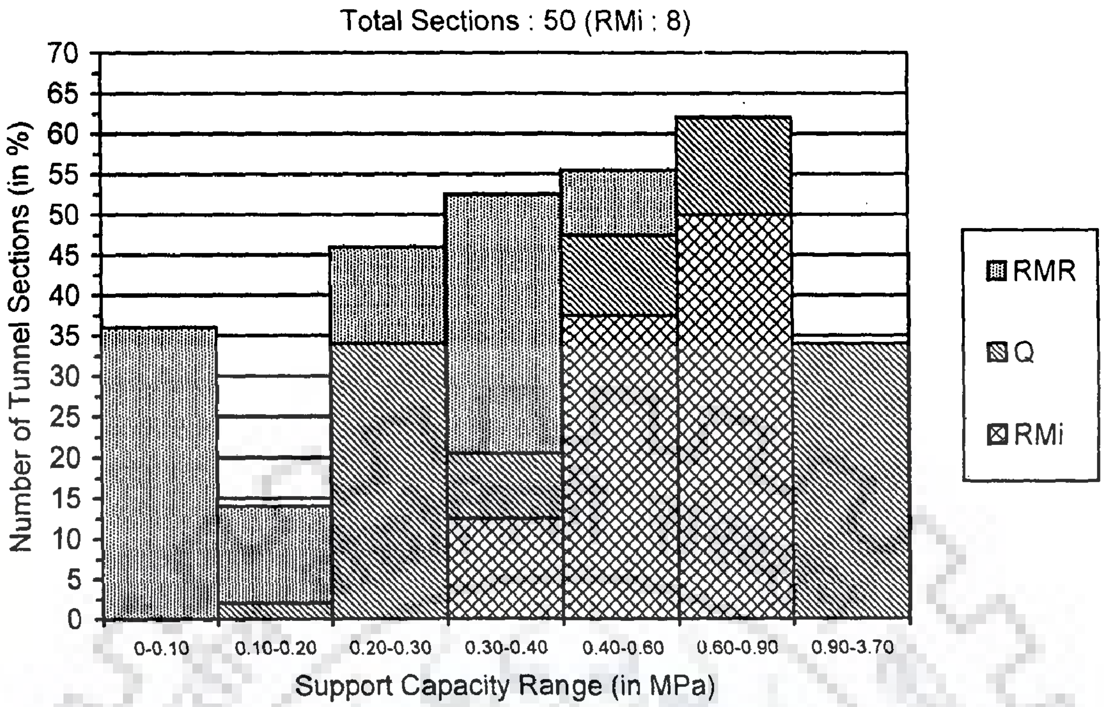


Fig. 7.9 Number of tunnel sections in different support capacity ranges for squeezing ground conditions

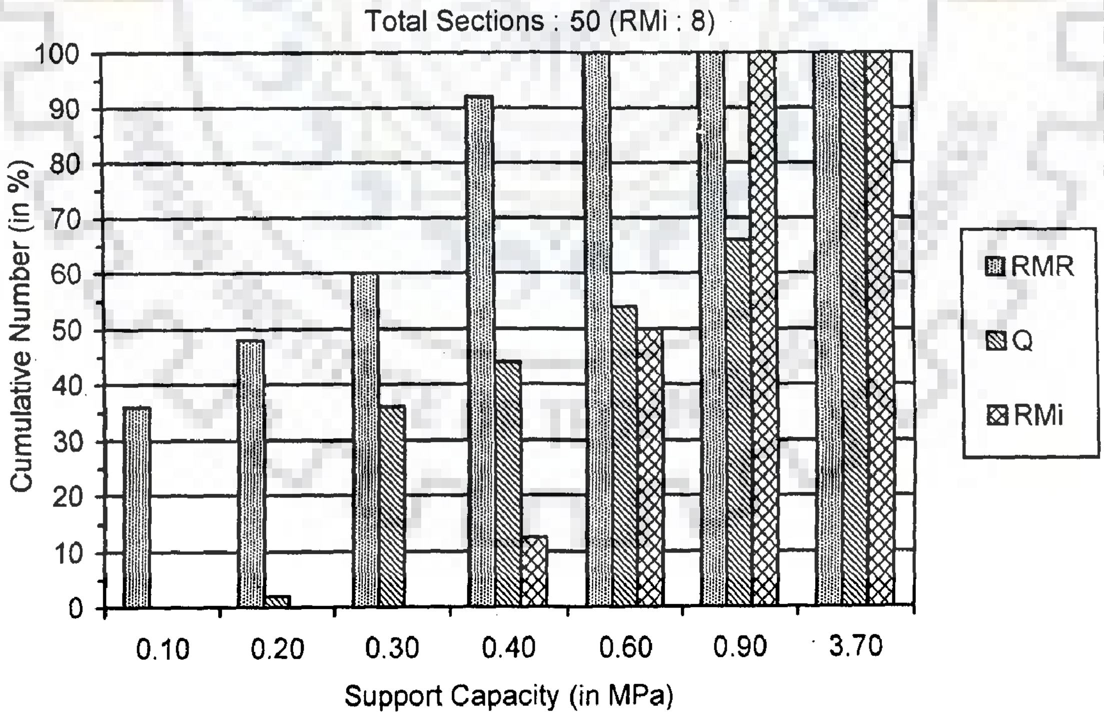


Fig. 7.10 Cumulative number of tunnel sections below support capacity for squeezing ground conditions

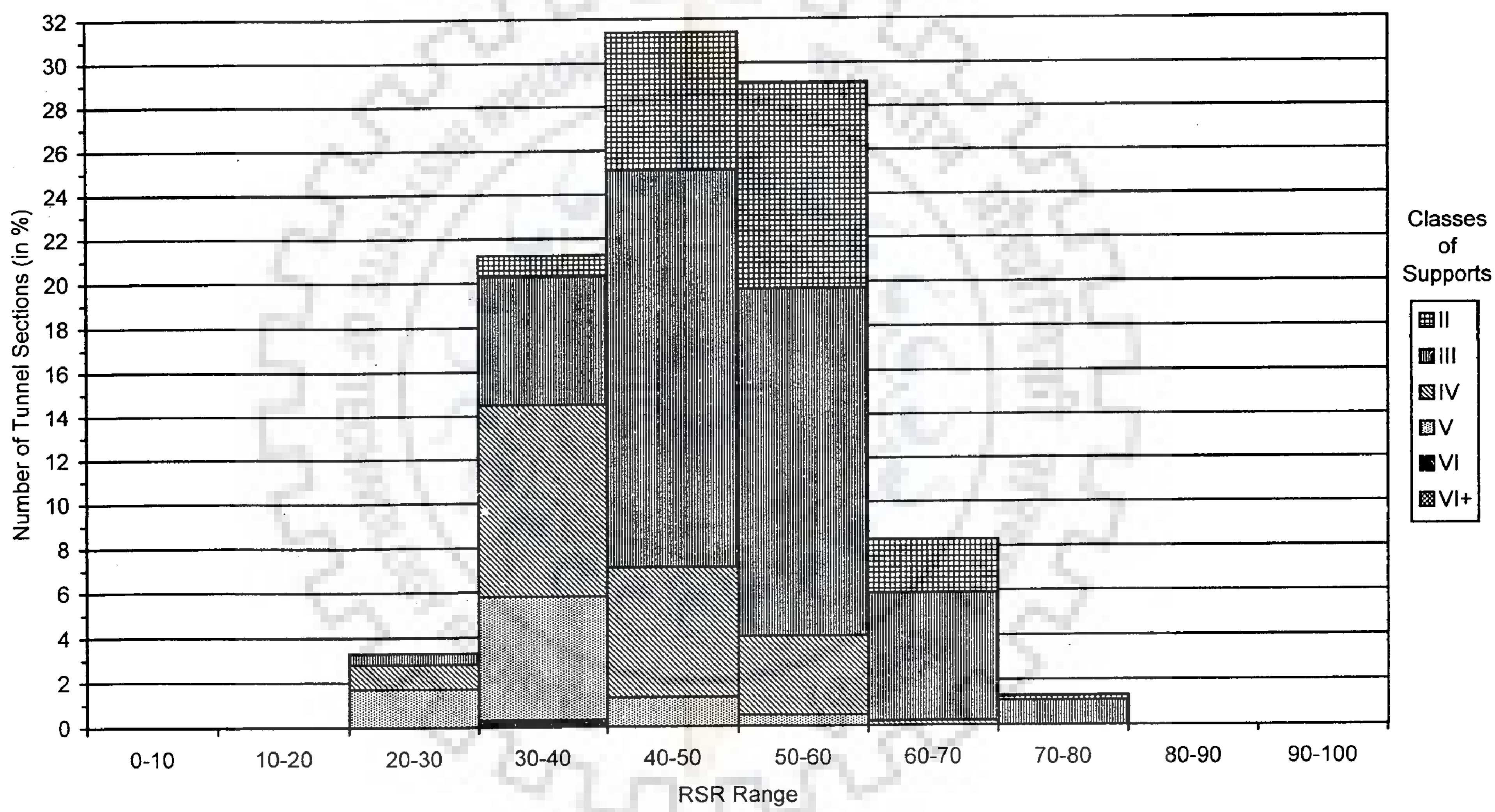


Fig. 7.11 Number of tunnel sections in classes of supports installed by the Project Authorities for RSR-values

and III. From this disposition of project classes of low order of support capacity (Class II and III) in higher ranges (40 - 70) makes clear that supports installed by the Project Authorities are somewhat in conformity with RSR-values. In a similar way, it is observed that in 30 - 40 and 20 - 30 ranges, Class IV and V constitute major portion of the distribution. Therefore, in general it may be said that in relative terms higher RSR-values correspond to lower project class of supports.

Figures 7.12 and 7.13 compare support capacity by RSR and the Project Authorities. Figure 7.12 a is an x-y scatter of both support capacities for different support classes by the Project Authorities. The support capacity by RSR ranges from 0.064 MPa to 0.467 MPa whereas for Class II to VI, capacity ranges from 0.037 MPa to 1.135 MPa as indicated in Table 7.5 also. There are only two sections with Class VI, therefore it may be said that support capacity lies between Classes II and V i.e. 0.037 MPa to 0.325 MPa. Figure 7.12 b compares support capacity against RSR-values for all project classes. From Fig. 7.12 b, it is clear again that almost all supports have been provided by the Project Authorities for RSR range of 30 - 70.

Figures 7.13 a through 7.13 e indicate capacity of supports installed by the Project Authorities as per Table 7.5, which is a constant value for respective support class and since all these classes fall in 30 - 70 range, capacity variations in both are bound to take place. In Class II and III, RSR recommended higher supports whereas for Class IV and V, they intersect each other at a particular RSR-value. If RSR-value in other sections, where Class IV and V have been provided, is lower than that at the point of intersection, RSR obviously recommended higher supports and vice versa. In Class VI, capacity of support by the Project Authorities is higher in both sections. It may be noted that no support failed in the non-squeezing ground conditions.

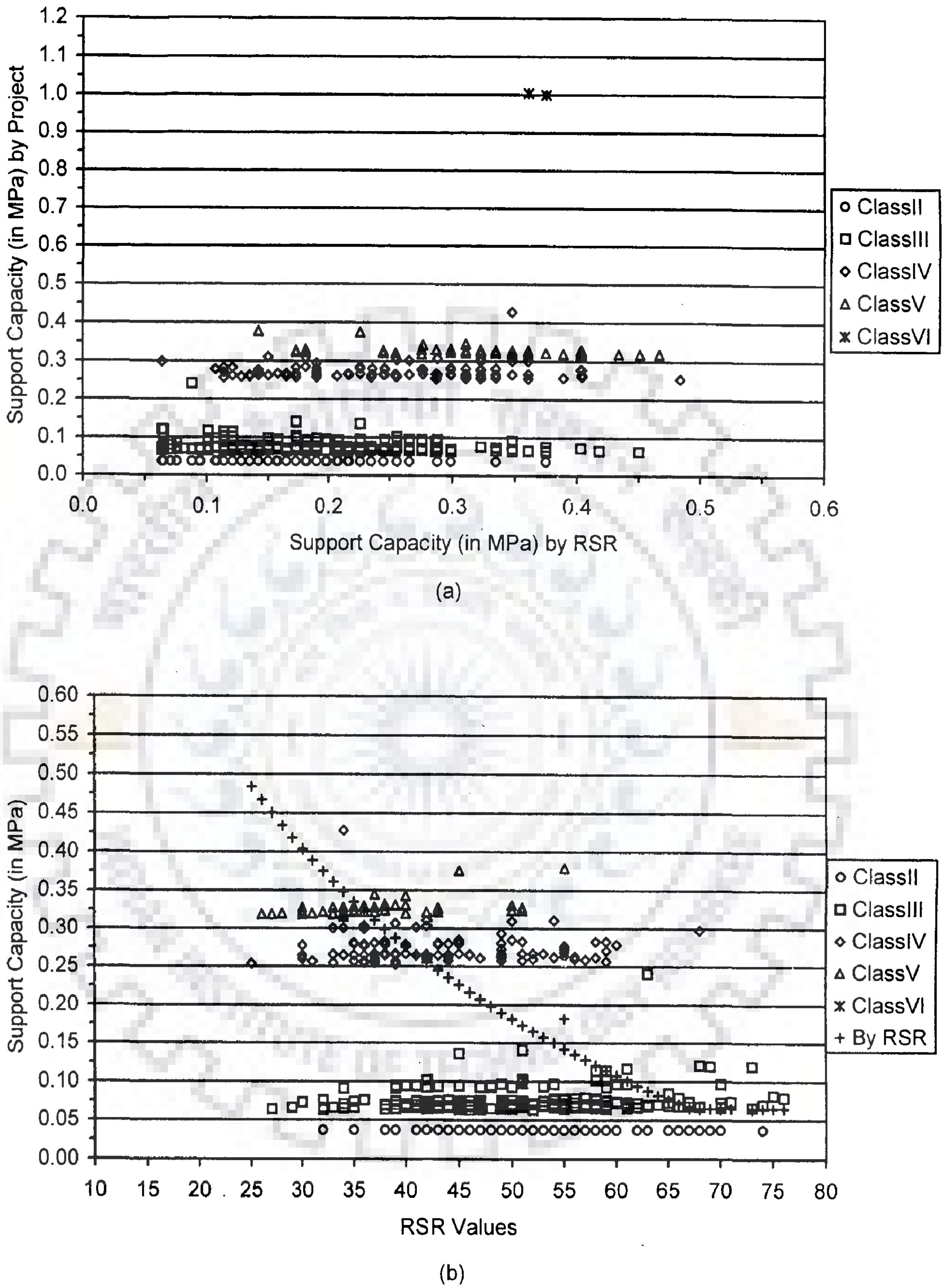


Fig. 7.12 Comparison of capacity of support recommended by RSR and installed by the Project Authorities

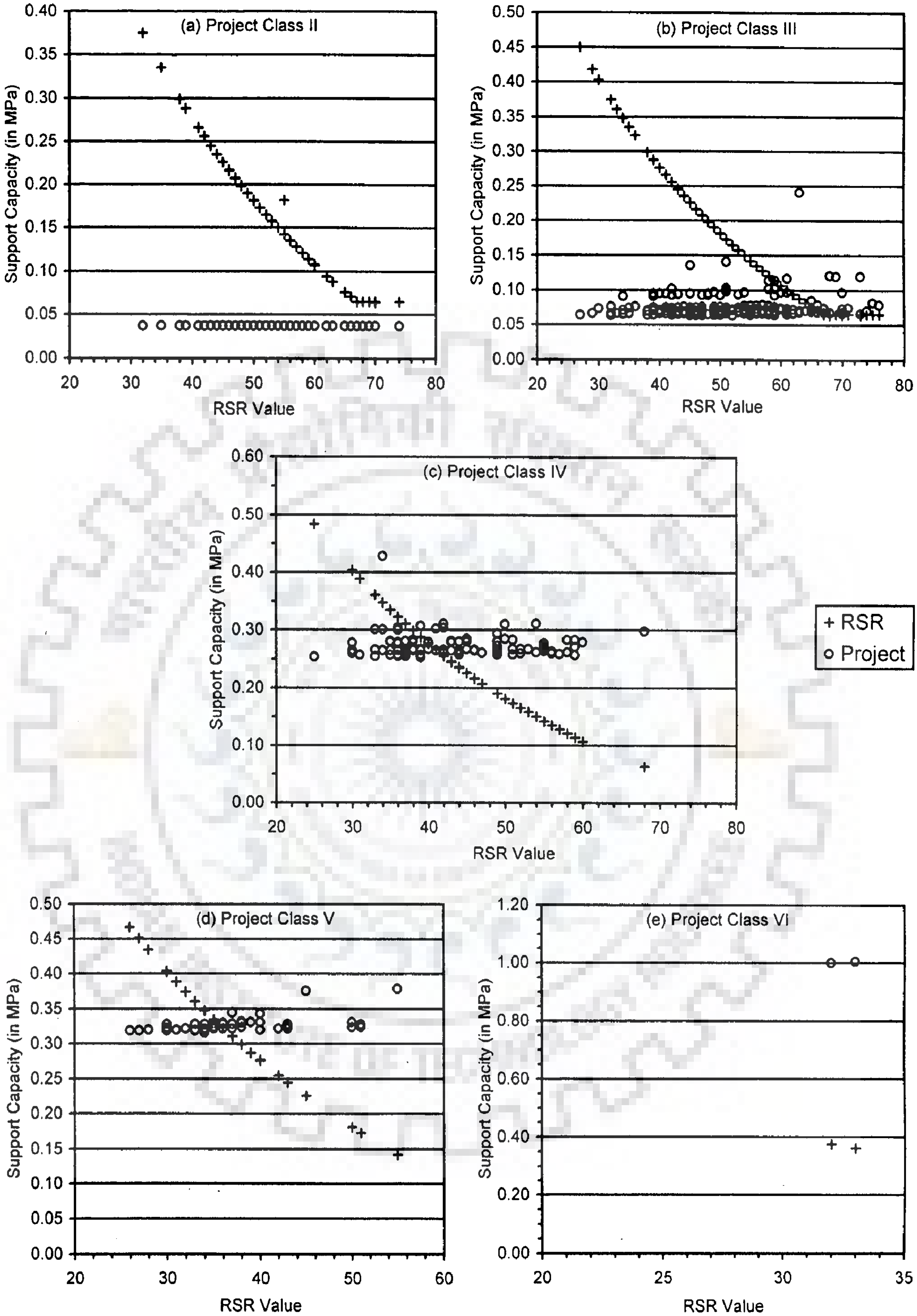


Fig. 7.13 Comparison of capacity of support recommended by RSR and installed by the Project Authorities for different project classes

7.2.2.2 RMR v/s Project

Figure 7.14 indicates number of tunnel sections in support classes by the Project Authorities for RMR-values. In this figure, most of the tunnel sections fall in RMR range of 40 - 70. In 70 - 80 range (Good rock), 1.9 out of 4.4 percent project support is Class II (Good) and rest 2.5 percent is Class III (Fair), in 60 - 70 RMR range (again Good rock), only one third is 'Good' and rest two third is 'Fair'. It means that there is degradation by one class in large number of sections by the Project Authorities for 'Good' rock following RMR. Similarly, in 40 - 60 range (Fair rock), only 20 percent fall in 'Fair' class whereas about 50 percent fall in 'Poor' and 'Very poor' classes. Similarly, range 30 - 40 (Poor) contains higher percentage of 'Very poor' class. In view of this, it may be said that the Project Authorities have in general provided supports one class lower (heavier supports) than those recommended by RMR. The Project Authorities have actually used RMR support system (Table 7.6).

Figures 7.15 and 7.16 compare support capacity by RMR and the Project Authorities. Figure 7.15 a is an x-y scatter of both support capacities for different support classes by the Project Authorities. The support capacity by RMR ranges from 0.022 MPa to 0.42 MPa whereas for Class II to VI, capacity ranges from 0.037 MPa to 1.135 MPa as indicated in Table 7.5 also. Figure 7.15 b compares support capacity against RMR-values for all project classes. In Fig. 7.15 b, Class II, III, IV and V are lying in range of 30 - 70. Figures 7.16 a through 7.16 e indicate capacity of supports installed by the Project Authorities as per Table 7.5, which is a constant value for respective support class and since most of these classes fall in 30 - 70 range, capacity variations in both classes are bound to take place. In Class II at higher RMR values, both capacities are equal but in other classes, RMR recommended lighter supports.

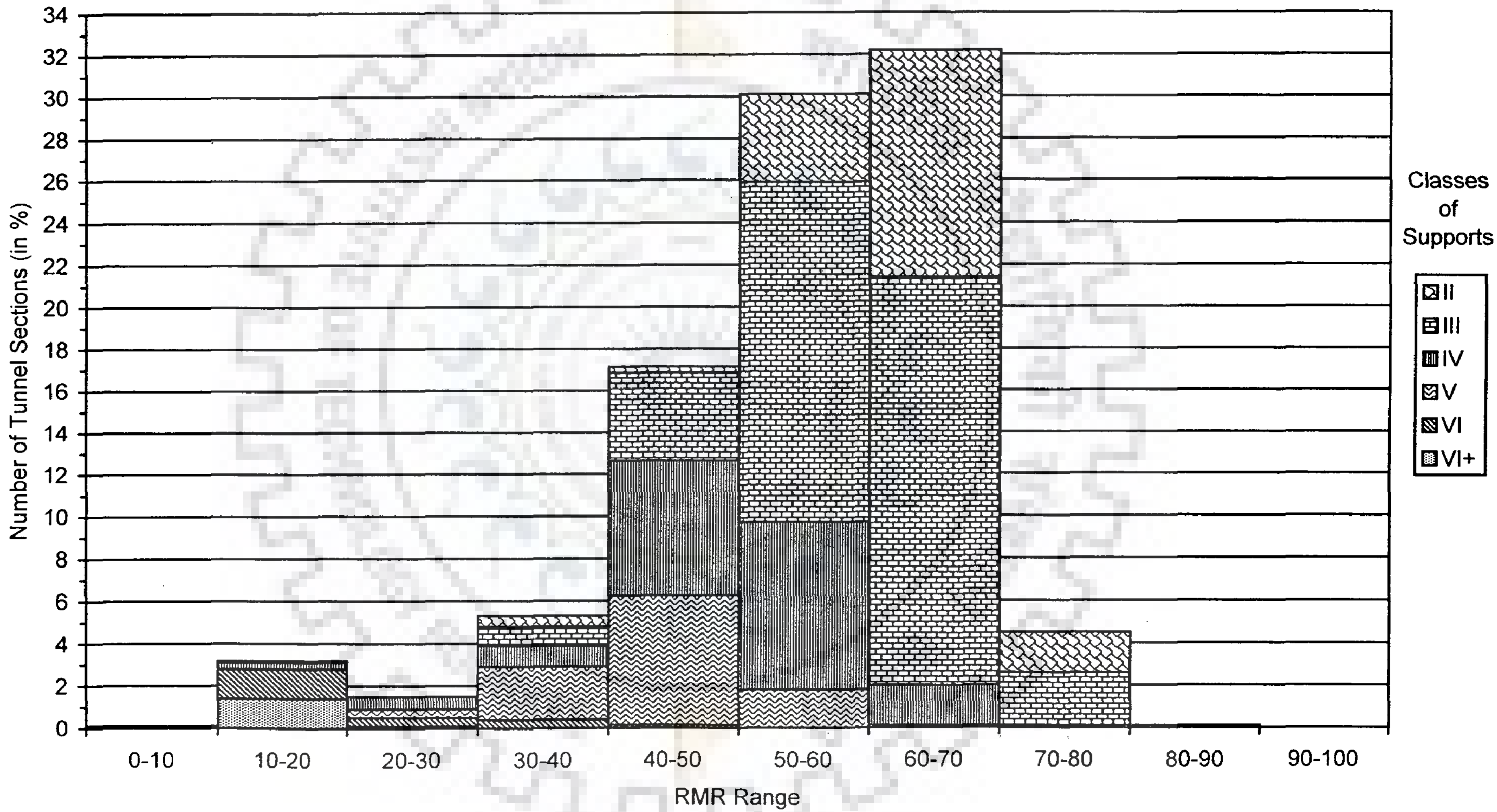
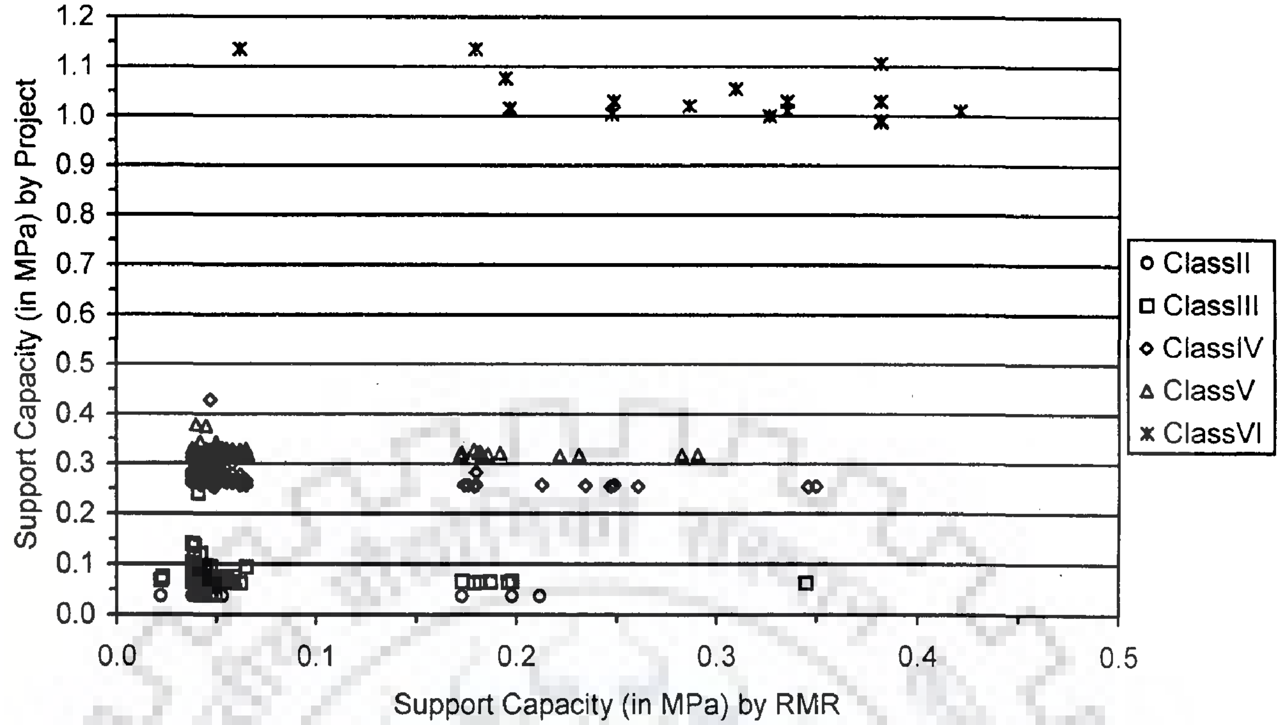
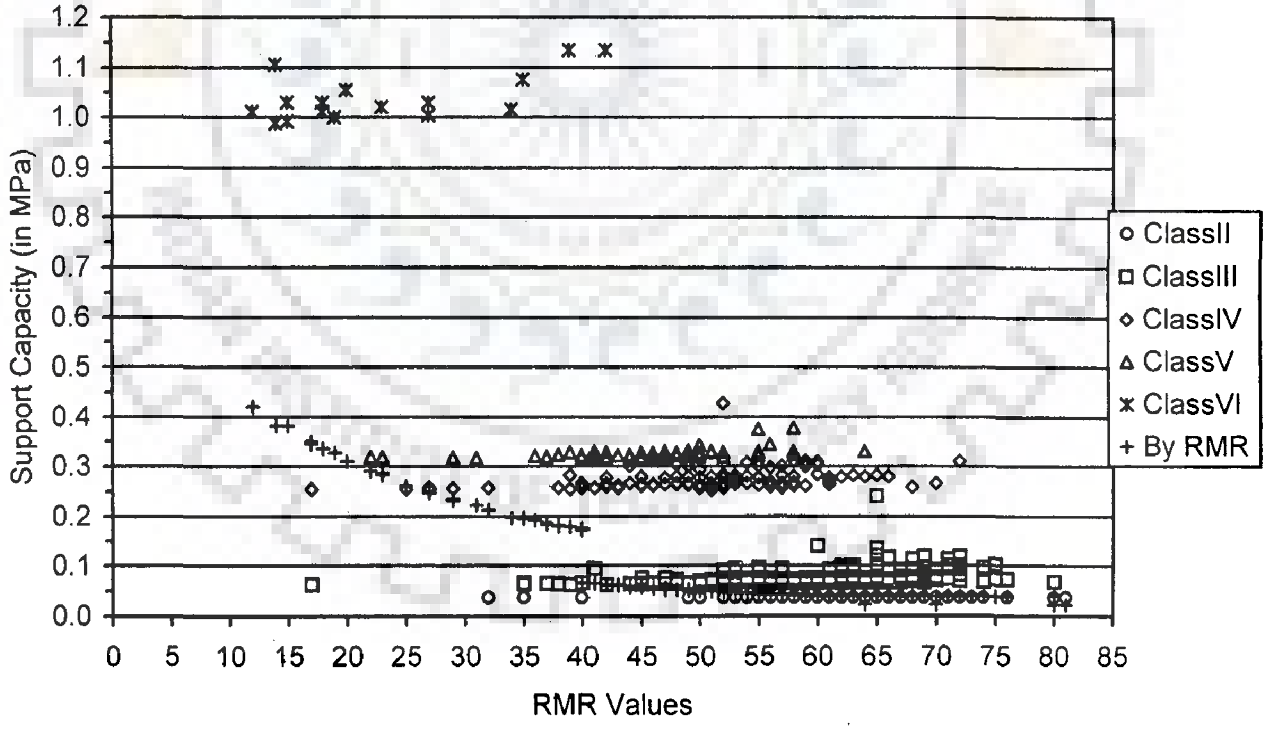


Fig. 7.14 Number of tunnel sections in classes of supports installed by the Project Authorities for RMR-values



(a)



(b)

Fig. 7.15 Comparison of capacity of support recommended by RMR and installed by the Project Authorities

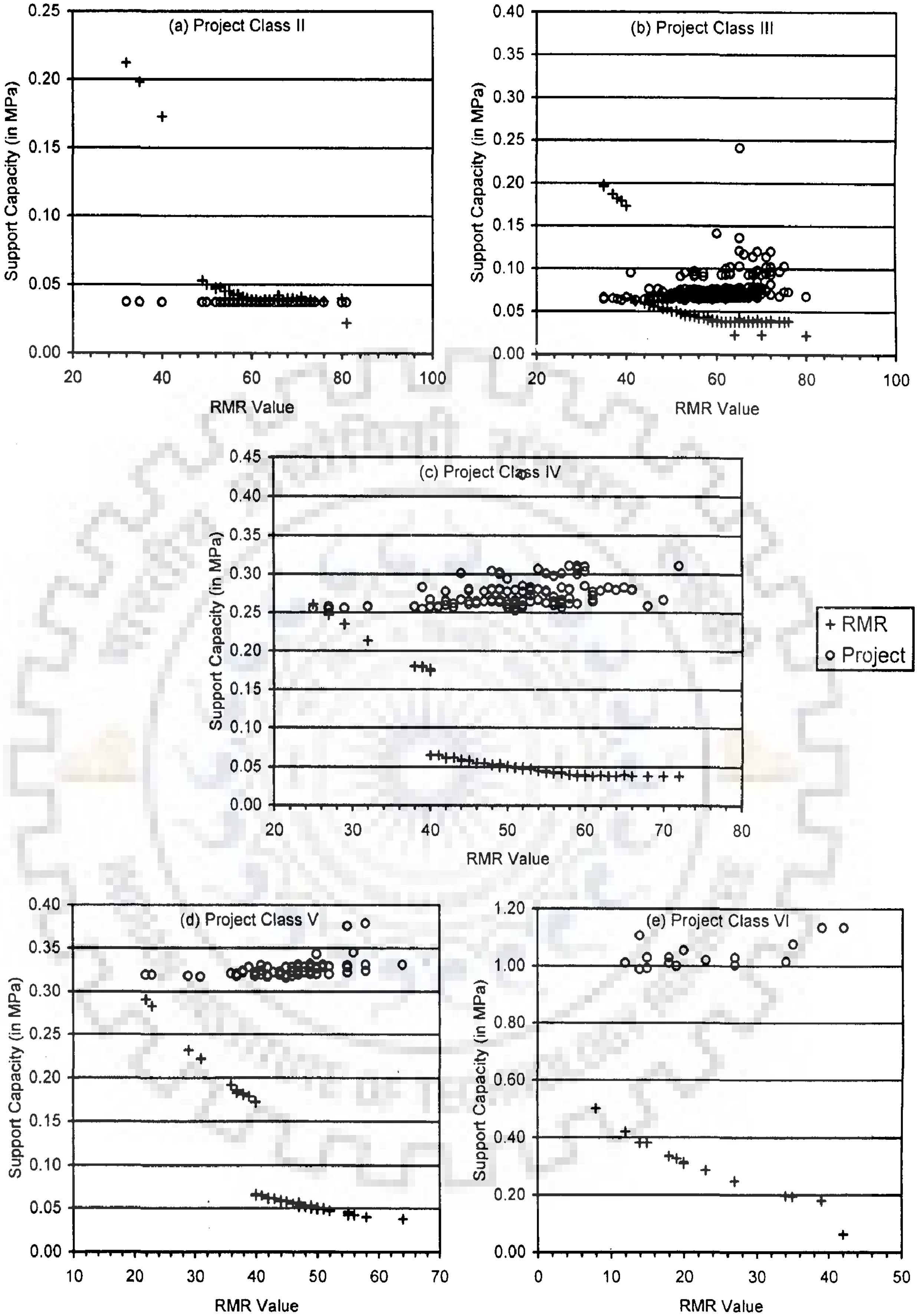
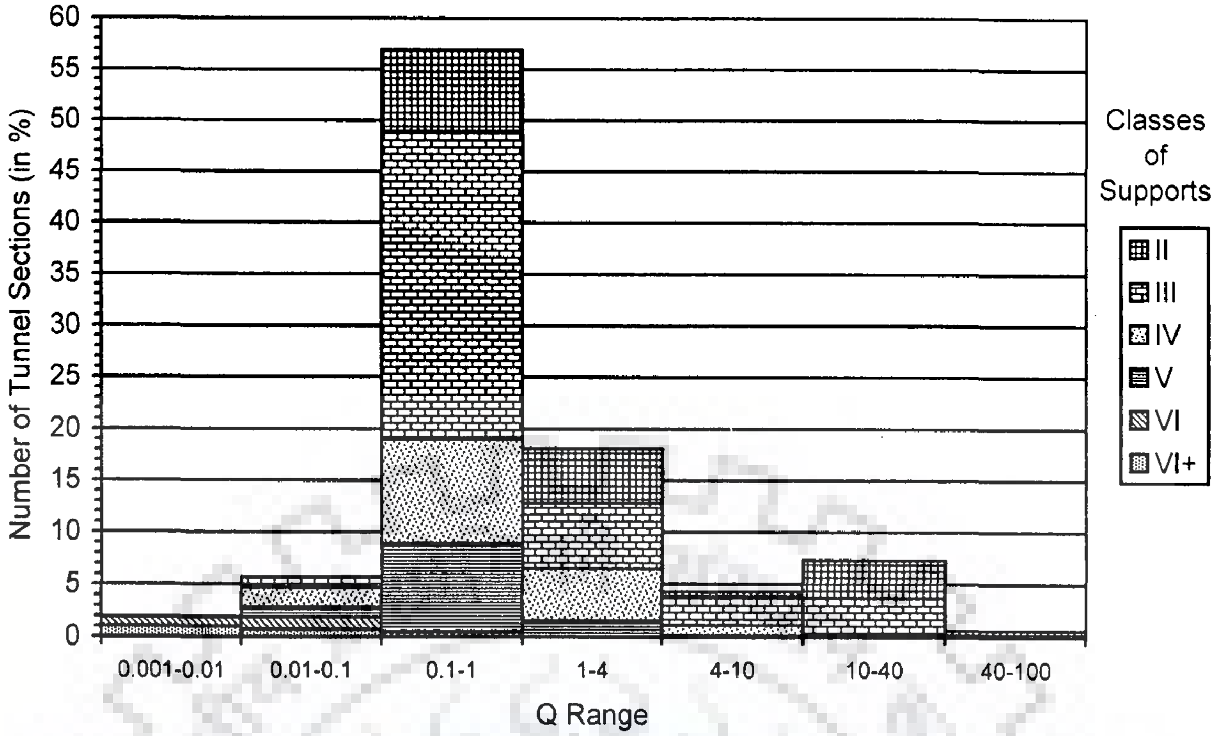


Fig. 7.16 Comparison of capacity of support recommended by RMR and installed by the Project Authorities for different project classes

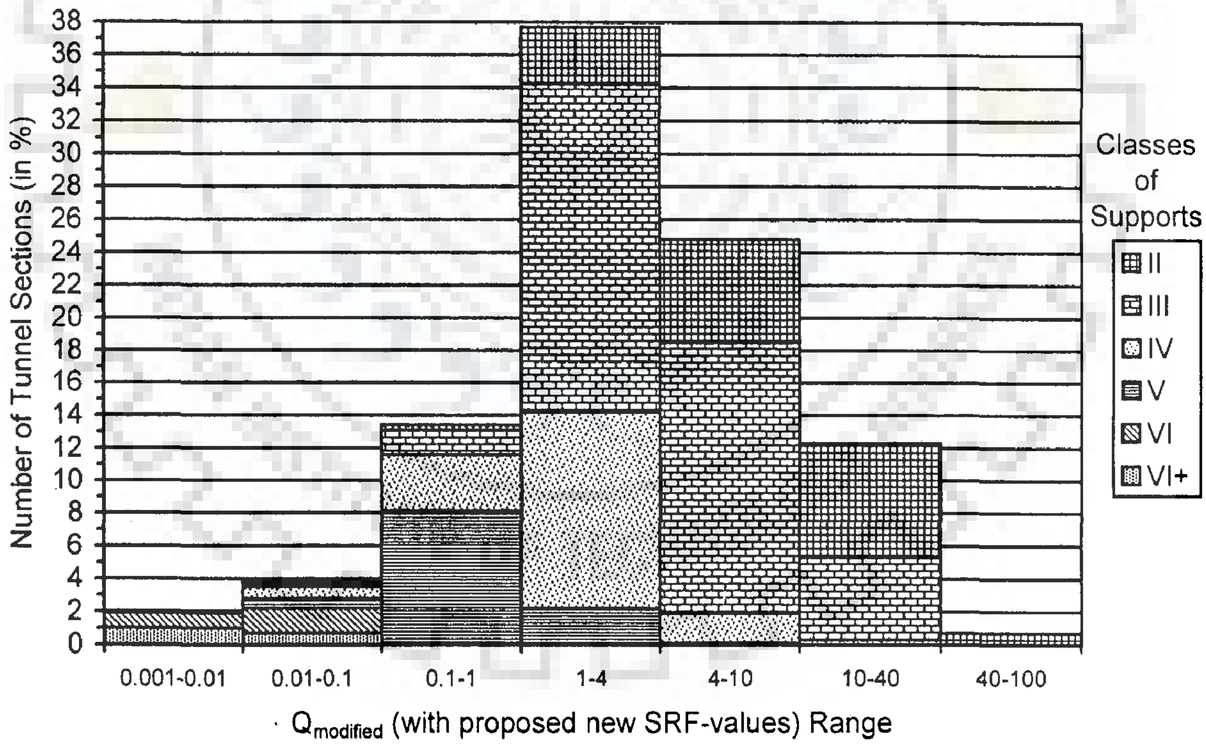
7.2.2.3 Q v/s Project

Figure 7.17 indicates number of tunnel sections in support classes by the Project Authorities for original and modified Q-values. Original Q-values ($Q_{original}$) are those values which have been computed by considering SRF-values under 'competent rock, rock stress problem' category as 9, 15 and 20 and modified Q-values ($Q_{modified}$) are those values which have been computed by considering new SRF-values proposed in this study as 1.5, 2.0 and 2.5 respectively. The basis of this is discussed in section 9.1.1 and its effect will be evident in subsequent paragraphs. From Figs. 7.17 a and b, it is clear that by reducing SRF-values, a significant number of sections shift towards higher Q-ranges. 'Very poor' (Q = 0.1 - 1) sections, about 56.9 percent earlier, now reduce to about 13.4 percent only, whereas 'Poor' (Q = 1 - 4), 'Fair' (Q = 4 - 10) and 'Good' (Q = 10 - 40) sections get increased from 16.1 to about 37.7 percent, from 4.3 to about 24.8 percent and from 7.3 to about 12.3 percent respectively. As discussed previously in this thesis, the Project Authorities have classified rock masses following the Q-system, it is appropriate here to compare the number of tunnel sections as per $Q_{original}$, $Q_{modified}$ and by the Project Authorities. The comparison is made in Fig. 7.18. From this figure, it is clear that there is large variation between supports by $Q_{original}$ and by the Project Authorities. Hence, it is more appropriate to use proposed new SRF-values for moderately jointed rock masses, as there is better match between supports by $Q_{modified}$ and by the Project Authorities.

As indicated in Figs. 7.17 b and 7.18 majority of the tunnel sections fall in Q range of 0.1 - 40. In 10 - 40 range, about 7 out of 12.3 percent are Class II, in 4 - 10 range about 16.6 out of 24.8 percent are Class III, in 1 - 4 range about 12.0 out of 37.7 percent are Class IV and in 0.1 - 1 range about 8 out of 13.4 percent are Class V. Rest of the sections in these Q ranges are one class better (less support). These better rocks, though classified following the Q-system, have been provided actually one class lower (more support) supports. Therefore, this slight discrepancy



(a) Q-values



(b) $Q_{modified}$ (with proposed new SRF-values) - values

Fig. 7.17 Number of tunnel sections in classes of supports installed by the Project Authorities for Q-values

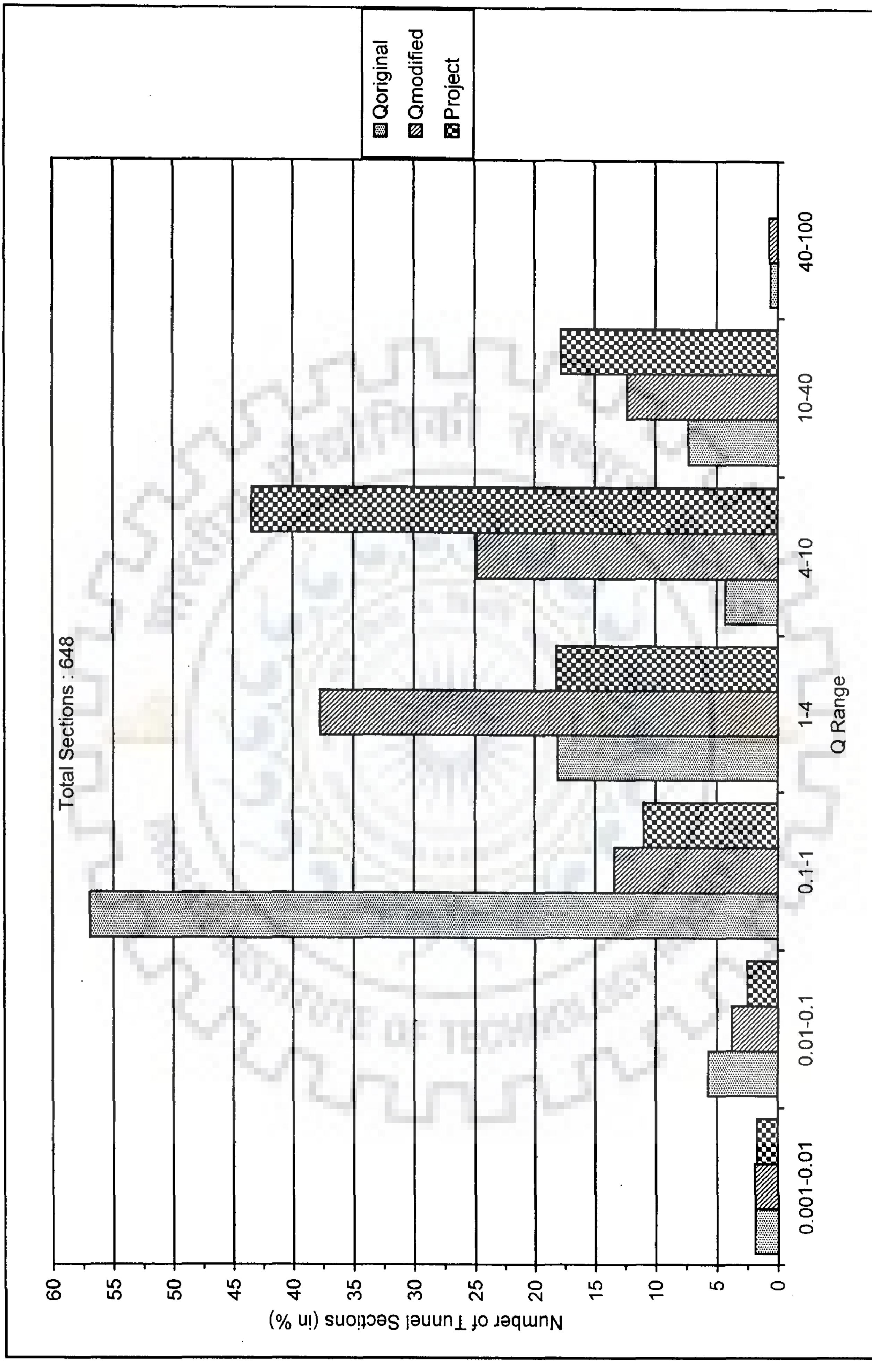


Fig. 7.18 Comparison of number of tunnel sections in Q ranges

between $Q_{modified}$ and the Project Authorities disappears with minor variations since Q is very sensitive to the selected SRF-value.

The Project Authorities have classified and designated the rock masses using the Q-system. Later, following the same designation they tried to choose supports using the RMR-system, which seems unsound as indicated in Table 7.6

Table 7.6 Q-Values and Comparison of Supports

Q Range	Designation using Q-system	Project Class	Project Support	RMR Support
100 - 40	Very good	Very good (I)	Unsupported	Unsupported
40 - 10	Good	Good (II)	50 mm thick shotcrete + spot bolting	50 mm thick shotcrete + rock bolts @ 2.5 m locally.
10 – 4	Fair	Fair (III)	100 mm thick shotcrete + rock bolts @ 1.5 m	50-100 mm thick shotcrete + rock bolts @ 1.5-2 m
4 - 1	Poor	Poor (IV)	100 mm thick shotcrete + rock bolts @ 1-1.5 m + ISHB 150 @ 1.0 m	100-150 mm thick shotcrete + rock bolts @ 1-1.5 m + light to medium ribs @ 1.5 m
1 - 0.1	Very poor	Very poor (V)	100 mm thick shotcrete + rock bolts @ 1-1.5 m + ISHB 150 @ 0.75 m	150-200 mm thick shotcrete + rock bolts @ 1-1.5 m + medium to heavy ribs @ 0.75 m

Since the RMR-system has not recommended supports beyond 'Very poor' class, the Project Authorities have designed supports at their own as shown in APPENDIX - VIII. But designation of project class literally follow the Q-system and then designing supports following the RMR-system may be inappropriate because literally Q and RMR are not same as indicated in Table 7.7.

Table 7.7 Comparison of Designation of Class following Q and RMR Systems

Q Range	Designation of Class following	
	Q-system	RMR-system
100 - 40	Very good	Very good
40 - 10	Good	Good
10 - 4	Fair	Good
4 - 1	Poor	Fair
1 - 0.1	Very poor	Poor
0.1 - 0.01	Extremely poor	Very poor

From Table 7.7, it may be noted that first two classes in both the systems have same designation i.e. 'Very Good' and 'Good', but thereafter the Q-system designates rock class lower by one class as compared to the RMR-system. Therefore, if one goes by the designation of RMR for designing the support system, correspondence has to be according to Table 7.7, not merely by Q-designation. For example, if support is designed for 'Fair' rock class following RMR, Q-values then fall in 4 - 1 range i.e. 'Poor', not 'Fair' rock class following the Q-system according to the comparison shown in Table 7.7.

Therefore, by applying this logic, discrepancy observed in number of tunnel sections in $Q_{modified}$ and the Project Authorities (Fig. 7.18) is eliminated. Further, it can be observed that number of tunnel sections falling in each Q range according to $Q_{modified}$ and the Project Authorities is comparable.

Figures 7.19 and 7.20 compare support capacity by Q and the Project Authorities. Figure 7.19 is an x-y scatter of both support capacities for different support classes by the Project

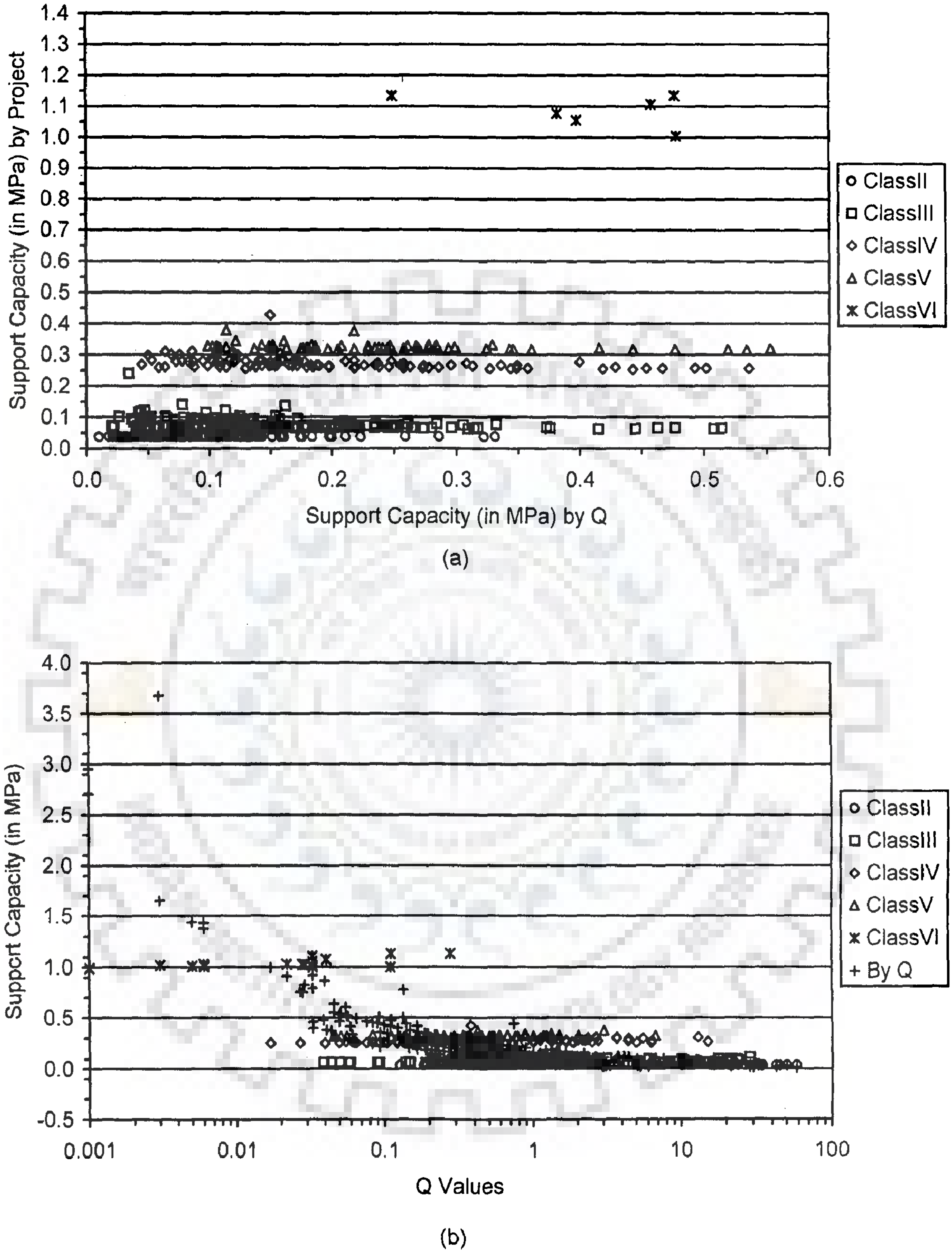


Fig. 7.19 Comparison of capacity of support recommended by Q and installed by the Project Authorities

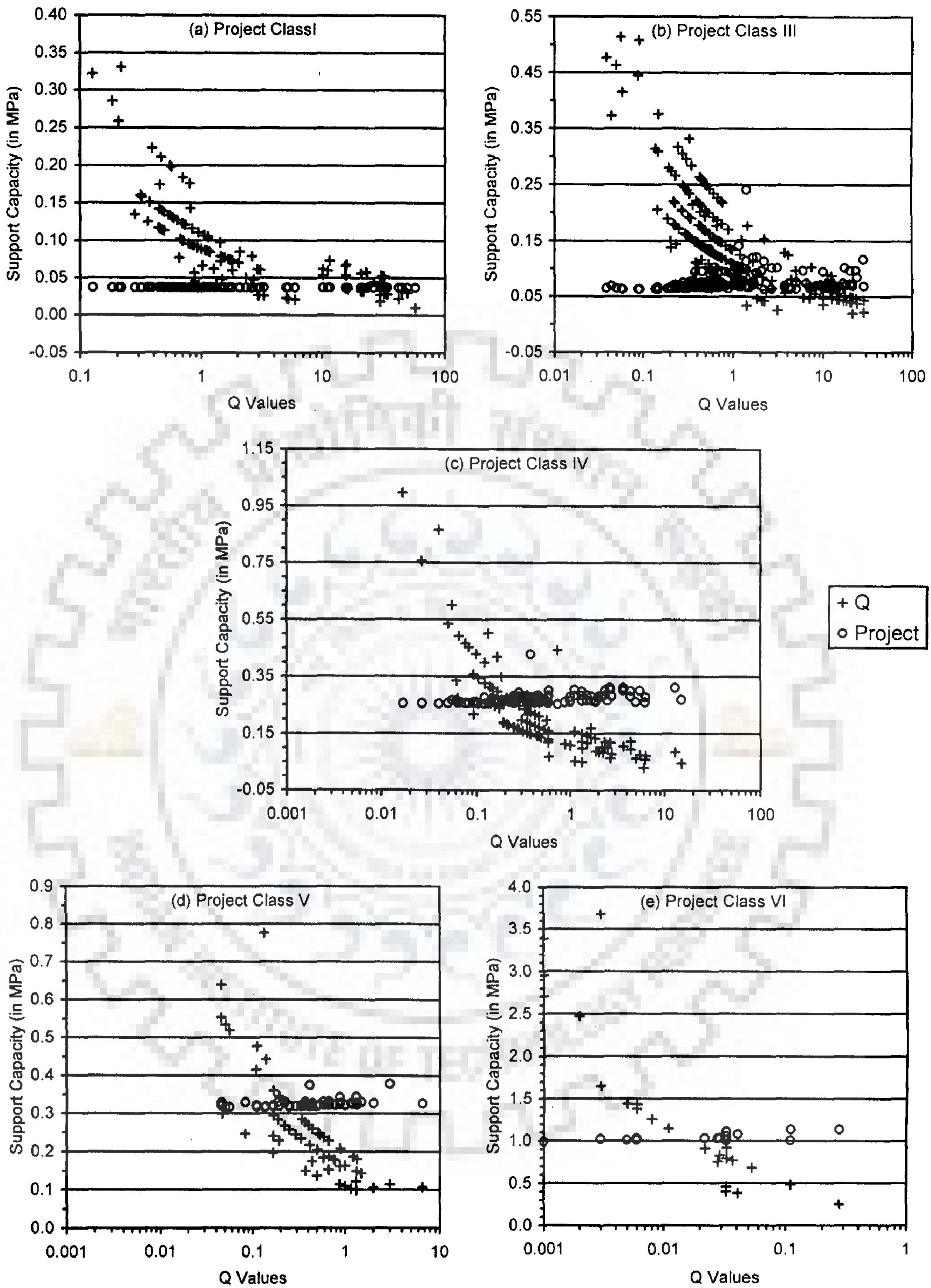


Fig. 7.20 Comparison of capacity of support recommended by Q and installed by the Project Authorities for different project classes

Authorities. The support capacity by Q ranges from 0.01 MPa to 3.68 MPa whereas for Class II to VI, capacity ranges from 0.037 MPa to 1.135 MPa as indicated in Table 7.5 also. Figure 7.19 b compares support capacity against Q-values for all project classes and the detailed comparison for all project classes separately is shown in Fig. 7.20. Figure 7.19 b shows that generally capacities by both Q and project classes are matching each other, though on a detailed study of Fig. 7.20 indicates that for Class IV, V and VI, they are intersecting each other. In Class III, for higher Q-values, there is similarity but at lower Q-values, the Project Authorities provided less support and the same is true for Class II as well.

Table 7.8, containing tunnel sections in squeezing ground conditions, indicates the actual behaviour of supports installed at various sections. Graphical presentation of the table has also been shown in Fig. 7.21 in which comparison has been made for pressures estimated from Q before and after applying correction factors given by Singh et al. (1992) and capacity by the Project Authorities for squeezing ground conditions. It may be noted from both Table 7.8 and Fig. 7.21 that majority of sections are unstable since pressures after corrections are more than the support capacities. In Ch. 21240-21247 m, where mild squeezing conditions exist, bending/twisting of ribs has been observed due to the fact that pressure after corrections is more than the support capacity notwithstanding the fact that pressure before corrections is less than the support capacity. It strengthens the viewpoint by Singh et al. (1992) that corrections for overburden, closure and time after excavation were necessary in squeezing ground conditions. It is feared that other sections might also show signs of instability in future as commented in the table where pressures exceeded capacities of supports installed at the site.

It is interesting to note that in Ch.26105-26108 m, Ch. 26108-26112 m, Ch. 26126-26133 m and Ch. 26545-26601 m, where stable condition is predicted, bending/twisting of ribs have been noticed. Supports installed in these sections are as per Class VI and VI+. Reasons for this may be

Table 7.8 Comparison of Pressure Estimated from Q before and after Corrections and Behaviour of Support Installed by the Project Authorities in Squeezing Ground Conditions

S. No.	Chainage (in m)	Time after Excavation (in days)	Q Value	Pressure before Correcions (in MPa)	Pressure after Correcions (in MPa)	Project Capacity (in MPa)	Squeezing Condition	Support Behaviour	Predicted Stability
1	20390-20410	239	0.417	0.218	0.668	0.258	Mild	safe	Unstable
2	20918-20925	929	0.194	0.343	1.209	0.319	Mild	safe	Unstable
3	20988-21046	983	0.333	0.235	0.867	0.321	Mild	safe	Unstable
4	21240-21247	1095	0.167	0.241	0.476	0.321	Mild	Bending/Twisting of Ribs	Unstable
5	21247-21310	1102	0.333	0.235	0.751	0.320	Mild	safe	Unstable
6	23894-23901	732	0.25	0.258	0.646	0.320	Mild	safe	Unstable
7	26093-26105	361	0.033	0.919	1.376	1.016	Mild	safe	Unstable
8	26105-26108	326	0.033	0.398	0.856	1.055	Mild	Bending/Twisting of Ribs	Stable
9	26108-26112	158	0.033	0.458	0.583	1.107	Mild	Bending/Twisting of Ribs	Stable
10	26126-26133	97	0.028	0.753	0.774	1.030	Mild	Bending/Twisting of Ribs	Stable
11	26398-26407	298	0.053	0.682	1.172	VI+	Mild	Bending/Twisting of Ribs	
12	26415-26478	317	0.037	0.769	1.072	VI+	Mild	Bending/Twisting of Ribs	
13	26545-26601	457	0.022	0.91	0.973	1.030	Mild	Bending/Twisting of Ribs	Stable
14	26623-26626	497	0.029	0.827	1.167	1.030	Mild	Bending/Twisting of Ribs	Unstable
15	21230-21240	1095	0.056	0.519	0.666	0.317	Moderate	Bending/Twisting of Ribs	Unstable
16	26361-26398	270	0.005	1.438	1.801	VI+	Moderate	Bending/Twisting of Ribs	
17	26478-26506	250	0.006	1.378	1.398	VI+	Moderate	Bending/Twisting of Ribs	
18	26522-26545	476	0.006	1.378	1.479	1.011	Moderate	Bending/Twisting of Ribs	Unstable
19	26601-26608	542	0.005	1.438	1.152	1.011	Moderate	Bending/Twisting of Ribs	Unstable
20	26620-26623	257	0.006	1.433	1.284	1.030	Moderate	Bending/Twisting of Ribs	Unstable
21	26112-26120	151	0.001	2.707	2.934	0.988	High	Bending/Twisting of Ribs	Unstable
22	26120-26126	109	0.003	1.651	1.089	1.021	High	Bending/Twisting of Ribs	Unstable
23	26506-26522	542	0.001	2.952	7.437	VI+	High	Bending/Twisting of Ribs	
24	26608-26620	556	0.001	2.952	7.071	0.992	High	Bending/Twisting of Ribs	Unstable

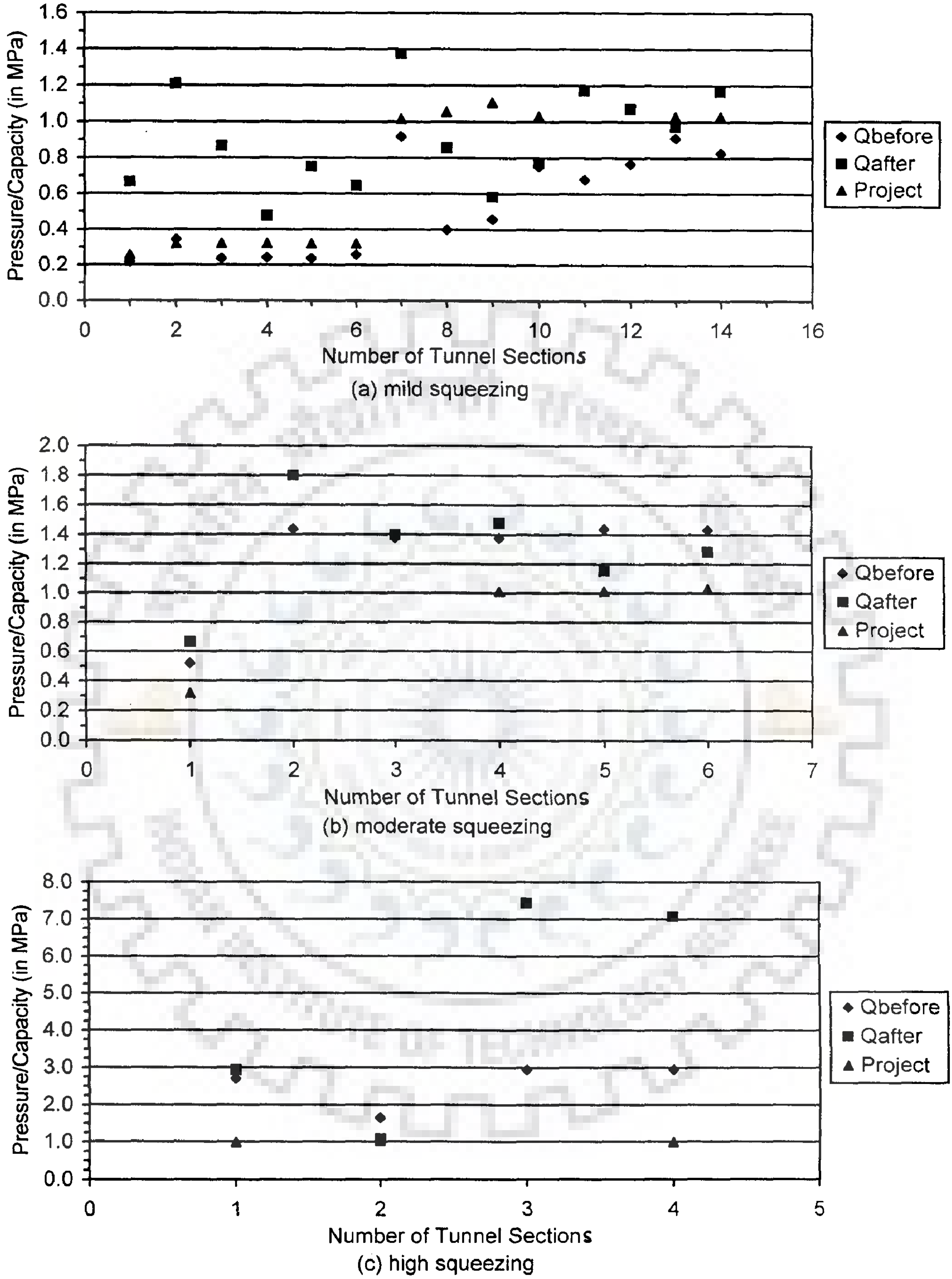


Fig. 7.21 Comparison of pressure estimated from Q before and after corrections and capacity of support installed by the Project Authorities for squeezing ground conditions

that rock mass might be poorer than recorded and stiffer supports installed too early thereby inviting higher rock loads in these sections. The observed tunnel closure is 2.5 percent of tunnel diameter.

In moderate and high squeezing sections, pressures are excessive and even Class VI, VI+ supports may prove unstable as bending/twisting of ribs is observed.

7.2.2.4 RMi v/s Project

Figure 7.22 indicates number of tunnel sections in support classes by the Project Authorities for RMi-values. It may be noted that almost all the sections are falling in RMi range of 0.1 - 10. A larger chunk of about 31.2 percent of Class III is lying in 1 - 10 range. Since RMi ranges from 0.1 - 10, it is quite obvious that it covers all the support classes provided by the Project Authorities in this range.

Figures 7.23 and 7.24 compare support capacity by RMi and the Project Authorities. Figure 7.23 a is an x-y scatter of both support capacities for different support classes by the Project Authorities. The support capacity by RMi ranges from 0.022 MPa to 1.08 MPa whereas for Class II to VI, capacity ranges from 0.037 MPa to 1.135 MPa as indicated in Tables 7.5 also. In supports where RMi recommended ISC or SDSC type supports in very weak rock masses, no quantifiable capacity could be estimated. Figure 7.23 b compares support capacity against RMi-values for all project classes and the detailed comparison for all project classes separately is shown in Fig. 7.24. In Fig. 7.24, it is seen that majority RMi capacities lie in 1 - 10 RMi value range, some values in Class IV and a few values in Class V have been compared. It is because in 0.1 - 1 RMi range, ISC or SDSC supports have been recommended. In Fig. 7.24 a and 7.24 b, most of the RMi capacities are lying over capacities of supports by the Project Authorities indicating that for better quality

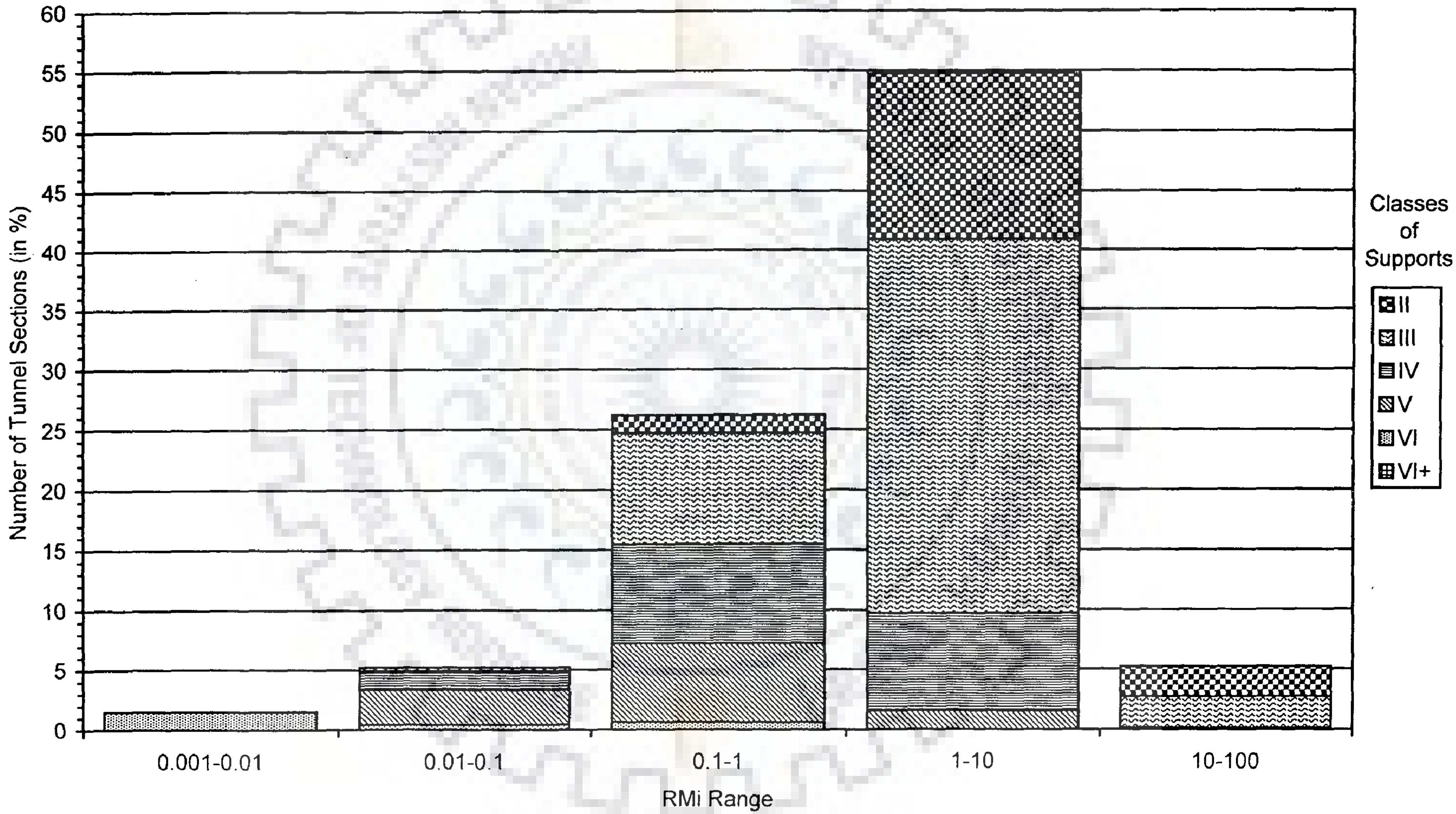


Fig. 7.22 Number of tunnel sections in classes of supports installed by the Project Authorities for RMi-values

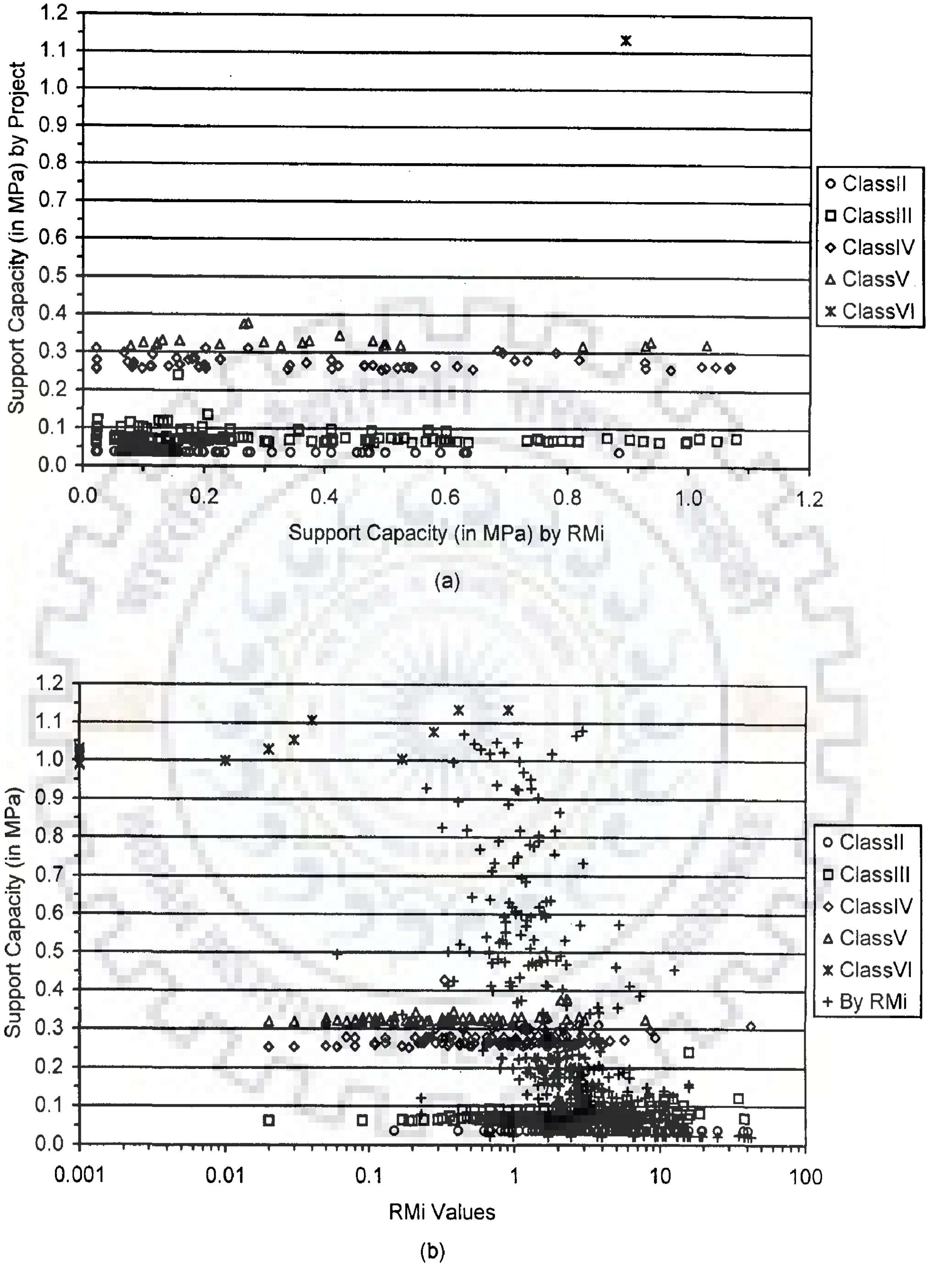


Fig. 7.23 Comparison of capacity of support recommended by RMi and installed by the Project Authorities

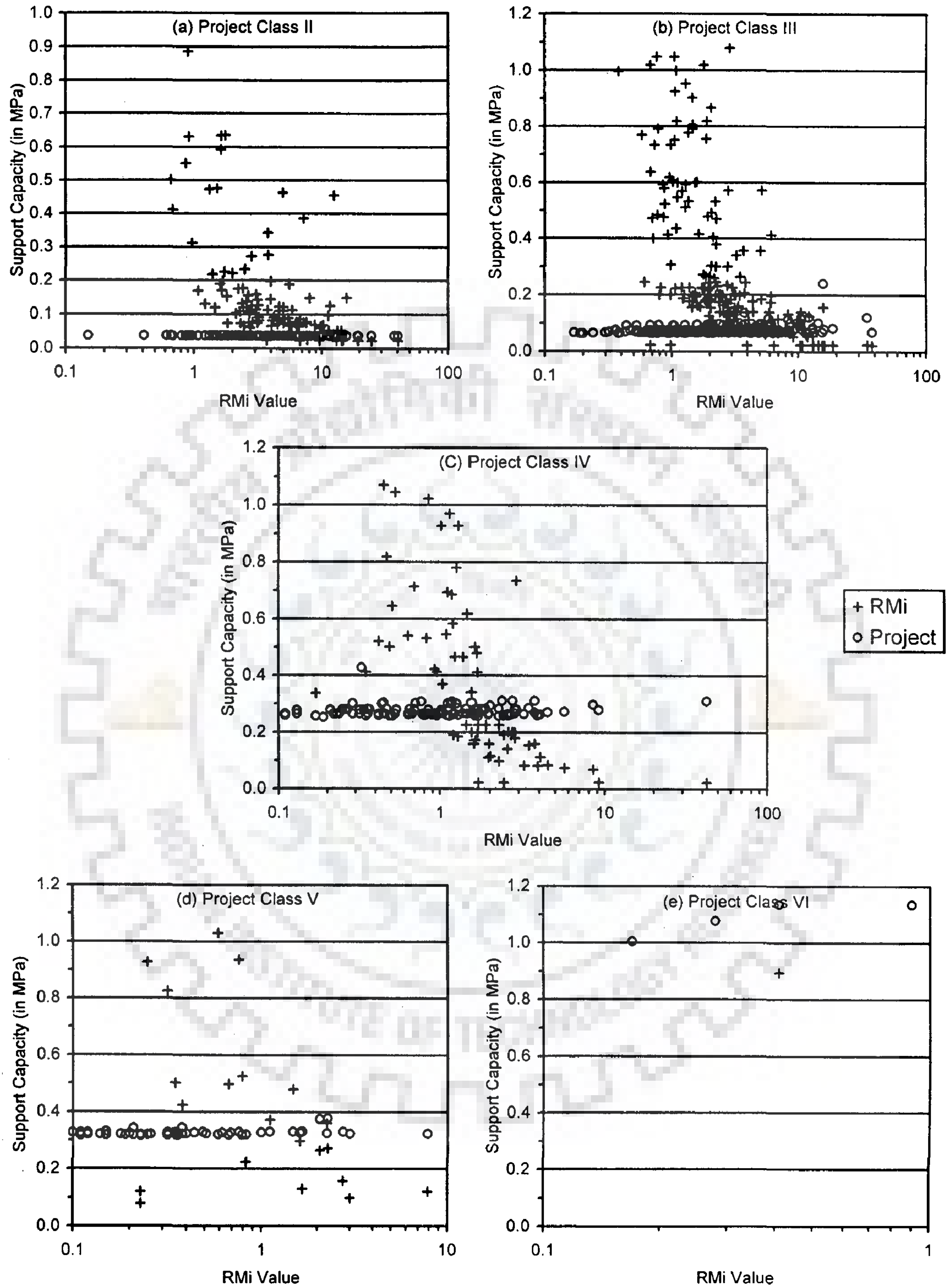


Fig. 7.24 Comparison of capacity of support recommended by RMi and installed by the Project Authorities for different project classes

rocks, RMi recommended very heavy supports. In Class IV and V, RMi capacities are evenly scattered above and below capacities of supports by the Project Authorities.

7.2.2.5 N v/s Project

Figure 7.25 indicates number of tunnel sections in support classes by the Project Authorities for N-values. In this figure, it is observed that majority of the sections fall in N range of 1 - 40. In 10 - 40 range Classes II and III, in 4 - 10 range Classes II, III and IV and in 1 - 4 range Classes II, III, IV and V are placed which indicates that support classes follow N-values. Figure 7.26 shows comparison of pressure estimated from N and capacity by the Project Authorities for non-squeezing ground conditions. As mentioned above and also shown in Fig. 7.26, capacity by the Project Authorities in 10 - 40 range is 0.037 MPa and 0.07 MPa i.e. Class II and III respectively. Therefore, in this range the sections which have been supported by Class III are stable but those with Class II are unsafe according to N. Similarly in 4 - 10 range Classes II and III are unsafe but Class IV is excess and in 1 - 4 range Classes II and III are unsafe but IV and V are over safe. Actually supports were stable.

Figure 7.27 shows comparison of N and capacity of support by the Project Authorities for squeezing ground conditions. In Fig. 7.27 a (mild squeezing), support capacity is 0.325 MPa and 1.135 MPa i.e. Class V and VI whereas in other figures it is Class VI alone. Class V is unsafe according to N whereas in mild squeezing Class VI is excess. In moderate squeezing, Class VI is according to N whereas in high squeezing conditions N predicts abnormally high support pressures.

Comparison of Fig. 7.25 with Fig. 7.17 indicates a striking similarity in the two figures, which is quite obvious since N is nothing but the stress-free Q. The N ranges chosen in Fig. 7.25 are same as those for Q for better comparison. The similarity of these two figures underlines the fact that if SRF is selected judiciously, the two systems almost behave identically.

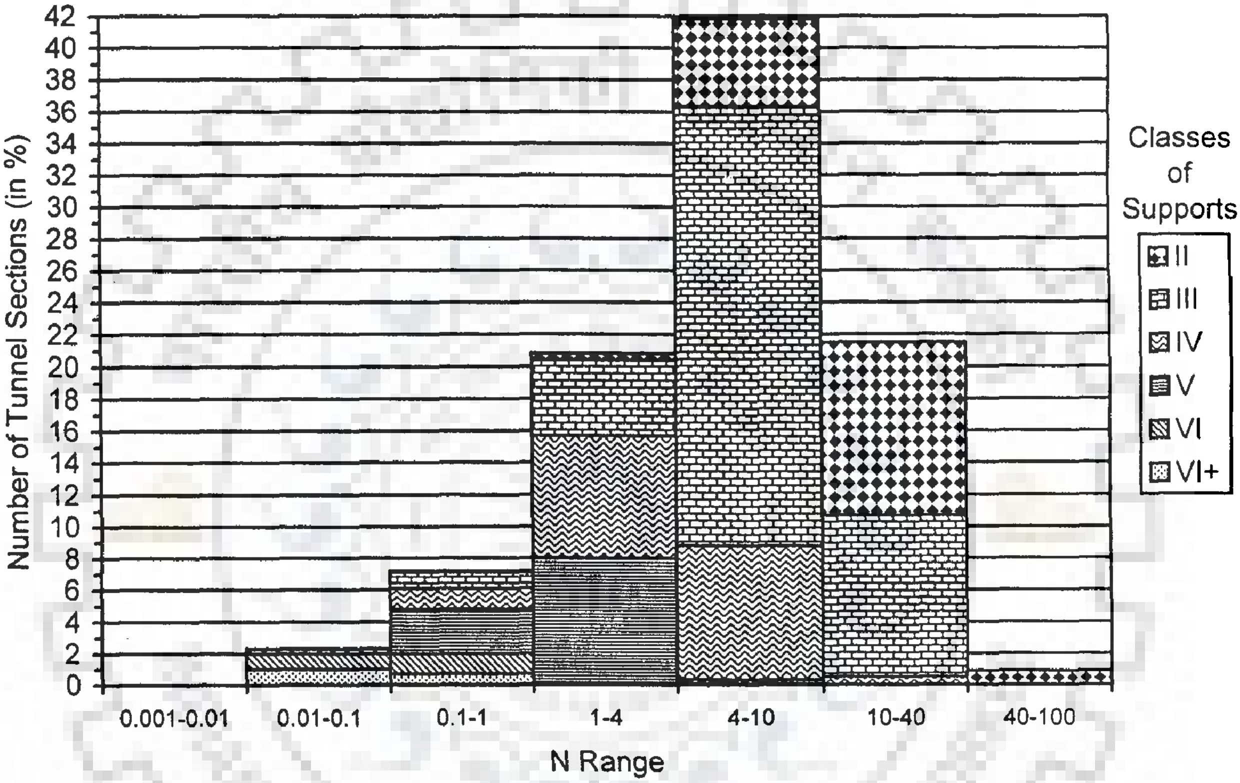


Fig. 7.25 Number of tunnel sections in classes of supports installed by the Project Authorities for N-values

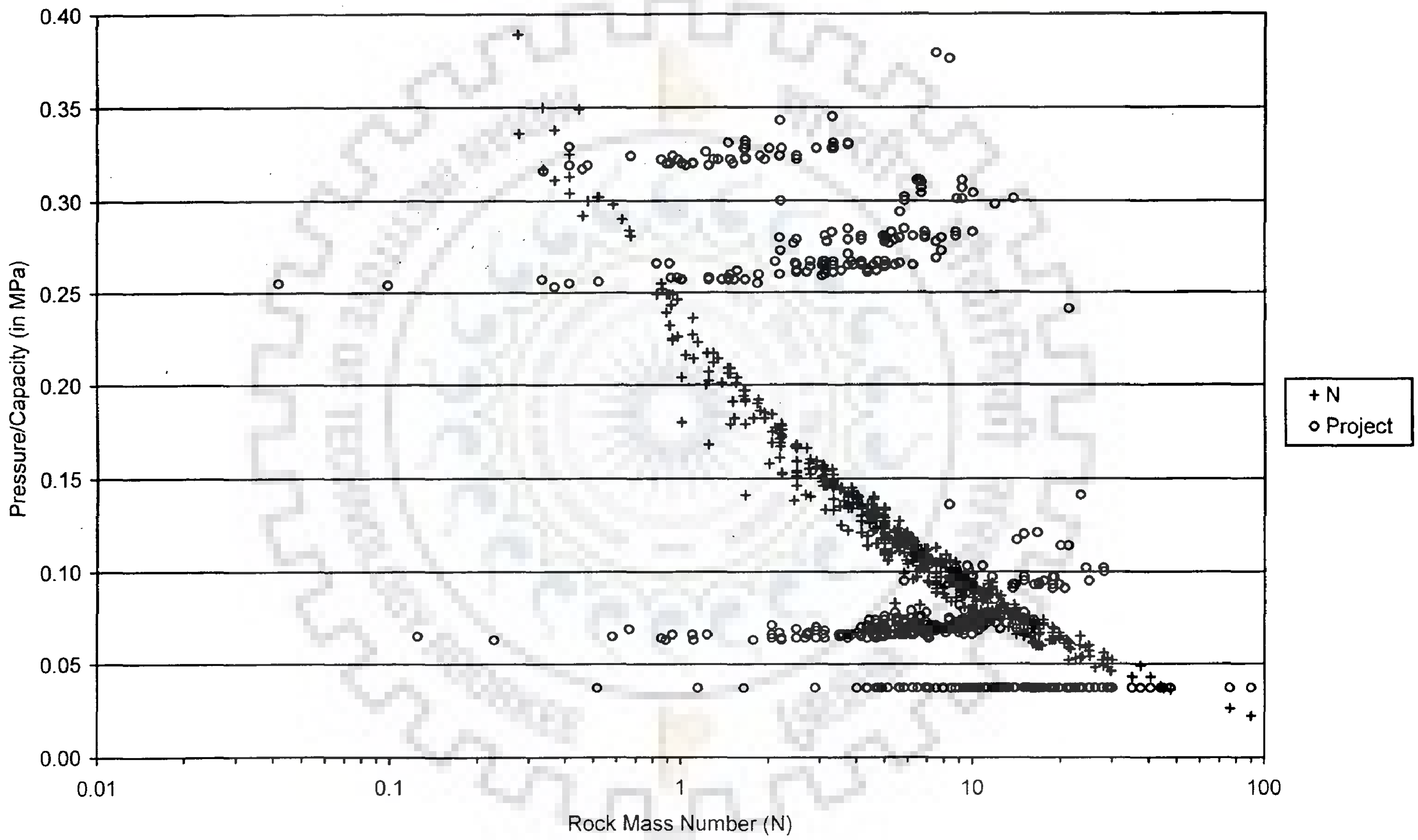
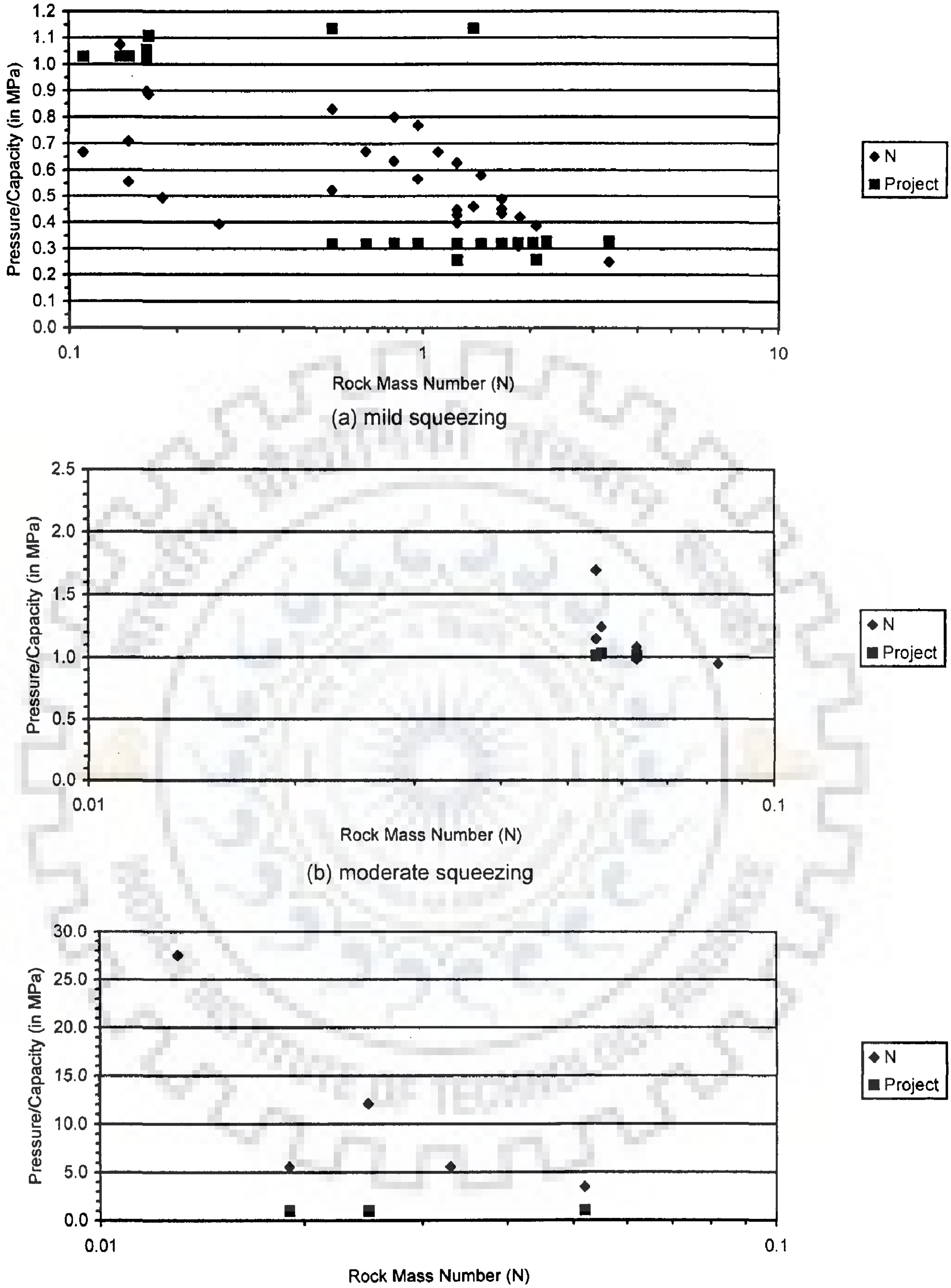


Fig. 7.26 Comparison of pressure estimated from N and capacity of support installed by the Project Authorities for non-squeezing ground conditions



(a) mild squeezing

(b) moderate squeezing

(c) high squeezing

Fig. 7.27 Comparison of pressure estimated from N and capacity of support installed by the Project Authorities for squeezing ground conditions

7.2.2.6 RCR v/s Project

Figure 7.28 indicates number of tunnel sections in support classes by the Project Authorities for RCR-values. In this figure, it may be observed that majority of the sections fall in RCR range of 40 - 70. This figure is similar to Fig. 7.14, as RCR has been derived from RMR only. The comments in section 7.2.2.2 are valid here also, i.e. there appears to be degradation by one class for 40 - 70 range.

Figure 7.29 shows comparison of pressure estimated from RCR and capacity by the Project Authorities for non-squeezing ground conditions. In range 50 - 80, Class II and III have been installed, obviously indicating that one is unsafe and other is excess according to RCR. Below 50, pressure estimated from RCR, are very low compared to installed supports.

Figure 7.30 deals with squeezing conditions and indicates that pressures estimated from RCR are low as compared to the Project Authorities.

Tables 7.9 and 7.10 compare capacity of support recommended by classification systems and those installed by the Project Authorities in non-squeezing and squeezing ground conditions respectively involving a total of 562 and 35 sections respectively. The pressures estimated from N and RCR have been considered for comparison, as they do not recommend support systems. Number of sections falling in each group of inequality has been indicated in the table, on the basis of which a conclusion has been drawn in the last columns of Tables 7.9 and 7.10.

Summarizing above and from Tables 7.9 and 7.10 a general trend, which suggests position of support installed by the Project Authorities may be given as follows:

Support Capacity (Non-Squeezing): $C_{RMR} < P_{RCR} < \mathbf{C_{PROJECT}} < P_N = C_Q < C_{RSR} < C_{RMi}$

Support Capacity (Squeezing): $C_{RMR} < P_{RCR} < C_Q < \mathbf{C_{PROJECT}} < P_N < C_{RMi}$

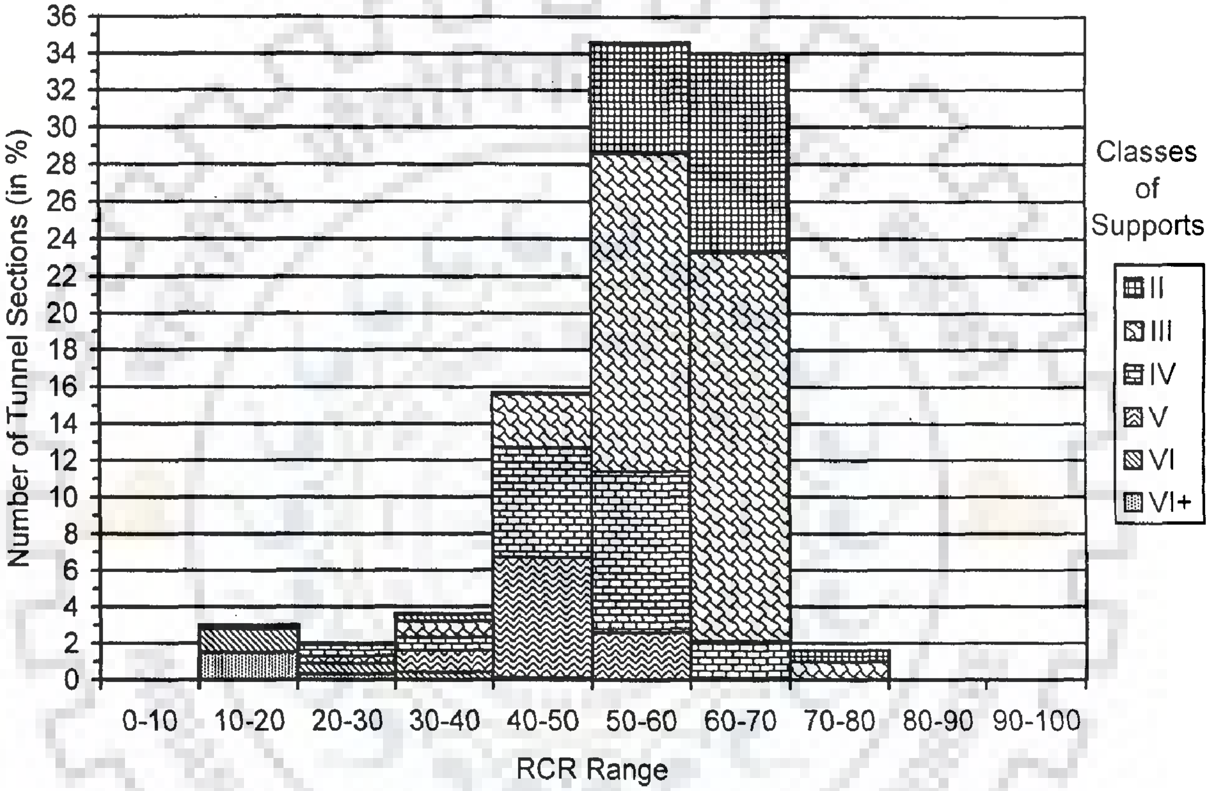


Fig. 7.28 Number of tunnel sections in classes of supports installed by the Project Authorities for RCR-values

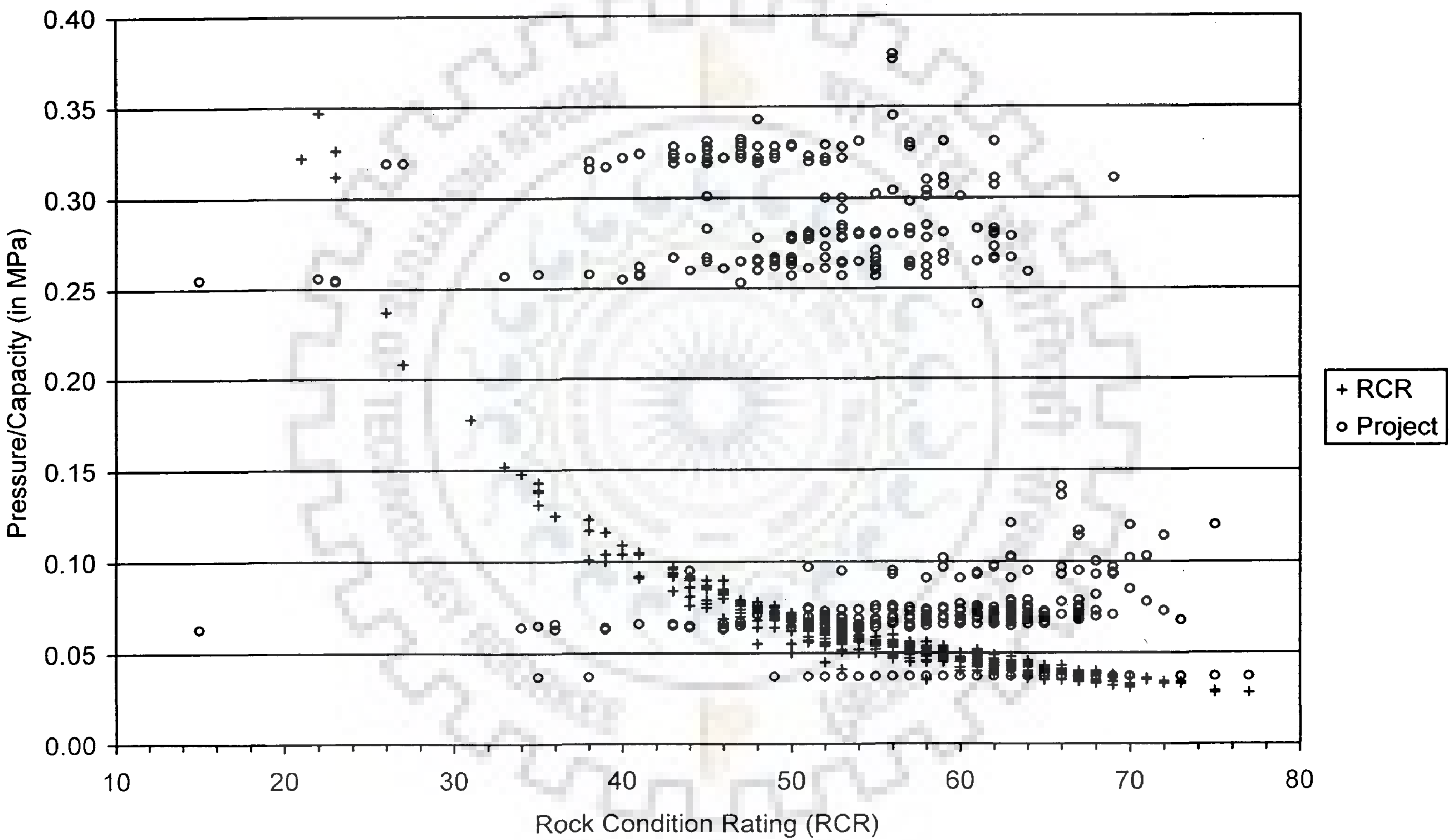
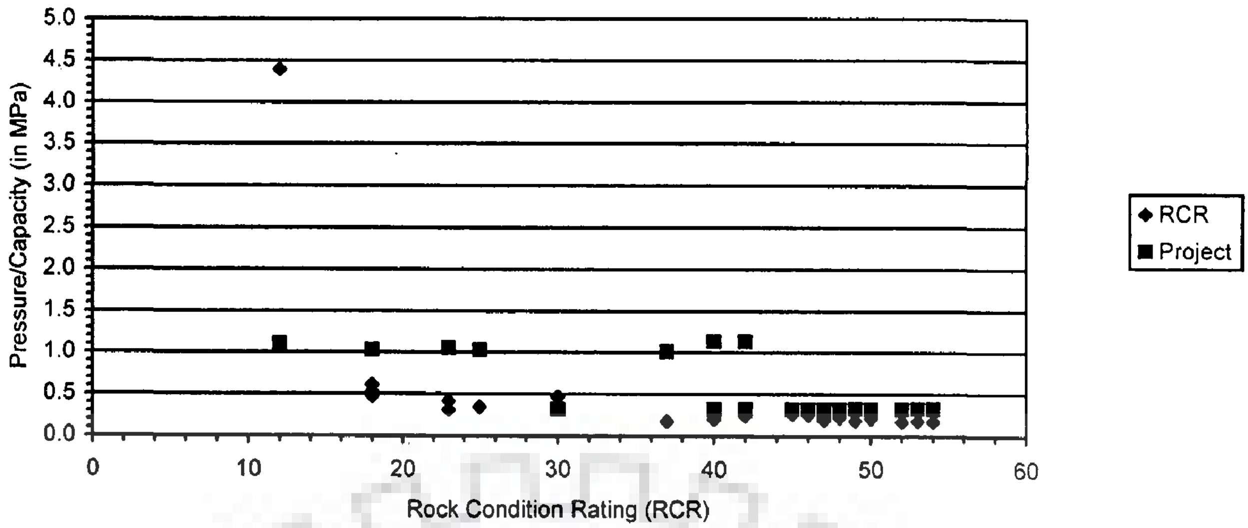
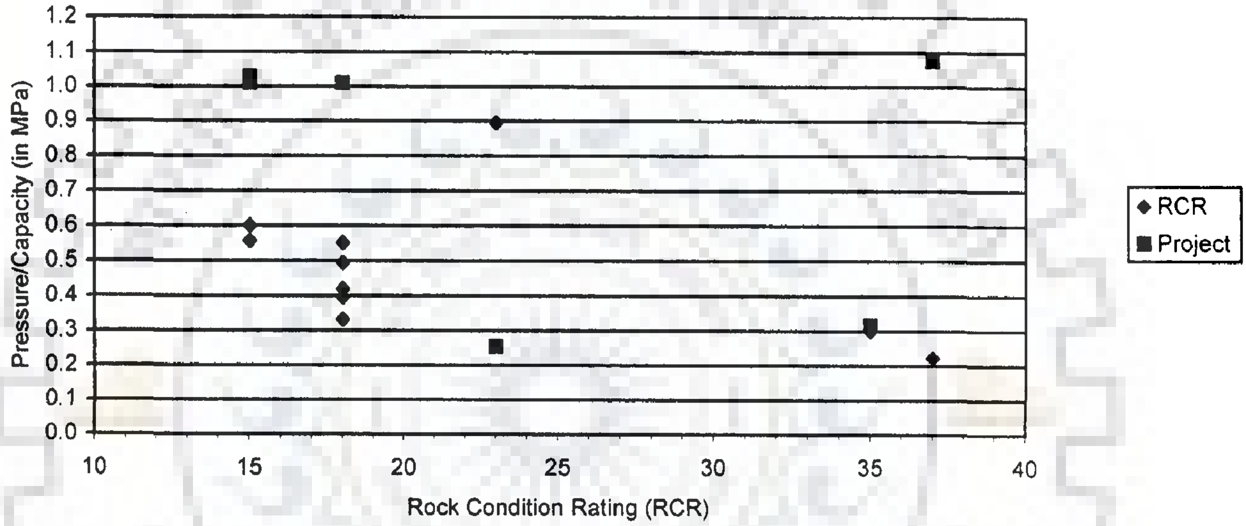


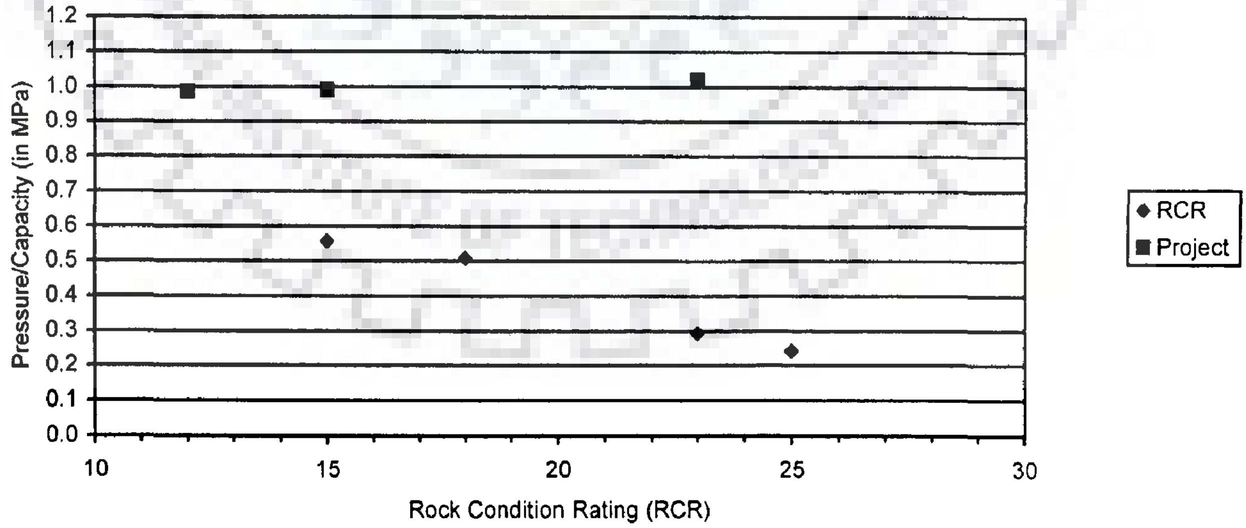
Fig. 7.29 Comparison of pressure estimated from RCR and capacity of support installed by the Project Authorities for non-squeezing ground conditions



(a) mild squeezing



(b) moderate squeezing



(c) high squeezing

Fig. 7.30 Comparison of pressure estimated from RCR and capacity of support installed by the Project Authorities for squeezing ground conditions

Table 7.9 Comparisons of Capacity of Support Recommended by Classification Systems and Installed by the Project Authorities in Non-Squeezing Ground Conditions

S. No.	Classification Systems	Inequality	Maximum Deviation (in MPa)	Number of Sections (in percent)	Conclusion
1.	Project and RMi	$C_{Project} < C_{RMi}$	1.0	82*	$C_{Project} < C_{RMi}$
		$C_{Project} = C_{RMi}$	±0.015	8	
		$C_{Project} > C_{RMi}$	0.3	10	
2.	Project and RSR	$C_{Project} < C_{RSR}$	0.3	77	$C_{Project} < C_{RSR}$
		$C_{Project} = C_{RSR}$	±0.015	10	
		$C_{Project} > C_{RSR}$	0.2	13	
3.	Project and Q	$C_{Project} < C_Q$	0.4	59	$C_{Project} < C_Q$
		$C_{Project} = C_Q$	±0.015	12	
		$C_{Project} > C_Q$	0.3	29	
4.	Project and N	$C_{Project} < P_N$	0.23	56	$C_{Project} < P_N$
		$C_{Project} = P_N$	±0.015	11	
		$C_{Project} > P_N$	0.3	33	
5.	Project and RCR	$C_{Project} < P_{RCR}$	0.1	6	$C_{Project} > P_{RCR}$
		$C_{Project} = P_{RCR}$	±0.015	28	
		$C_{Project} > P_{RCR}$	0.4	66	
6.	Project and RMR	$C_{Project} < C_{RMR}$	0.2	1	$C_{Project} > C_{RMR}$
		$C_{Project} = C_{RMR}$	±0.015	24	
		$C_{Project} > C_{RMR}$	0.4	75	

* Including SDSC

Table 7.10 Comparisons of Capacity of Support Recommended by Classification Systems and Installed by the Project Authorities in Squeezing Ground Conditions

S. No.	Classification Systems	Inequality	Number of Sections (in percent)	Conclusion
1.	Project and RMi	$C_{Project} < C_{RMi}$	57	$C_{Project} < C_{RMi}$
		$C_{Project} > C_{RMi}$	43	
2.	Project and N	$C_{Project} < P_N$	69	$C_{Project} < P_N$
		$C_{Project} > P_N$	31	
3.	Project and Q	$C_{Project} < C_Q$	37	$C_{Project} > C_Q$
		$C_{Project} > C_Q$	63	
4.	Project and RCR	$C_{Project} < P_{RCR}$	11	$C_{Project} > P_{RCR}$
		$C_{Project} > P_{RCR}$	89	
5.	Project and RMR	$C_{Project} < C_{RMR}$	0	$C_{Project} > C_{RMR}$
		$C_{Project} > C_{RMR}$	100	

7.2.2.7 Length of tunnel in classes of supports installed by the Project Authorities

Figures 7.31 through 7.36 indicate length of tunnel in classes of supports installed by the Project Authorities for RSR, RMR, Q, RMi, N and RCR values respectively. On comparison of these figures with Figs. 7.11, 7.14, 7.17 a, 7.22, 7.25 and 7.28 respectively, it is observed that they are almost matching each other.

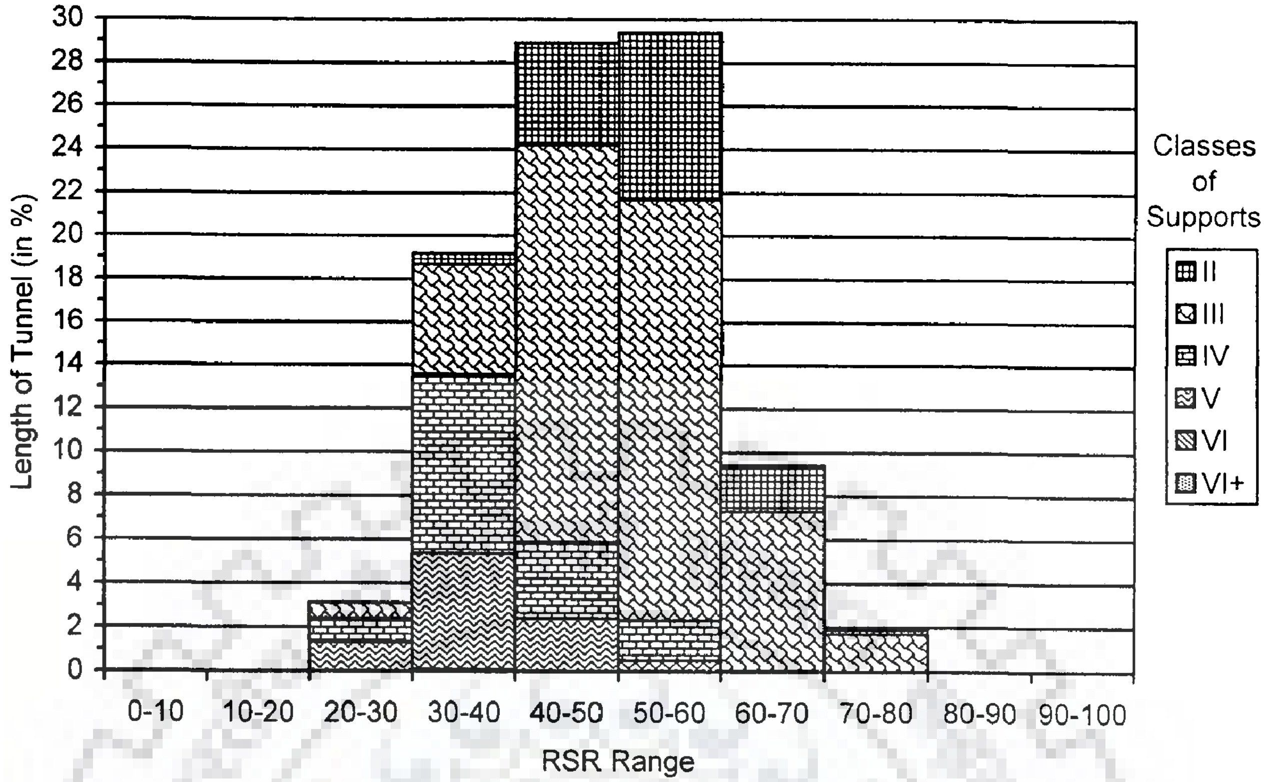


Fig. 7.31 Length of tunnel in classes of supports installed by the Project Authorities for RSR-values

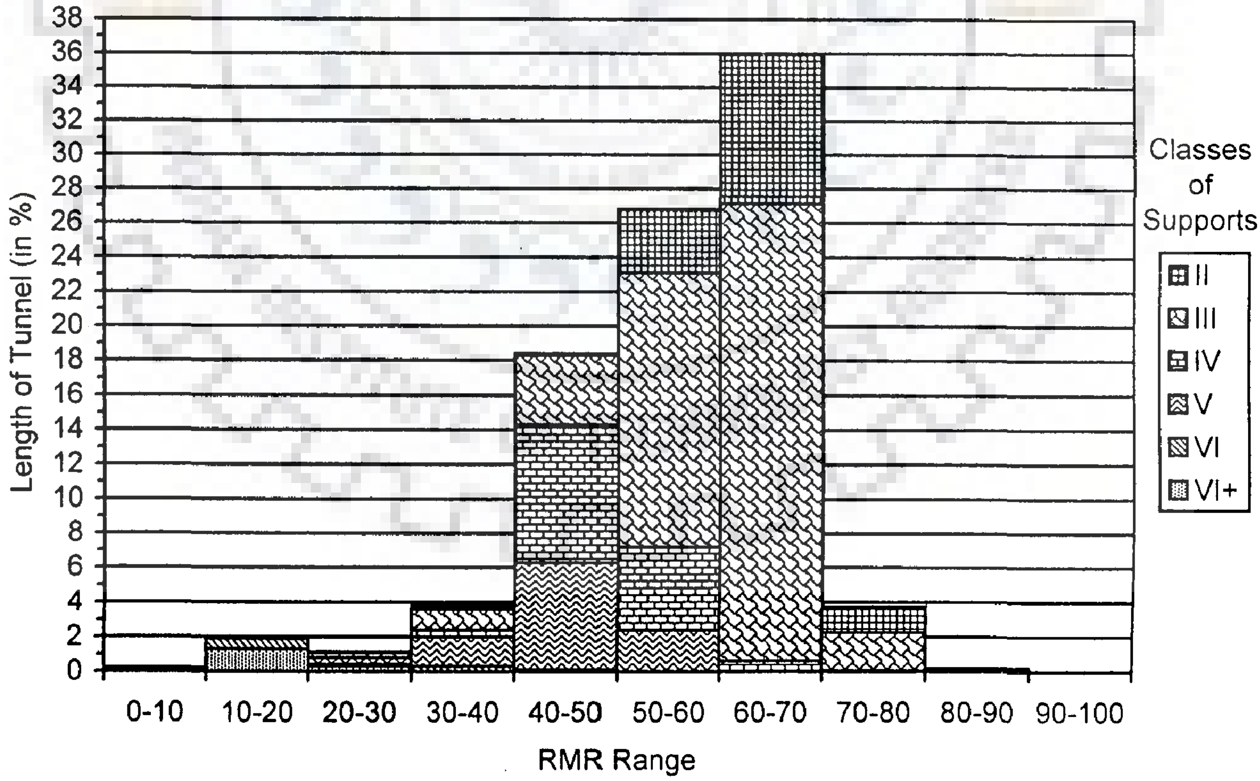


Fig. 7.32 Length of tunnel in classes of supports installed by the Project Authorities for RMR-values

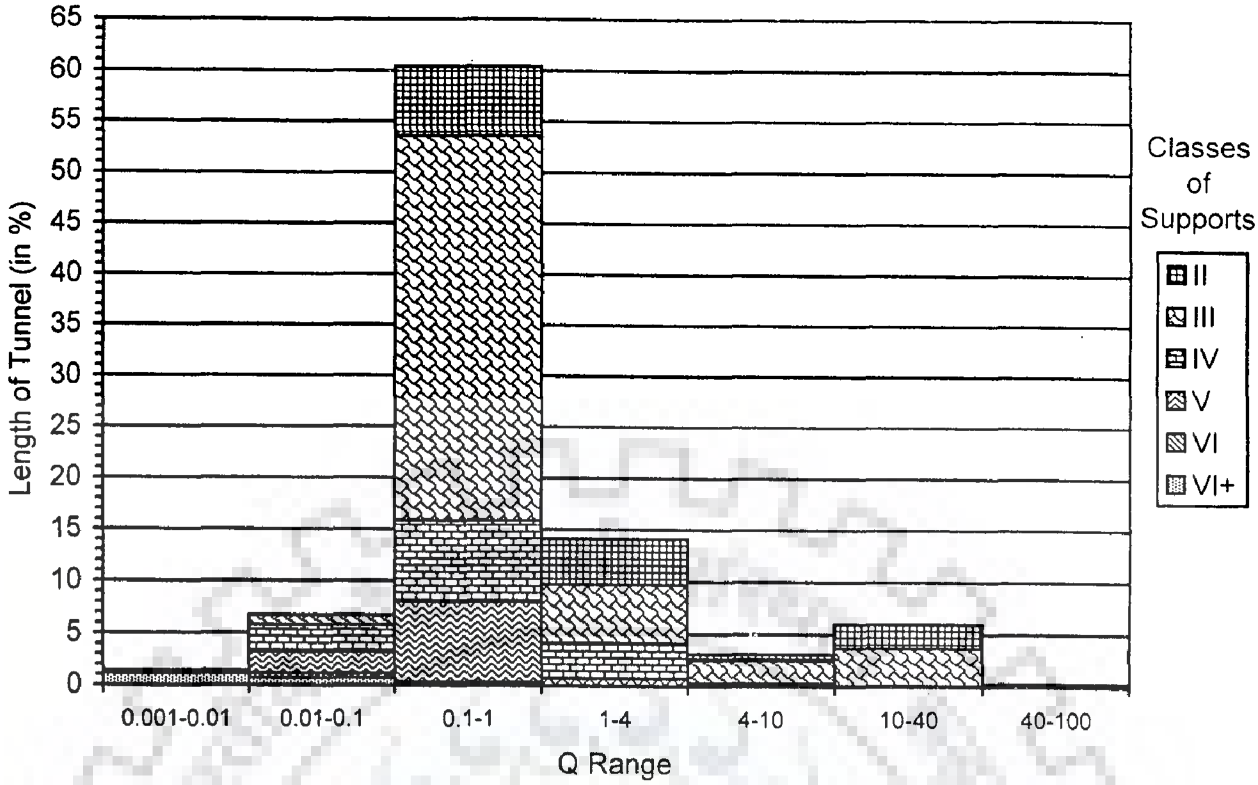


Fig. 7.33 Length of tunnel in classes of supports installed by the Project Authorities for Q-values

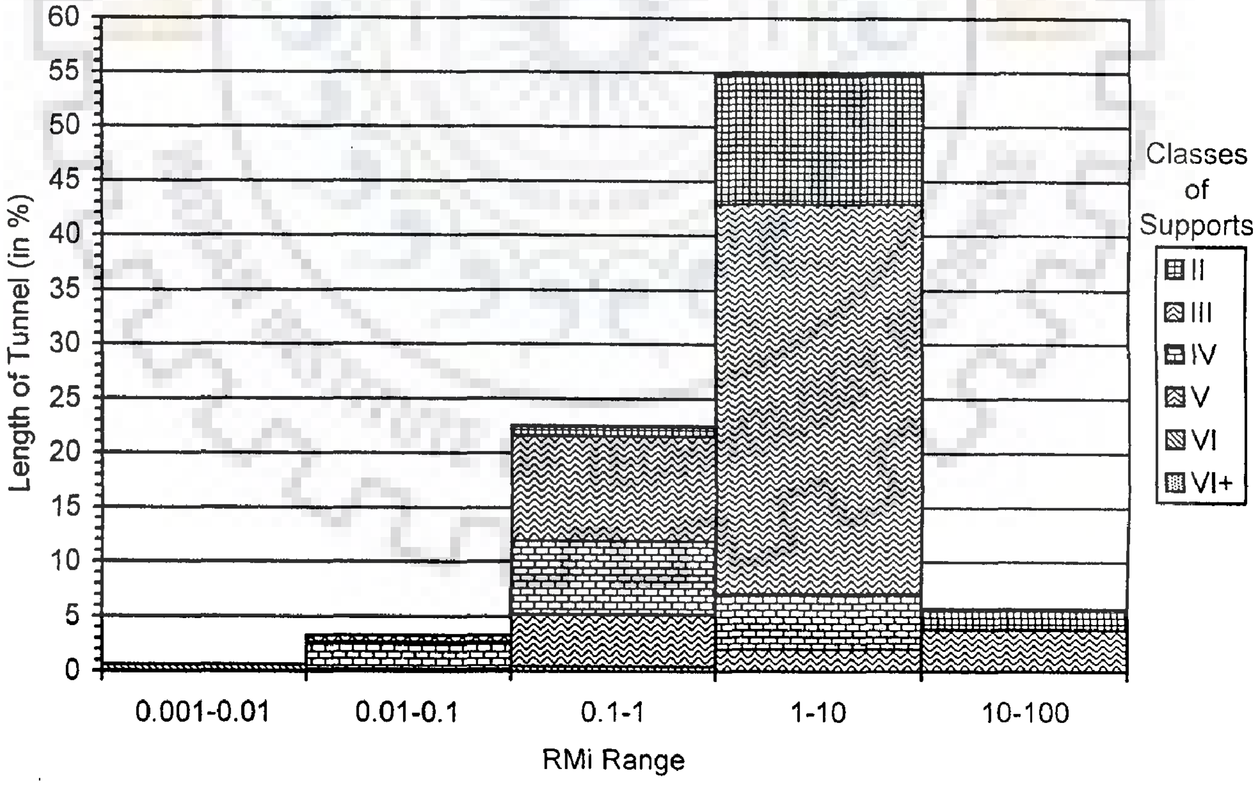


Fig. 7.34 Length of tunnel in classes of supports installed by the Project Authorities for RMi-values

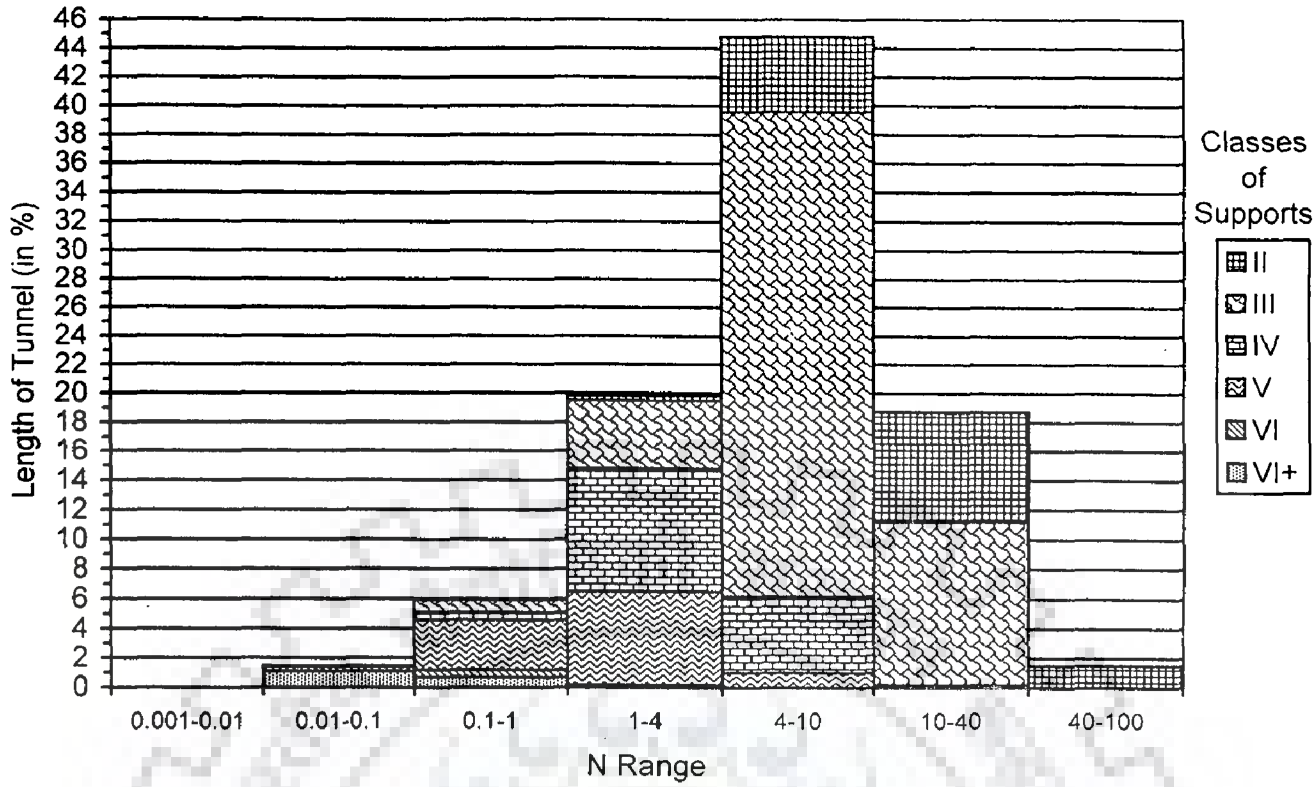


Fig. 7.35 Length of tunnel in classes of supports installed by the Project Authorities for N-values

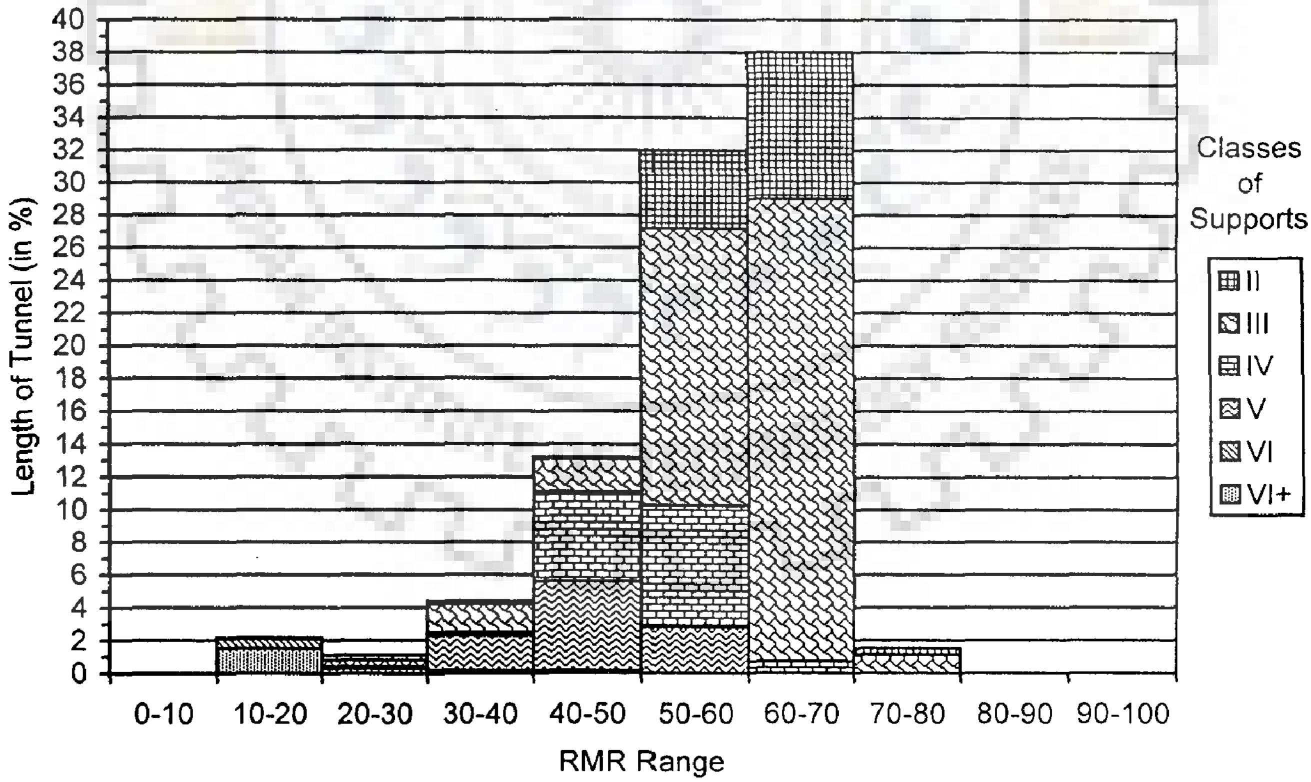


Fig. 7.36 Length of tunnel in classes of supports installed by the Project Authorities for RCR-values

ROCK MASS – SUPPORT INTERACTION ANALYSIS

Stresses and displacements in the rock surrounding tunnels and in the support elements depend, not only on the rock mass properties and the in-situ stress field, but also on the type and stiffness of the support and the timing of its installation. The interdependence of these various factors is commonly represented by ground response curve (GRC) and support reaction curve (SRC) on a ground – support interaction diagram.

Rock mass – support interaction analysis is a well-known method of design of tunnel support system. The technique of employing two characteristic curves: one for the ground and the other for the support has been described in detail, by Lombardi (1970), Daemen and Fairhurst (1970), Ladanyi (1974) and Lombardi (1977). It is one of the most promising methods for understanding the mechanics of tunnel deformation and development of rock loads. The evaluation of characteristics calls for assumptions that do simplify the theoretical solution obtained for the complex practical problem. Sakurai (1993) stressed the importance of back analysis in rock engineering. Parameters obtained through careful monitoring of the tunnel through instrumentation might be utilized in the selected model to predict performance of the support system.

The GRC is the relation between the inward radial displacement at the tunnel periphery and the radial pressure or support pressure applied at the periphery. It is assumed that the sequence of face advance and excavation can be represented by a gradual continuous increase of the tunnel convergence and the GRC is then obtained by calculating the support pressure that would be required to maintain equilibrium at each point of this convergence curve. The support characteristic is the load deflection curve of the support. The intersection between the two characteristic curves refers to the equilibrium point between the ground movement and the support reaction, and thus the load mobilized by the support to maintain the stability of the tunnel at the corresponding convergence. A complete evaluation of the two characteristic curves requires knowledge of the virgin stresses and the strength and deformation characteristics of the rock mass and the support.

8.1 GROUND RESPONSE CURVE (GRC)

An interactive computer program has been developed in C++ language for obtaining the GRC using the formulations of Ladanyi (1974) and Brown et al. (1983). The program has been run for non-squeezing ground conditions only. Output of the program gives deformations, roof and side pressures and corresponding depth of plastic zone if any (Figs. 8.1 a through 8.1 l). The failure characteristics of original and broken rock masses are defined by Eqs. 2.2 and 2.3 respectively.

According to Daemen (1975), GRC is quite useful for designing the supports specially for tunnels through squeezing ground conditions. The method given by Goel et al. (1998) has been used for obtaining the GRC for squeezing ground conditions. Goel et al. (1998) have developed an easy to use empirical approach for obtaining GRC using Eqs. 6.15 and 6.30 involving RCR and N respectively in squeezing ground conditions. In Eqs. 6.15 and 6.30, f(RCR) and f(N) are the correction factors respectively for tunnel closure. For different values of permitted normalized tunnel closure (u_a/a), different values of f(RCR) and f(N) are proposed in Table 6.1. From Table 6.1 and Eqs. 6.15 and/or 6.30, the support pressures are estimated for various values of u_a/a. Subsequently using values of p_v and u_a/a, GRC is plotted.

GRCs have been plotted using both N and RCR values. Purpose of plotting GRCs with both N and RCR is that a comparison can be made between the two. These curves have been indicated in Figs. 8.2, 8.3 and 8.4 for mild, moderate and high squeezing ground conditions respectively.

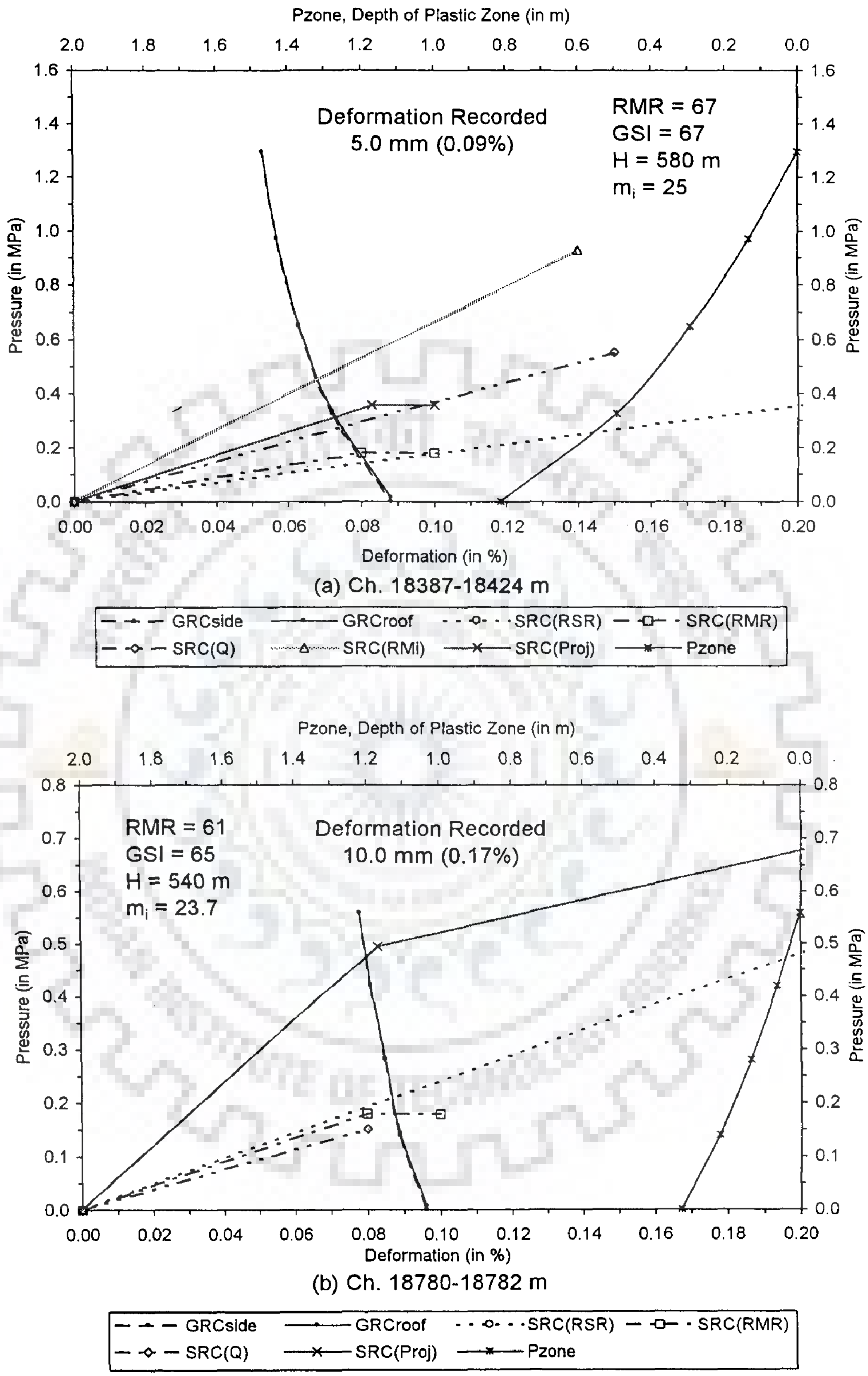
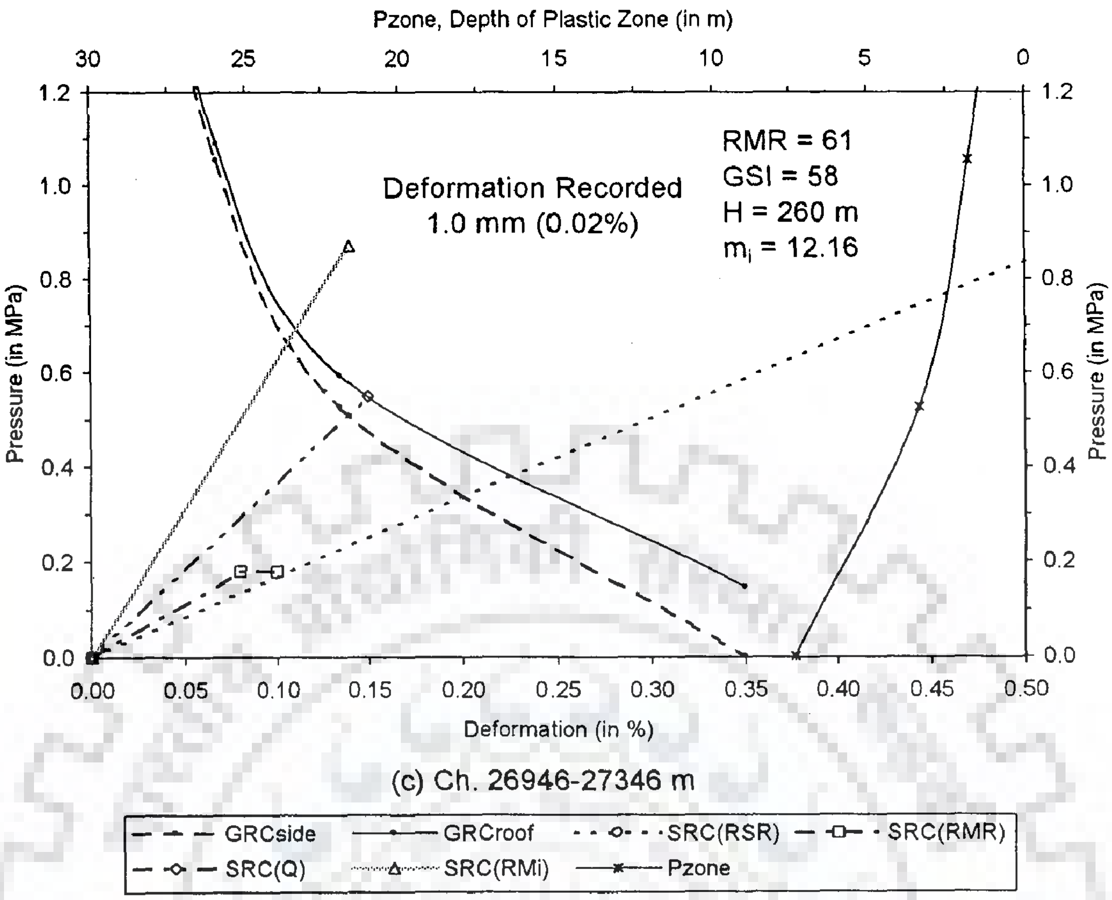
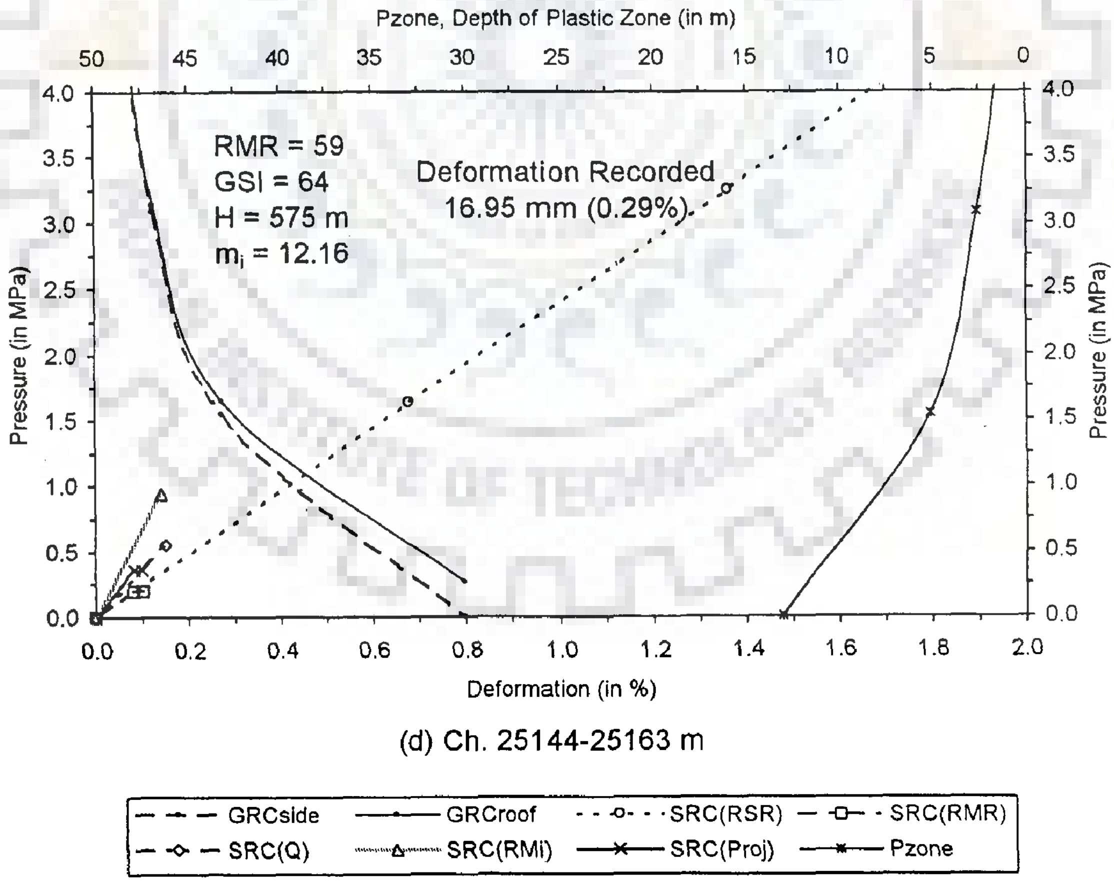


Fig. 8.1 Convergence-confinement curves for non-squeezing ground conditions

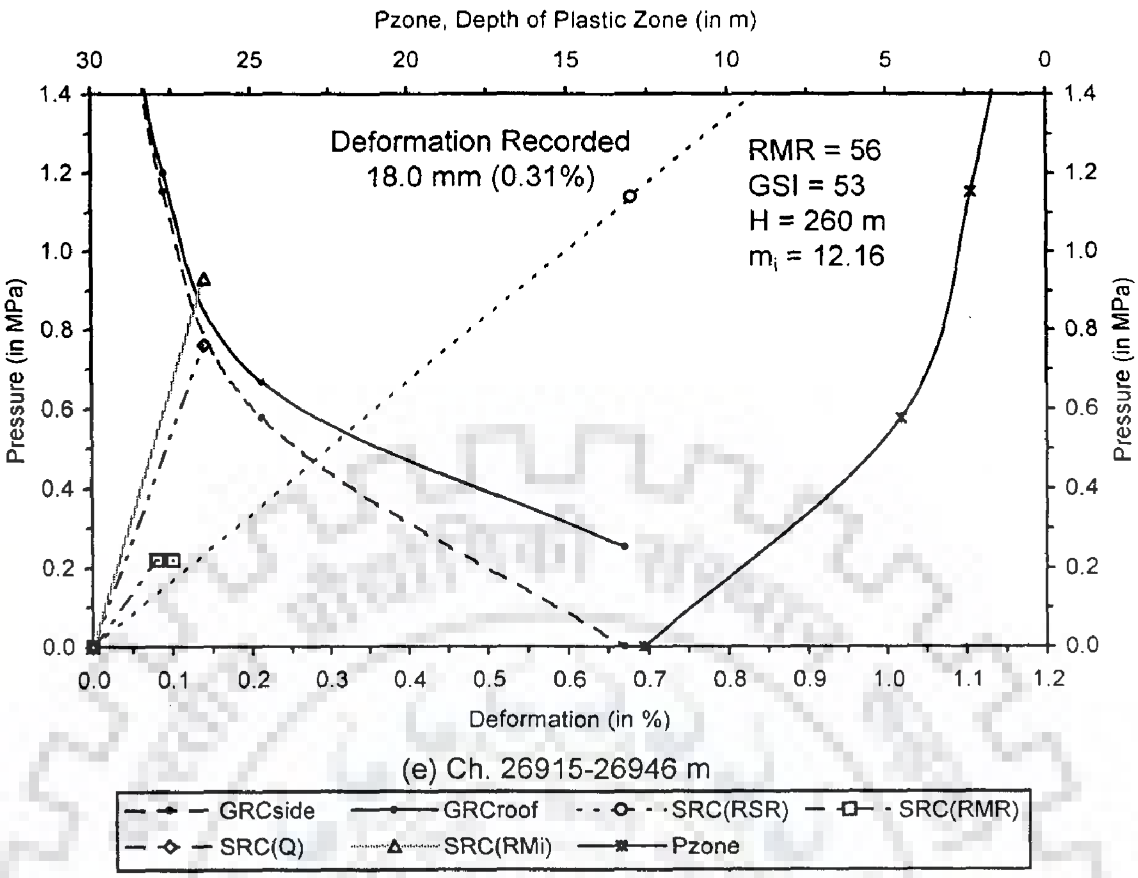


(c) Ch. 26946-27346 m

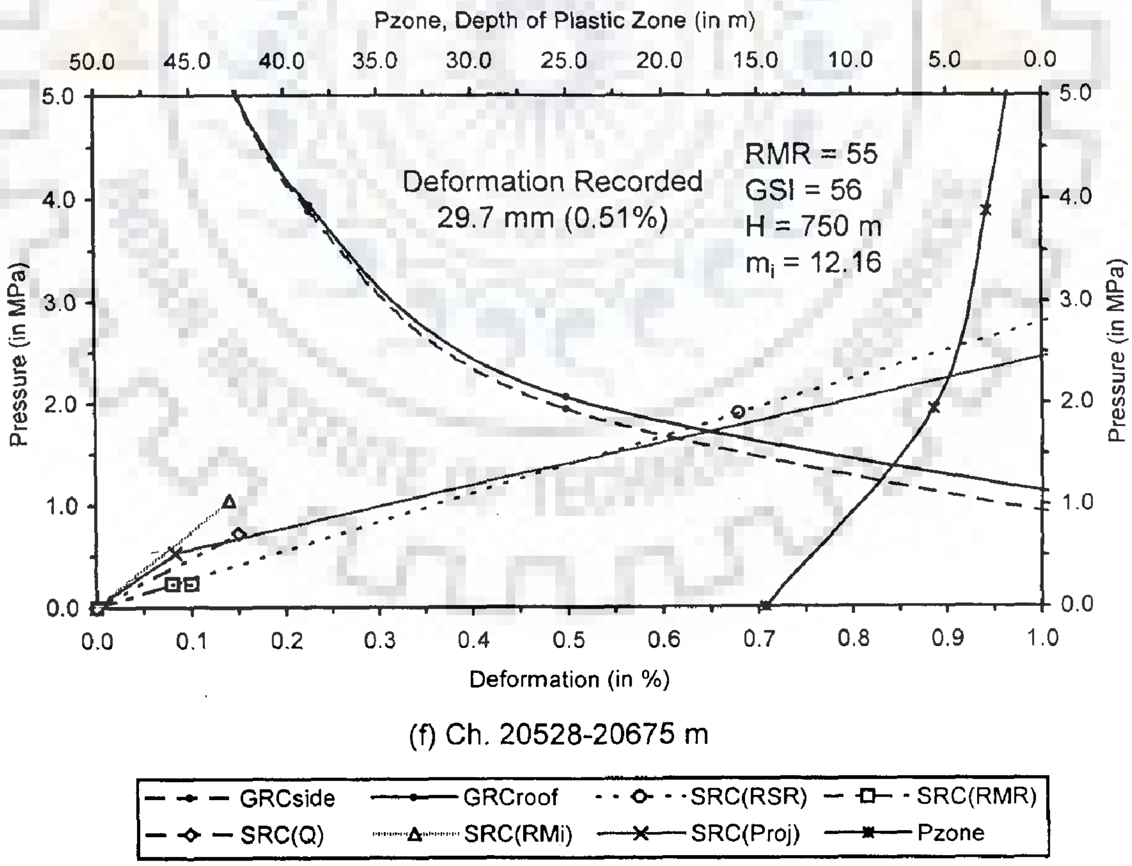


(d) Ch. 25144-25163 m

Fig. 8.1 Convergence-confinement curves for non-squeezing ground conditions

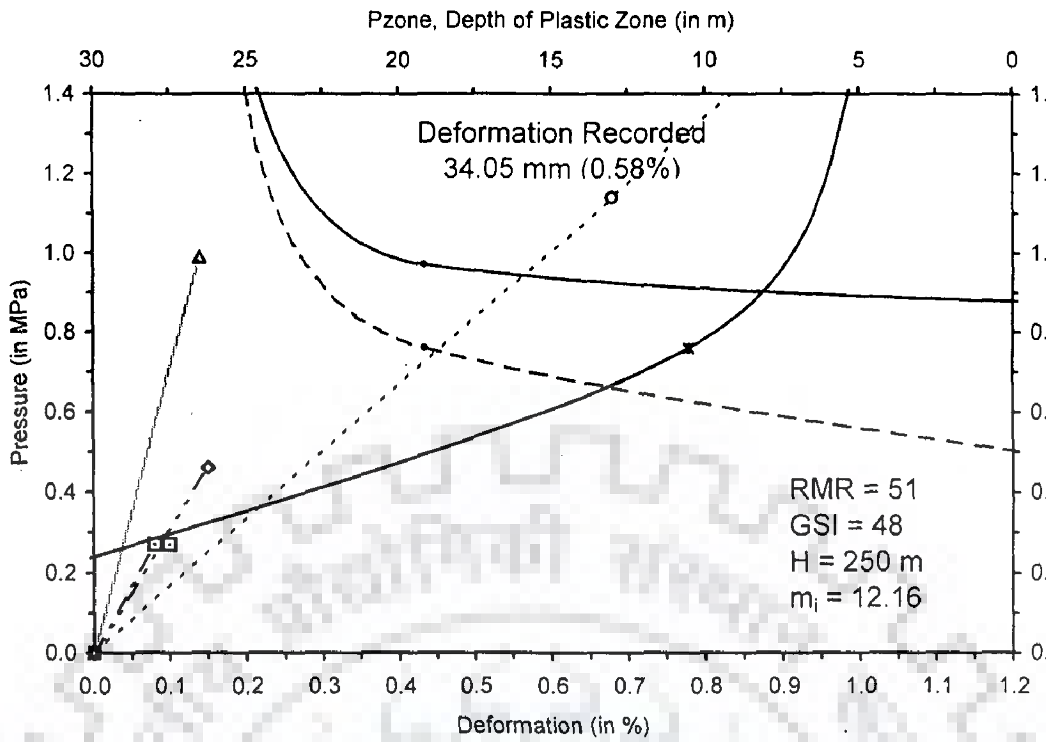


(e) Ch. 26915-26946 m

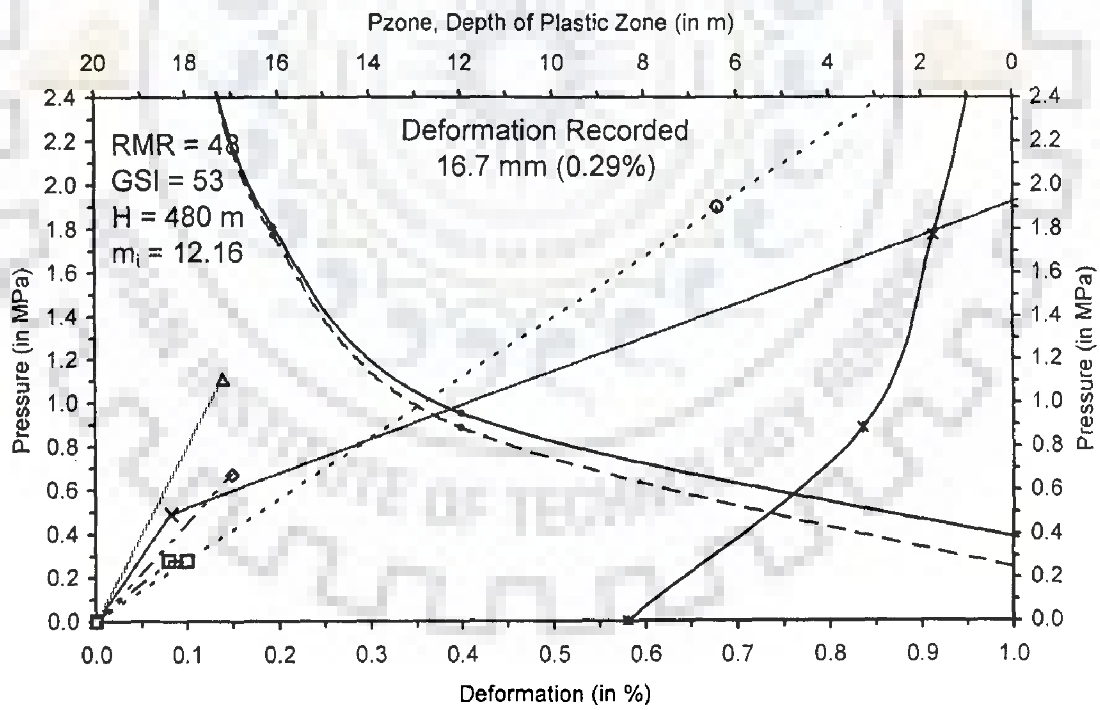


(f) Ch. 20528-20675 m

Fig. 8.1 Convergence-confinement curves for non-squeezing ground conditions

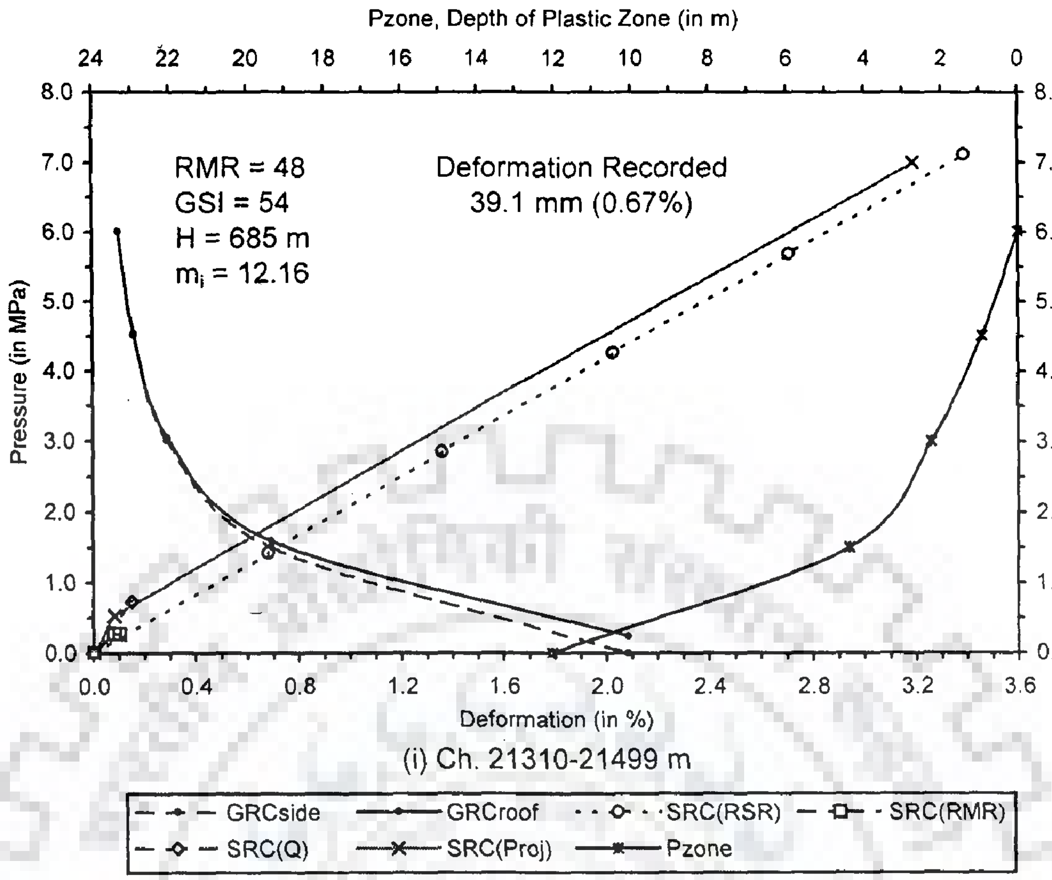


(g) Ch.26802-26915 m

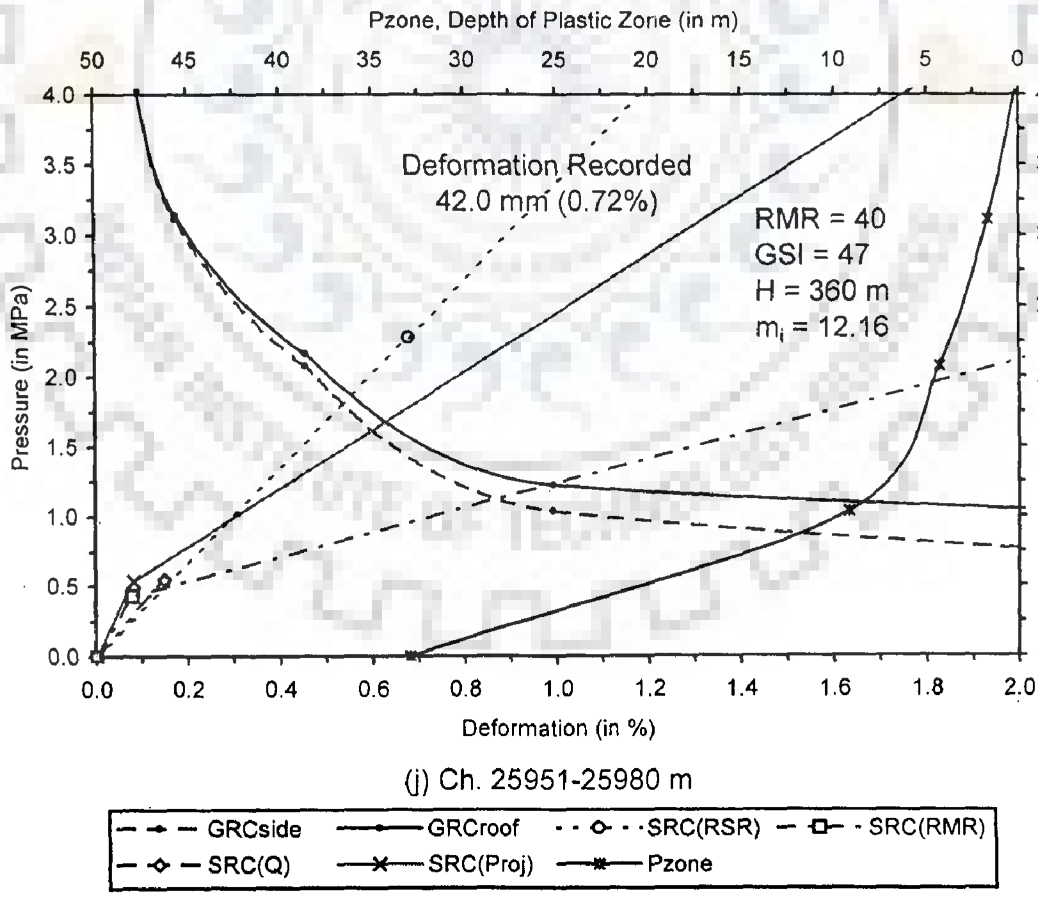


(h) Ch. 23627-23785 m

Fig. 8.1 Convergence-confinement curves for non-squeezing ground conditions



(i) Ch. 21310-21499 m



(j) Ch. 25951-25980 m

Fig. 8.1 Convergence-confinement curves for non-squeezing ground conditions

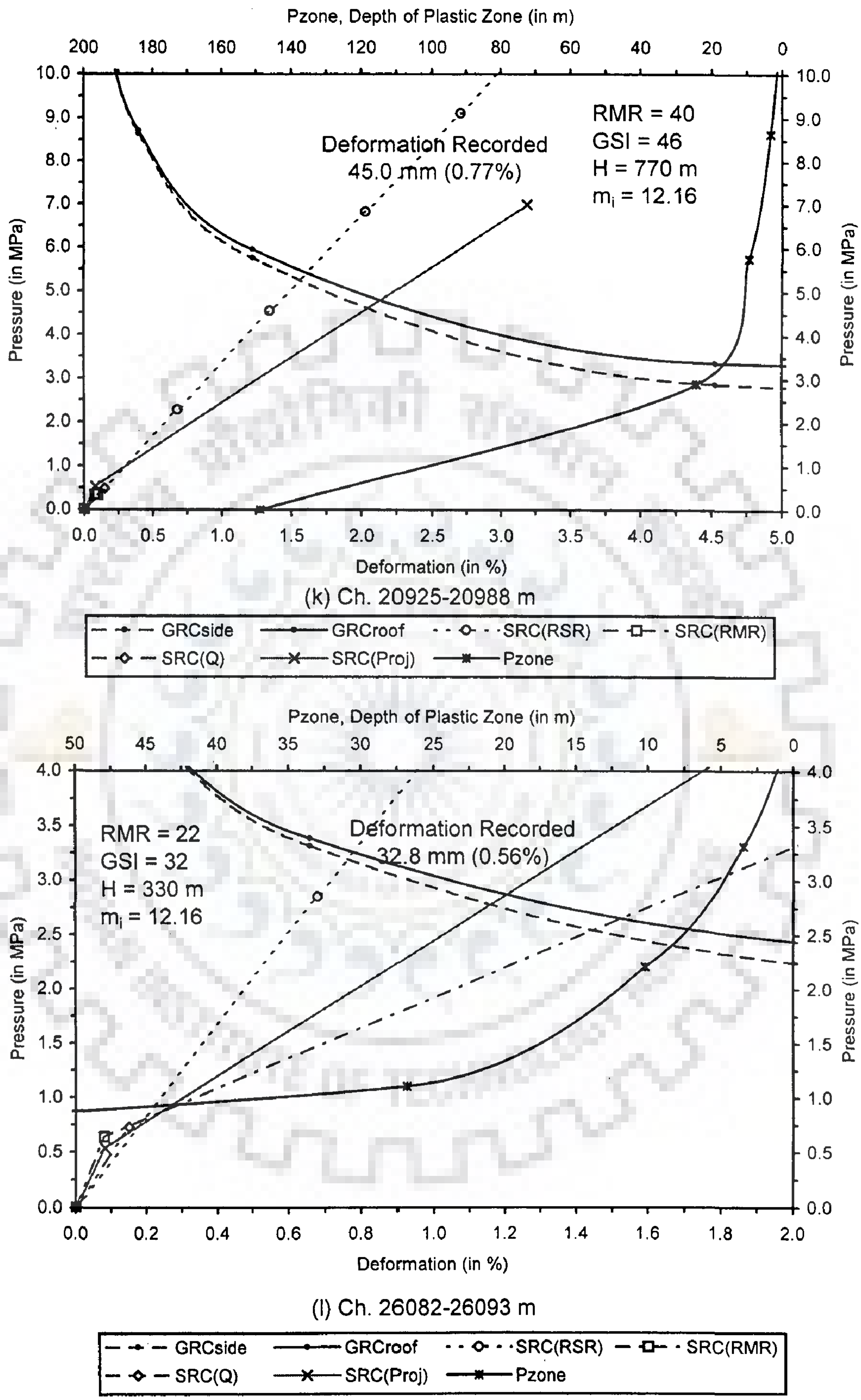
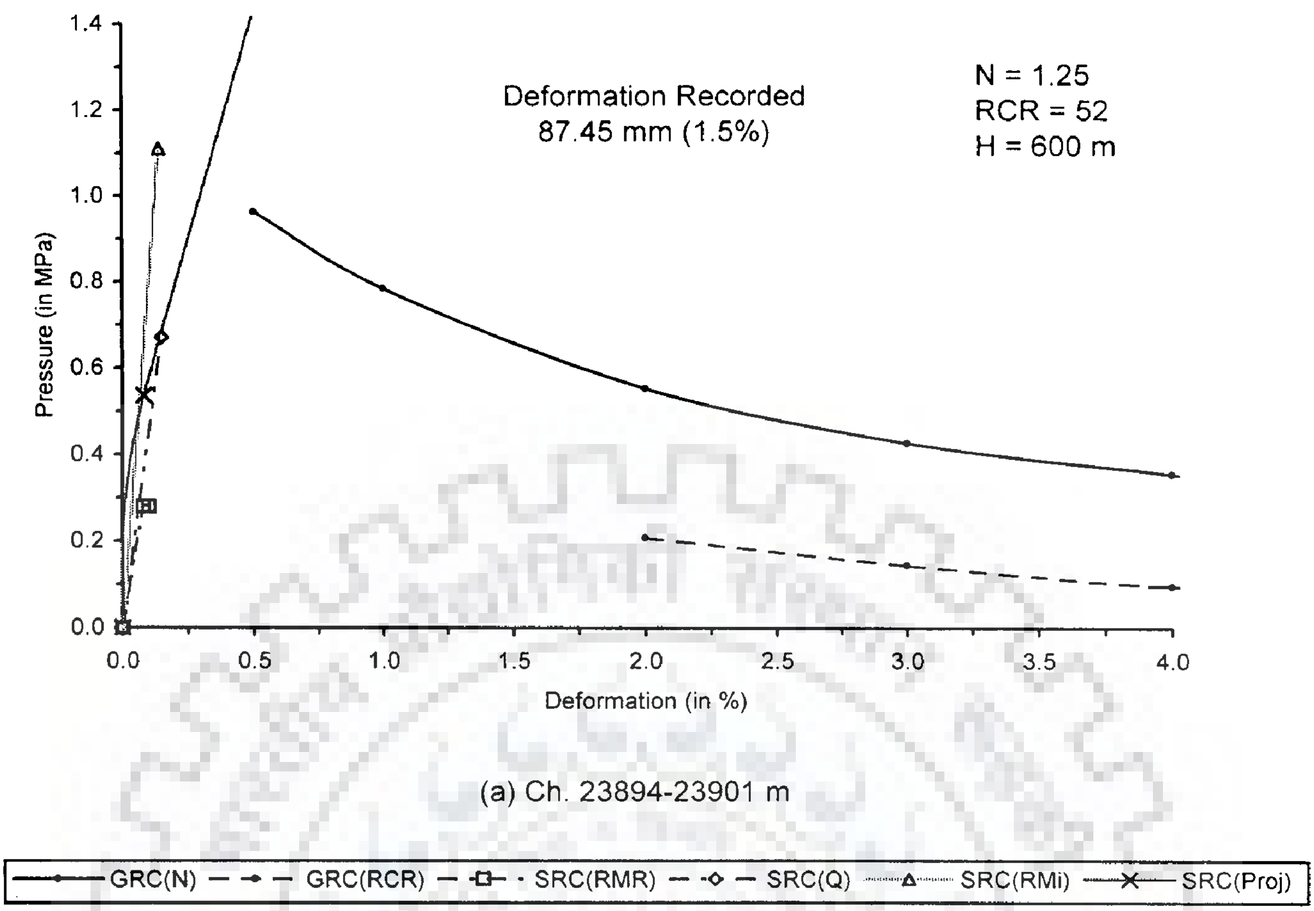
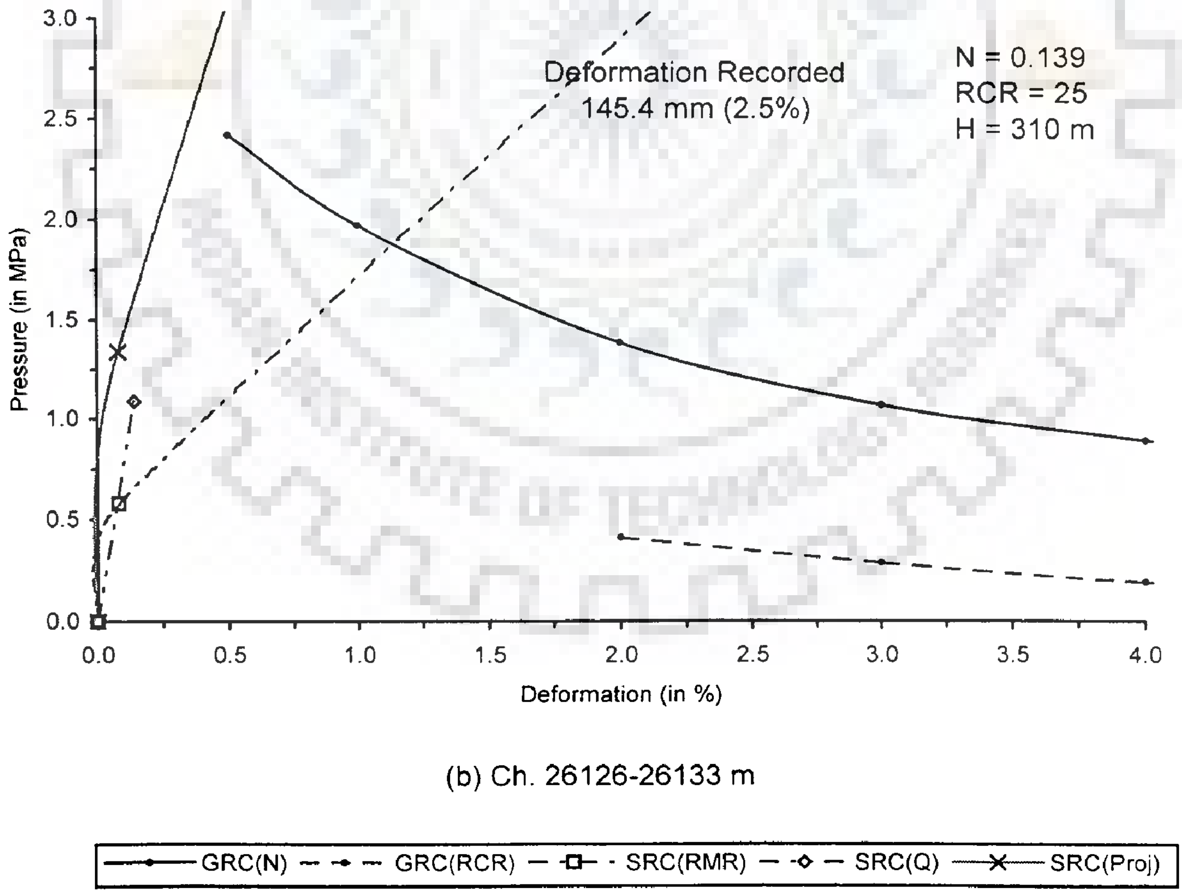


Fig. 8.1 Convergence-confinement curves for non-squeezing ground conditions

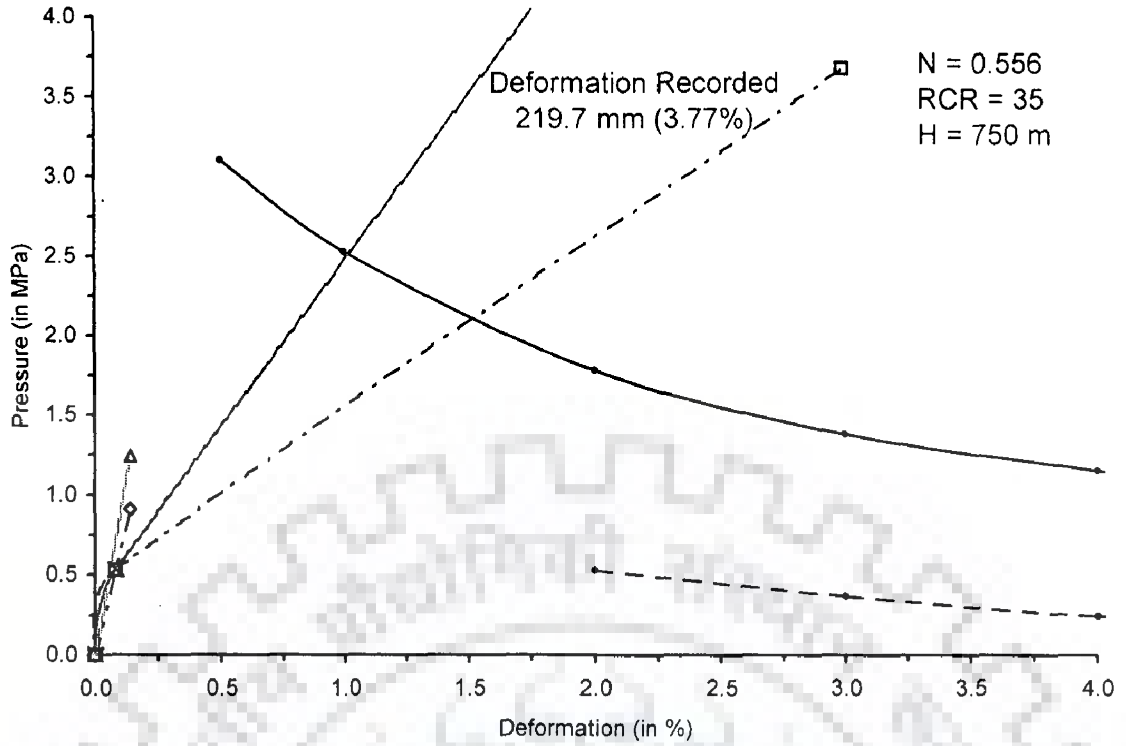


(a) Ch. 23894-23901 m

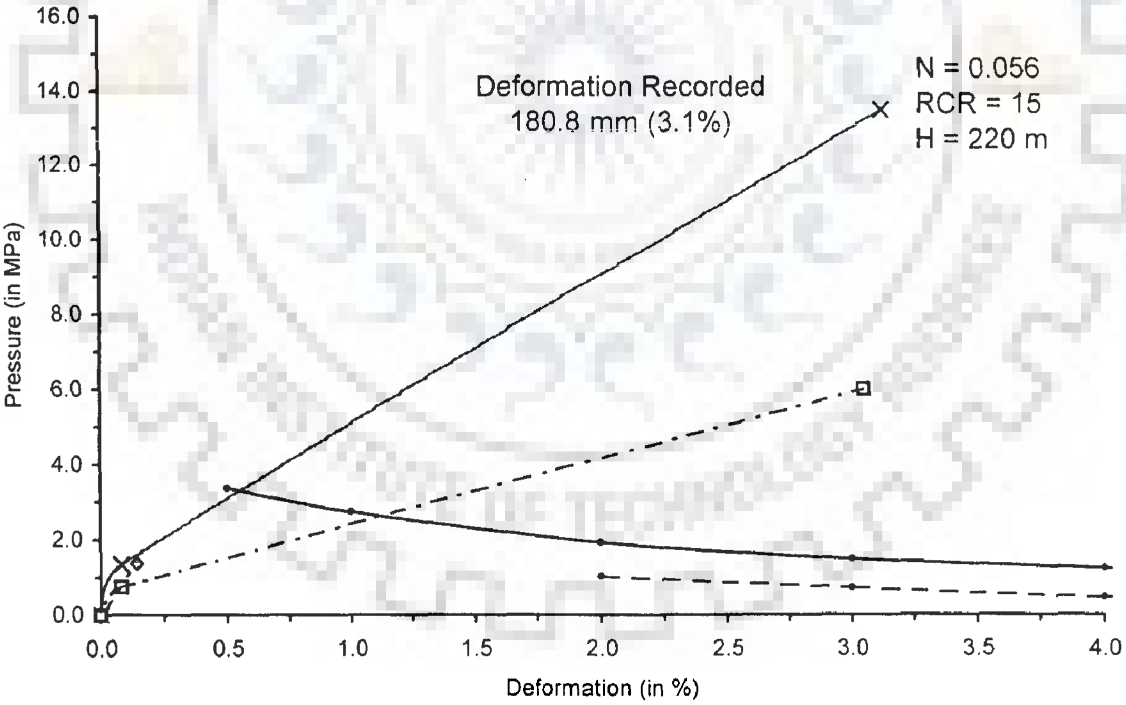


(b) Ch. 26126-26133 m

Fig. 8.2 Convergence-confinement curves for mild squeezing ground conditions

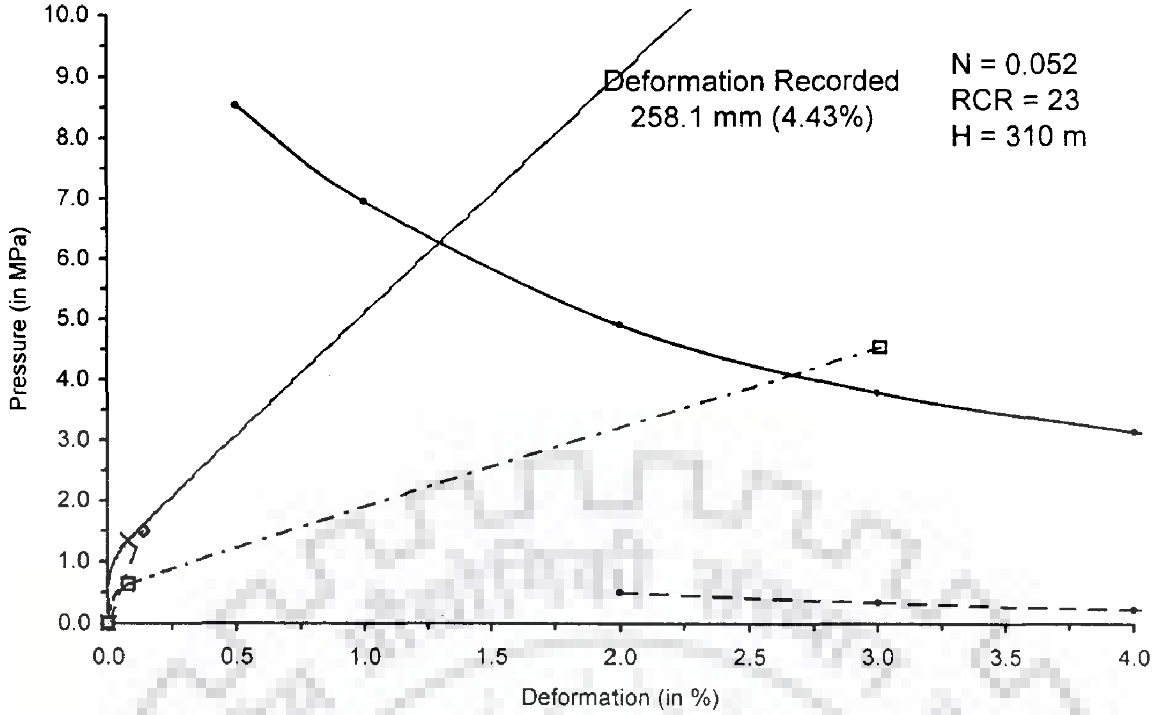


(a) Ch. 21230-21240 m

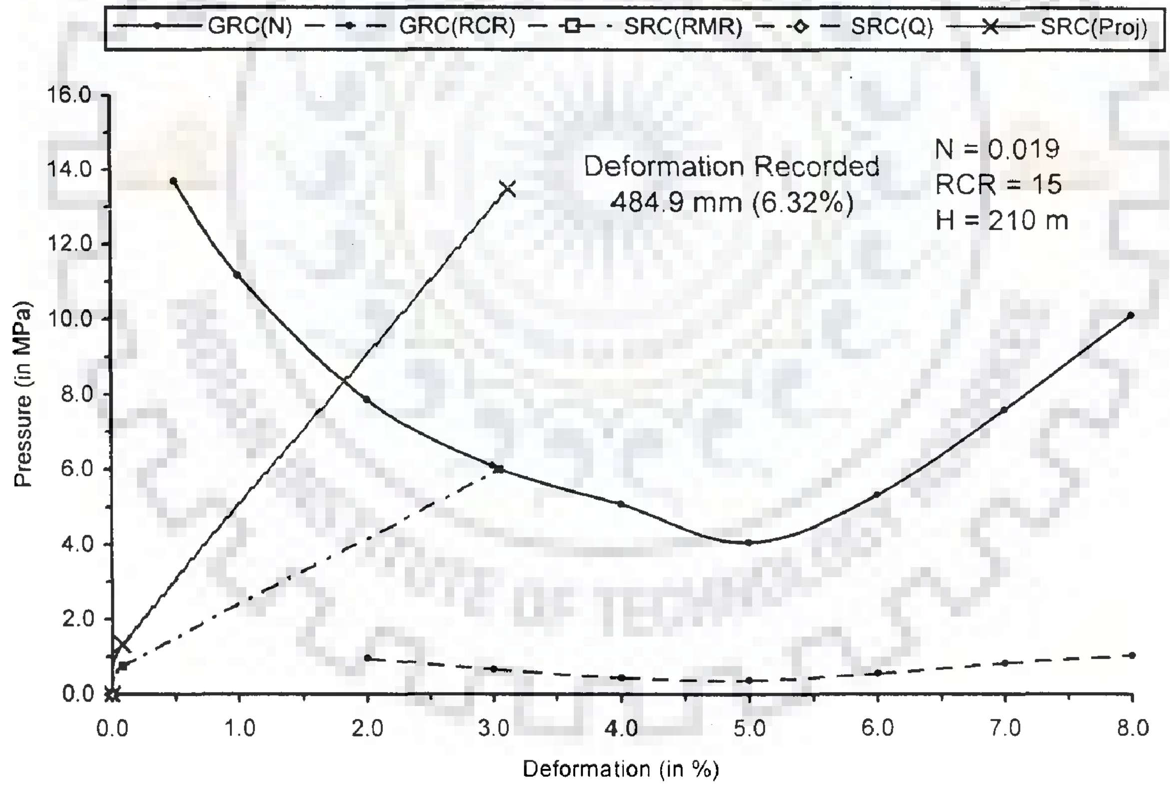


(b) Ch. 26620-26623 m

Fig. 8.3 Convergence-confinement curves for moderate squeezing ground conditions



(a) Ch. 26120-26126 m



(b) Ch. 26608-26620 m

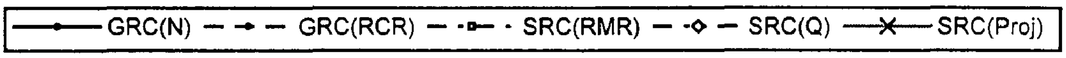


Fig. 8.4 Convergence-confinement curves for high squeezing ground conditions

8.2 SUPPORT REACTION CURVE (SRC)

Table 6.3 presents the support recommendations by RSR, RMR, Q and RMi systems. An attempt has been made to verify the efficacy of these support systems and also those actually installed at the site by the Project Authorities, by plotting their SRCs in the same graphs, which have been plotted for GRCs as indicated in Figs. 8.1, 8.2, 8.3 and 8.4. Two separate interactive computer programs have been developed in C++ language for determination of SRC: one with shotcrete and rock bolt supports and the other with steel rib and concrete supports. Stiffness of supports and maximum support pressure has been computed as per the formulations given in APPENDIX - IX. The values of input parameters used in these programs have been given in Table 8.1.

Table 8.1 Input Parameters for Support Reaction Curve

Parameter	Value	Parameter	Value
α	1^0	E_s	207 GPa
t_b	250 mm	$q_{c.conc}$ for concrete	20 MPa
d_b	25 mm	$q_{c.conc}$ for SFRS	35 MPa
R	0.0001 mm/N	E_c for shotcrete	22 GPa
T_{bf}	200 kN	E_c for SFRS	22 GPa
σ_{ys}	245 MPa	ν_c	0.25

8.3 MATERIAL PROPERTIES

Material properties for non-squeezing ground conditions have been compiled in Table 8.2. The classification values pertaining to N and RCR and the depth of overburden for squeezing

Table 8.2 Material Properties used in Interaction Analysis for Non-Squeezing Ground Conditions

S. No.	Chainage (in m)	Rock Type	Depth (in m)	UCS (MPa)	m_i	$\boxed{\alpha}$	RMR	GSI	Original Rock Mass Constants		E_d (in GPa)	E_e (in GPa)	$\boxed{\upsilon}$	Broken Rock Mass Constants	
									m	s				m_r	s_r
1	18387-18424	amphibolite	580	70	25.00	0.20	67	67	7.7	0.026	18.48	35	0.29	2.37	0.004
2	18780-18782	quartzite	540	125	23.70	0.20	61	65	6.8	0.020	12.66	24	0.33	1.95	0.003
3	26946-27346	QMS	260	16.5	12.16	0.20	61	58	2.7	0.009	10.94	21	0.22	0.61	0.001
4	25144-25163	QMS	575	16.5	12.16	0.24	59	64	3.4	0.018	14.65	28	0.22	0.93	0.002
5	26915-26946	QMS	260	16.5	12.16	0.24	56	53	2.3	0.005	10.09	19	0.22	0.42	0.000
6	20528-20675	QMS	750	31.7	12.16	0.24	55	56	2.5	0.008	12.25	23	0.22	0.52	0.001
7	26802-26915	QMS	250	9.5	12.16	0.24	51	48	1.9	0.003	7.39	14	0.22	0.30	0.000
8	23627-23785	QMS	480	42.4	12.16	0.24	48	53	2.3	0.005	7.20	14	0.22	0.42	0.000
9	21310-21499	QMS	685	42.4	12.16	0.24	48	54	2.4	0.006	7.84	15	0.22	0.45	0.000
10	25951-25980	QMS	360	16.5	12.16	0.24	40	47	1.8	0.003	4.14	8	0.22	0.28	0.000
11	20925-20988	quartz biotite schist	770	16.5	12.16	0.24	40	46	1.8	0.002	4.97	9	0.22	0.26	0.000
12	26082-26093	QMS	330	16.5	12.16	0.26	22	32	1.1	0.001	1.53	3	0.22	0.09	0.000

ground conditions have been compiled in Table 8.3. Tables 8.2 and 8.3 include 12 and 6 tunnel sections respectively, which have been chosen in such a way that in these sections tunnel closure data were available and it covered variety of rock masses, depth of overburden and degree of squeezing. (Since tunnel closures have been measured in horizontal direction only, the discussion in the subsequent sections pertains to walls or sides only). A brief discussion of their source is as follows:

Table 8.3 N and RCR Values used in Interaction Analysis for Squeezing Ground Conditions

S.No.	Chainage (in m)	Depth (in m)	N	RCR	Squeezing Condition
1.	23894-23901	600	1.25	52	Mild
2.	26126-26133	310	0.139	25	Mild
3.	21230-21240	750	0.556	35	Moderate
4.	26620-26623	220	0.056	15	Moderate
5.	26120-26126	310	0.052	23	High
6.	26608-26620	210	0.019	15	High

Rock type, depth of overburden and UCS have been taken from Table 5.19.

Values of m_i recommended by CSMRS for gneisses and amphibolite are 7.55 and 13.75 respectively and for quartzite they have not recommended any value (Table 5.8). Since, it is evident that these values are on a lower side, it has been decided in the present work that m_i for gneisses will be taken as 29.2 (Table 2 of Hoek, 1992). The m_i for amphibolite and quartzite are

25.0 and 24.0 respectively, taken from Table 2 of Hoek and Brown (1997). Value of m_i for quartz mica schist has been taken as 12.16 as per CSMRS test result (Table 5.8).

The values of RMR and basic RMR for dry rock mass have been taken from Table 5.21 and GSI has been computed as follows:

$$GSI = \text{basic RMR for dry rock mass} - 5 \quad (8.1)$$

Original rock mass constants m and s have been computed from Eqs. 2.23 and 2.24 respectively, whereas broken rock mass constants m_r and s_r from Eqs. 2.25 and 2.26 respectively.

Modulus of elasticity, E has been taken as 1.9 times E_d on the basis of results of field tests carried out by Mehrotra (1992). E_d has been estimated from Eq. 8.2 proposed by Verman (1993).

$$E_d = 0.3\, H^{\alpha}.10^{\left(\frac{RMR-20}{38}\right)} \quad \text{GPa} \quad (8.2)$$

where, $\alpha =$ 0.16 to 0.30 (higher for poor rocks) and $H \geq 50$ m.

Values of α assumed have been based on RMR-values as indicated in Table 8.4.

Table 8.4 Assumed Values of α in Eq. 8.2

Range of RMR	Value of α
100 - 80	0.16 - 0.19
80 - 60	0.19 - 0.22
60 - 40	0.22 - 0.24
40 - 20	0.24 - 0.27
< 20	0.27 - 0.30

Poisson's ratio, ν have been taken from Table 5.7 as per CSMRS test results.

Unit weight of broken rock mass, γ_r has been assumed as 20 kN/m^3.

8.4 ANALYSIS FOR NON-SQUEEZING GROUND CONDITIONS

8.4.1 Good Rock (following RMR-system)

(A) Ch. 18387-18424 m (RMR = 67, GSI = 67, H = 580 m, m_i = 25) (Fig. 8.1 a)

(B) Ch. 18780-18782 m (RMR = 61, GSI = 65, H = 540 m, m_i = 23.7) (Fig. 8.1 b)

In Figs. 8.1 a and 8.1 b, the maximum deformations for no support are 0.09 and 0.095 percent respectively and for which depths of plastic zones are 0.8 m and 0.35 m only. Considering these values of deformation and depth of plastic zone, which are very low, the section might have been left unsupported through the interaction analysis. Figure 8.1 a indicates that RSR recommended heaviest support followed by RMi, Q and RMR. Similarly in Fig. 8.1 b, RSR recommended heaviest support whereas RMR and Q recommended almost identical supports. In Figs. 8.1 a and 8.1 b, the SRCs for supports installed by the Project Authorities intersect the GRCs at 0.07 and 0.08 percent deformations respectively, but the deformations recorded in the horizontal direction are 0.09 and 0.17 percent respectively. Reason for this difference may be attributed to the fact that stiffness of shotcrete in the walls of tunnel is less compared to the one assumed in the analysis in which closed ring of support is considered to be installed in the entire periphery of the tunnel.

(C) Ch. 26946–27346 m (RMR = 61, GSI = 58, H = 260 m, m_i = 12.16) (Fig. 8.1 c)

The maximum deformation with no support is 0.35 percent for which depth of plastic zone is 7.5 m. Some amount of support is required here to arrest the deformation below 0.35 percent. RSR recommended heaviest support followed by RMi, Q and RMR. From Fig. 8.1 c, it is clear that only RMi and Q are able to arrest the deformation, but at a reasonably higher pressure values. Therefore, it is recommended that installation of supports might be delayed to achieve overall better support conditions. Actual deformation recorded is very low (0.02 percent) in this section which indicates that rock mass might actually be far better than what has been considered in the analysis but this could not be substantiated because of lack of information about support installed by the Project Authorities in this reach.

8.4.2 Fair Rock (following RMR-system)

(D) Ch. 25144–25163 m (RMR = 59, GSI = 64, H = 575 m, m_i = 12.16) (Fig. 8.1 d)

The maximum deformation with no support is 0.8 percent, which is more than double of section (C) as depth of overburden is also more than double of that of section (C) beside other parameters remaining almost same. RSR recommended heaviest support whereas RMi, Q and RMR prove inadequate to reduce the deformation below 0.8 percent unless supports are installed after substantial initial deformation has taken place. Since deformation recorded is 0.29 percent only and support installed by the Project Authorities is also inadequate, only conclusion drawn from it is that rock mass properties might be much better than envisaged in the analysis.

(E) Ch. 26915–26946 m (RMR = 56, GSI = 53, H = 260 m, m_i = 12.16) (Fig. 8.1 e)

Properties of rock mass in this section are not much different from those in section (C) but it is noteworthy that just by reduction of RMR from 61 to 56 and consequently of m, s, m_r and s_r by slightest of margin, maximum deformation for no support becomes more than two times that of section (C). Therefore, it shows the importance of careful estimation of rock mass constants, otherwise interaction analysis might yield erroneous results.

RSR in this section recommended heaviest support followed by RMi, Q and RMR. Supports recommended by RMi and Q stabilize the tunnel whereas RMR prove inadequate. Horizontal deformation recorded in this section is 0.31 percent but information regarding the details of supports installed by the Project Authorities could not be ascertained.

(F) Ch. 20528–20675 m (RMR = 55, GSI = 56, H = 750 m, m_i = 12.16) (Fig. 8.1 f)

Properties of rock mass in this section are almost same as in section (E) with only difference being in the depth of overburden, which is 750 m in this section whereas it is 260 m in section (E). The effect of higher overburden is clearly reflected in the GRC of Fig. 8.1 f when compared with Fig. 8.1 e. In-situ stresses are high and the maximum deformation for no support also goes higher. RSR recommended heaviest support and stabilizes tunnel at 0.6 percent deformation whereas those recommended by RMi, Q and RMR prove inadequate. Support installed by the Project Authorities stabilizes tunnel at 0.65 percent deformation whereas the recorded deformation is 0.51 percent.

(G) Ch. 26802–26915 m (RMR = 51, GSI = 48, H = 250 m, m_i = 12.16) (Fig. 8.1 g)

RMR-value in this section is 51 and in section (E) 56, whereas depth of overburden and m_i are almost same yet small reduction in RMR-value from 56 to 51 results in substantial reduction of rock mass constants to deteriorate the GRC. RSR stabilizes tunnel walls at 0.45 percent deformation, whereas Q and RMR prove inadequate, and RMi might just be sufficient if initial deformation is allowed. Horizontal deformation recorded in this section is 0.58 percent. The details of support installed by the Project Authorities could not be ascertained. It may be concluded that supports recommended by RMi and Q would prove to be effective only when they are installed after some initial deformation has taken place. RMR however prove to be inadequate whereas RSR over-estimates support requirements.

(H) Ch. 23627–23785 m (RMR = 48, GSI = 53, H = 480m, m_i = 12.16) (Fig. 8.1 h)

(I) Ch. 21310–21499 m (RMR = 48, GSI = 54, H = 685 m, m_i = 12.16) (Fig. 8.1 i)

In these two sections properties of rock masses are same, and only difference is in the depth of overburden. The maximum deformation in Fig. 8.1 h is approximately 1.3 percent where depth is 480 m whereas in Fig. 8.1 i it is more than 2.0 percent where depth is 685 m. Therefore, it may be appropriate to state that higher the depth of overburden, more will be the maximum deformation for no support situation. In both these sections, RSR and the Project Authorities have provided almost similar supports. In Fig. 8.1 h, support installed by the Project Authorities stabilizes tunnel at 0.35 percent deformation whereas recorded deformation is 0.29 percent. On the other hand in Fig. 8.1 i, support installed by the Project Authorities stabilizes tunnel at 0.6 percent whereas recorded deformation is 0.67 percent.

Supports as per RMi and Q are effective only when they are installed after certain amount of initial deformation is allowed to take place.

8.4.3 Poor Rock (following RMR-system)

(J) Ch. 25951–25980 m (RMR = 40, GSI = 47, H = 360 m, m_i = 12.16) (Fig. 8.1 j)

(K) Ch. 20925–20988 m (RMR = 40, GSI = 46, H = 770 m, m_i = 12.16) (Fig. 8.1 k)

In these two sections properties of rock masses are same, and only difference is in the depth of overburden, which is clearly indicated by the two GRCs in Figs. 8.1 j and 8.1 k.

In Fig. 8.1 j, RSR and RMR stabilize the tunnel at 0.55 and 0.85 percent deformation respectively whereas Q is inadequate. The support installed by the Project Authorities stabilizes the tunnel at 0.6 percent deformation whereas the recorded deformation is 0.72 percent, which suggests that results of interaction analysis in section (J) are acceptable.

In section (K), only RSR stabilizes the tunnel at 1.5 percent deformation whereas Q is inadequate in this section. On the other hand support installed by the Project Authorities stabilizes the tunnel at 2.0 percent deformation. But actual deformation recorded is 0.77 percent only which indicates that rock mass properties might be better than considered in the analysis.

(L) Ch. 26082–26093 m (RMR = 22, GSI = 32, H = 330 m, m_i = 12.16) (Fig. 8.1 l)

In this rock mass, RSR and RMR are able to stabilize the tunnel whereas Q is inadequate. The support installed by the Project Authorities also stabilizes the tunnel at 1.2 percent deformation whereas the recorded deformation is 0.58 percent only which indicates that properties of rock mass might be better than those considered in the analysis.

Summarizing above, following conclusions may be drawn for non-squeezing grounds:

(a) GRC is very sensitive towards rock mass constants both for the original and broken rock masses. Rock mass constants in turn are dependent on RMR or GSI values, which altered even slightly, produce much change in values of rock mass constants. Therefore, it may be said that GRC is very sensitive towards RMR or GSI values which need to be very carefully estimated for proper interaction analysis.

(b) In 'Good rock' following RMR-system (Figs. 8.1 a, b and c), all the four classification systems i.e. RSR, RMi, Q and RMR recommended some amount of supports (RSR recommending heaviest one followed by RMi, Q and RMR) whereas through the analysis it has been found that hardly any support is needed, as deformations are very small.

Deformations recorded are generally matching with the corresponding deformations for supports installed by the Project Authorities at the equilibrium.

(c) In 'Fair rock' following RMR-system (Figs. 8.1 d, e, f, g, h and i), RSR recommended the heaviest support (actually true for all rock classes) followed by RMi, Q and RMR. Supports recommended by RMR are usually inadequate whereas those by RMi and Q are to be installed after some initial deformation to make them effective.

Deformations recorded are generally matching with the corresponding deformations for support installed by the Project Authorities at the equilibrium.

(d) In 'Poor rock' following RMR-system (Figs. 8.1 j, k and l), only steel ribs as recommended by RSR and RMR systems are adequate for stability whereas supports recommended by Q are inadequate.

Deformation recorded is matching in Fig. 8.1 j only with the corresponding deformation for supports installed by the Project Authorities at the equilibrium. In rest of the figures either properties of rock masses are better than considered in the analysis or there might be error in recording the deformations.

(e) In all the sections considered in 'Good', 'Fair' and 'Poor' rocks (following RMR-system), rock mass behaviour has been found to be elasto-plastic including for non-squeezing ground conditions also where it should have been elastic. It indicates that there is strength enhancement in rock masses in the tunnels (section 9.1.3).

8.5 ANALYSIS FOR SQUEEZING GROUND CONDITIONS

8.5.1 Mild Squeezing

(A) Ch. 23894–23901 m (N = 1.25, RCR = 52, H = 600 m) (Fig. 8.2 a)

(B) Ch. 26126–26133 m (N = 0.139, RCR = 25, H = 310 m) (Fig. 8.2 b)

In Fig. 8.2 a, GRC using N is lying far above GRC using RCR indicating that RCR under-estimates pressures in squeezing ground conditions. Supports recommended by RMi and Q are adequate in stabilizing the tunnel whereas those by RMR prove to be inadequate. It is very important that installation of supports is delayed so that some initial deformation has taken place and pressures are within reasonable limits. Support installed by the Project Authorities is adequate in stabilizing the tunnel and equilibrium deformation is even less than 0.5 percent whereas recorded deformation is 1.5 percent. This anomaly clearly indicates that the deformation characteristics of rib installed are not same as assumed in the theoretical analysis. Since the value of K is more than 1, in-situ stresses act non-uniformly around the tunnel periphery and stiffness of rib will be lower in the horizontal direction. Past experience showed that invert struts are essential in the steel ribs in cases of squeezing ground (Terzaghi, 1946).

In Fig. 8.2 b also, GRC using N is lying above GRC using RCR. The effect of lower depth in section (B) compared to section (A) is clearly reflected in Fig. 8.2 b, even though N-value is also less in this case. Since RMR recommended steel rib as the main support, it is able to stabilize the tunnel at 1.2 percent deformation, whereas Q proves to be inadequate, and RMi recommended SDSC. Support installed by the Project Authorities is also able to stabilize the tunnel at much lower deformation level as compared to the recorded deformation of 2.5 percent. Reason for this has already been mentioned in the previous paragraph. Excessive deformation, bending/twisting of rib has taken place.

8.5.2 Moderate Squeezing

(A) Ch. 21230–21240 m (N = 0.556, RCR = 35, H = 750 m) (Fig. 8.3 a)

(B) Ch. 26620–26623 m (N = 0.056, RCR = 15, H = 220 m) (Fig. 8.3 b)

In Fig. 8.3 a, GRC using N is lying far above GRC using RCR. Support recommended by RMR stabilizes the tunnel at 1.5 percent deformation whereas RMi and Q prove inadequate. Support installed by the Project Authorities stabilizes tunnel at 1.0 percent deformation whereas recorded deformation is 3.77 percent, reason has already been discussed in section 8.6.1. Steel ribs have buckled here.

In Fig. 8.3 b, GRC is almost similar to that in Fig. 8.3 a. N-value is lower in section (B) compared to section (A), however lower depth of rock cover in section (B) compared to section (A) equalizes the ground condition in both the sections (A) and (B) thereby making GRCs similar to each other. Support recommended by RMR stabilizes tunnel at 1.0 percent deformation whereas Q proves inadequate and RMi recommended SDSC. Support installed by the Project Authorities stabilizes tunnel at 0.5 percent deformation whereas recorded deformation is 3.1 percent, reason for which has already been discussed in section 8.6.1. In both the sections, bending/twisting of ribs have been observed at the site. Steel ribs have buckled.

8.5.3 High Squeezing

(A) Ch. 26120–26126 m (N = 0.052, RCR = 23, H = 310 m) (Fig. 8.4 a)

(B) Ch. 26608–26620 m (N = 0.019, RCR = 15, H = 210 m) (Fig. 8.4 b)

In Fig. 8.4 a, GRC using N is lying far above GRC using RCR. Support recommended by RMR stabilizes the tunnel at 2.6 percent deformation whereas Q proves inadequate and RMi recommended SDSC. Support installed by the Project Authorities stabilizes tunnel at 1.3 percent deformation whereas deformation recorded is 4.43 percent and still rising, reason for which has already been discussed in section 8.6.1.

In Fig. 8.4 b also, GRC using N is lying far above GRC using RCR. Support recommended by RMR just stabilizes the tunnel at 3.0 percent deformation whereas Q recommended CCA and RMi, SDSC. Support installed by the Project Authorities stabilizes the tunnel at 1.7 percent deformation whereas recorded deformation is 6.32 percent reason for which has already been discussed in section 8.6.1. Both the sections face bending/twisting of ribs.

Summarizing above, following conclusions may be drawn for squeezing grounds:

(a) GRC using N lies far above GRC using RCR in all sections indicating that RCR underestimates pressures in squeezing ground conditions.

(b) Supports recommended by RMR (steel rib, shotcrete and rock bolt) are adequate whereas those by Q (SFRS and rock bolt) prove inadequate in squeezing ground conditions. RMi generally recommended SDSC in all sections.

(c) Deformations at equilibrium for the supports installed by the Project Authorities are far less as compared to recorded deformations in all the sections, which indicate that the theoretical SRCs do not match with the actual field conditions.

EVALUATION OF Q AND RMi SYSTEMS

Although RMCs are becoming more widely used throughout the world, they are still in a growth phase, and there is no single accepted system. The real advances in empirical design methods will have to wait for a greater measure of consensus on an improved, integrated classification approach (Franklin, 1993). Hudson (1993) cautioned that the existing RMCs are useful when used under the conditions for which they were designed and for which their proponents agree that they are suitable.

Ram and Jethwa (1986), on the basis of studies conducted on the Maneri-Bhali Hydroelectric Project Stage-II located in Himalaya, concluded that the feasibility of RMC systems in Himalayan formations has to be studied in the various works in Himalaya. They felt the need for modification in at least one of the existing systems, which holds good for the Himalayan region.

The Q and RMi systems have been evaluated in this chapter.

9.1 ROCK MASS QUALITY (Q) – SYSTEM

Bieniawski (1976) regards the Q-system as an excellent approach for initial support in tunnels but considers it to be complex to be easily applied to wide ranging geological conditions. According to him some analyses suggest that the basic rating suggested in the Q-system are highly sensitive to most difficult parameters to evaluate and this makes the choice of rating quite critical.

9.1.1 Proposed SRF Values for Moderately Jointed Rock

SRF is most contentious parameter to evaluate for estimating Q-value. Barton et al. (1974) have considered four broad conditions of rock masses for which ranges of SRF-values have been recommended (APPENDIX - V, section 6).

In the second rock condition i.e. 'competent rock, rock stress problems', further categories have been indicated based on ratios of q_c/σ_1 and σ_θ/q_c and corresponding SRF-values have been recommended. In these categories of rock conditions, first three i.e. H, J and K (low stress, medium stress and high stress respectively) are understood to be applicable for both massive as well as moderately jointed rock masses. On the other hand in rest of other three conditions named L, M and N (moderate slabbing, slabbing and rock burst and heavy rock burst respectively), it has been clearly mentioned that these pertain to massive rock only which should really be the case since slabbing, rock burst etc. are associated with competent and massive rocks only. But what about if rock is not massive but moderately jointed? Therefore, in a condition in which ratios q_c/σ_1 and q_c/σ_0 lie in ranges corresponding to conditions L, M or N and rock is jointed moderately, Barton's table do not suggest SRF-values to be considered. If SRF-values are selected purely on the basis of q_c/σ_1 and σ_θ/q_c, which is really the only option left for the user of the Q-system, unmindful of the fact whether rock mass is massive or moderately jointed, results for massive rock might be correct but for moderately jointed rock masses these are bound to be incorrect.

In the present RMC following the Q-system, for tunnel section falling in categories L, M and N, SRF-values selected are 9, 15 and 20 respectively. But in all these sections, rocks are jointed moderately and not massive; and two or more joint sets are present in all of them. It has therefore been apprehended that in these sections, which are 431 in numbers, Q-values might be erroneous and further estimation of pressures and design of support systems would consequently be incorrect.

Therefore to find a suitable solution of the problem concerning selection of SRF in moderately jointed rock masses experiencing high stresses, it has been considered appropriate to utilize N. Reasons for selecting N in this regard are as follows:

(i) N has been derived from Q with the only difference being that in N, SRF has been considered equal to 1 (Eq. 2.11). Therefore, uncertainties involved in selection of proper SRF have been removed.

(ii) The empirical correlation developed for estimating support pressure using N (Eq. 6.29) is based on detailed field studies carried out for several tunnelling projects located in Himalaya and the Peninsular India. Therefore, correlations using N are applicable in the present work since the HRT of Nathpa Jhakri Project lies in Himalayan region only.

Comparison has been made between pressures estimated from Q (Eq. 6.23) and N in non-squeezing ground conditions (Eq. 6.29) for categories L, M and N separately. Number of tunnel sections falling in L, M and N are 158, 132 and 141 respectively.

Figures 9.1, 9.2 and 9.3 show comparison between roof pressures estimated from N and Q for categories L, M and N respectively. From these figures, it is clear that Q overestimates pressure compared to that from N. Therefore, SRF-values have been lowered from current 9, 15 and 20 for L, M and N respectively so that the pressure estimated from Q is reduced and comes closer to that estimated from N.

Results of this study have been presented in Table 9.1 in which an estimate has been shown regarding number of sections lying in each group i.e. $P_N < P_Q$, $P_N = P_Q$ and $P_N > P_Q$. Figures in parenthesis show maximum deviation of pressure. Standard deviation and correlation have also been indicated in this table.

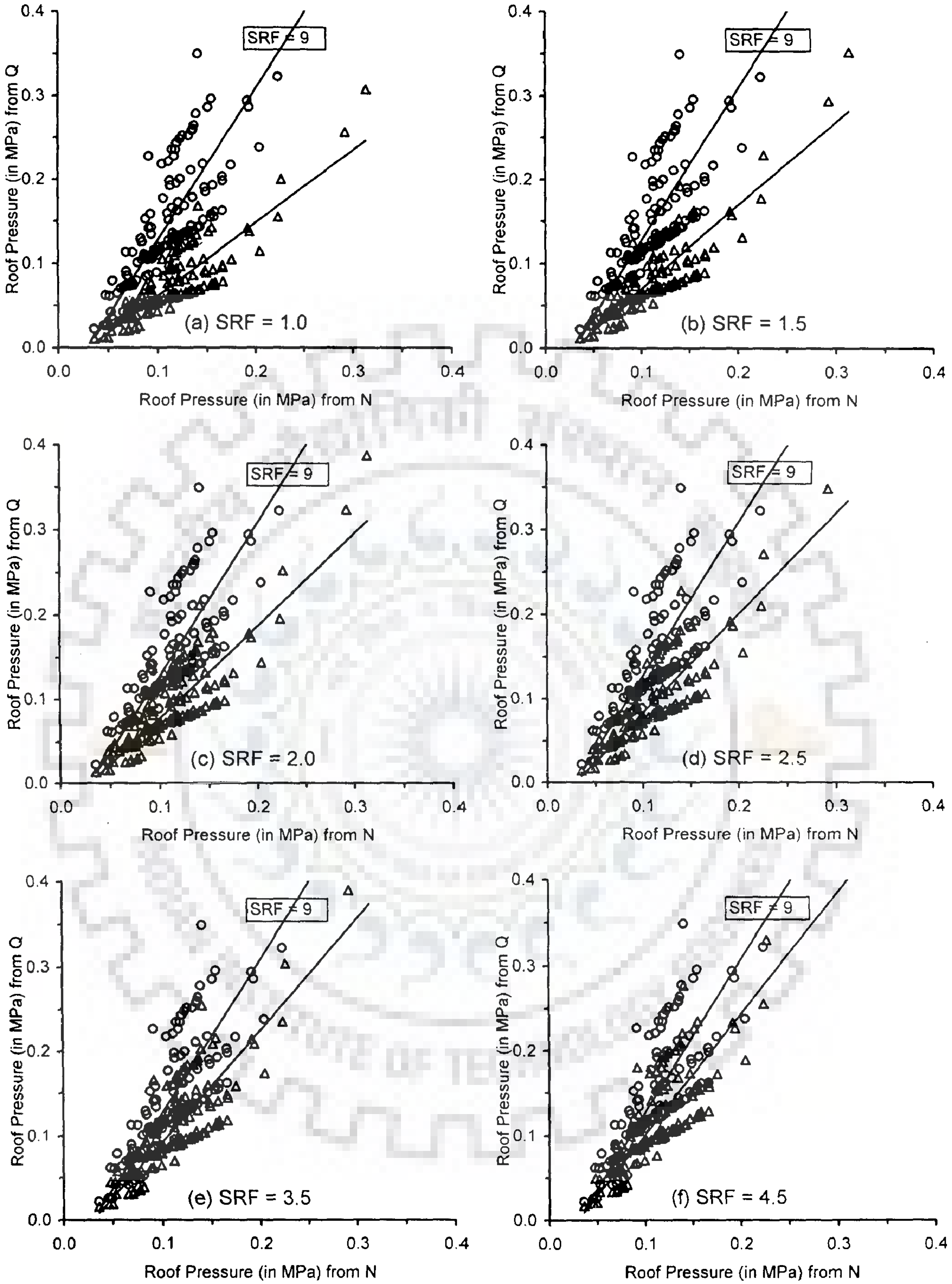


Fig. 9.1 Comparison of roof pressure estimated from N and Q for category L

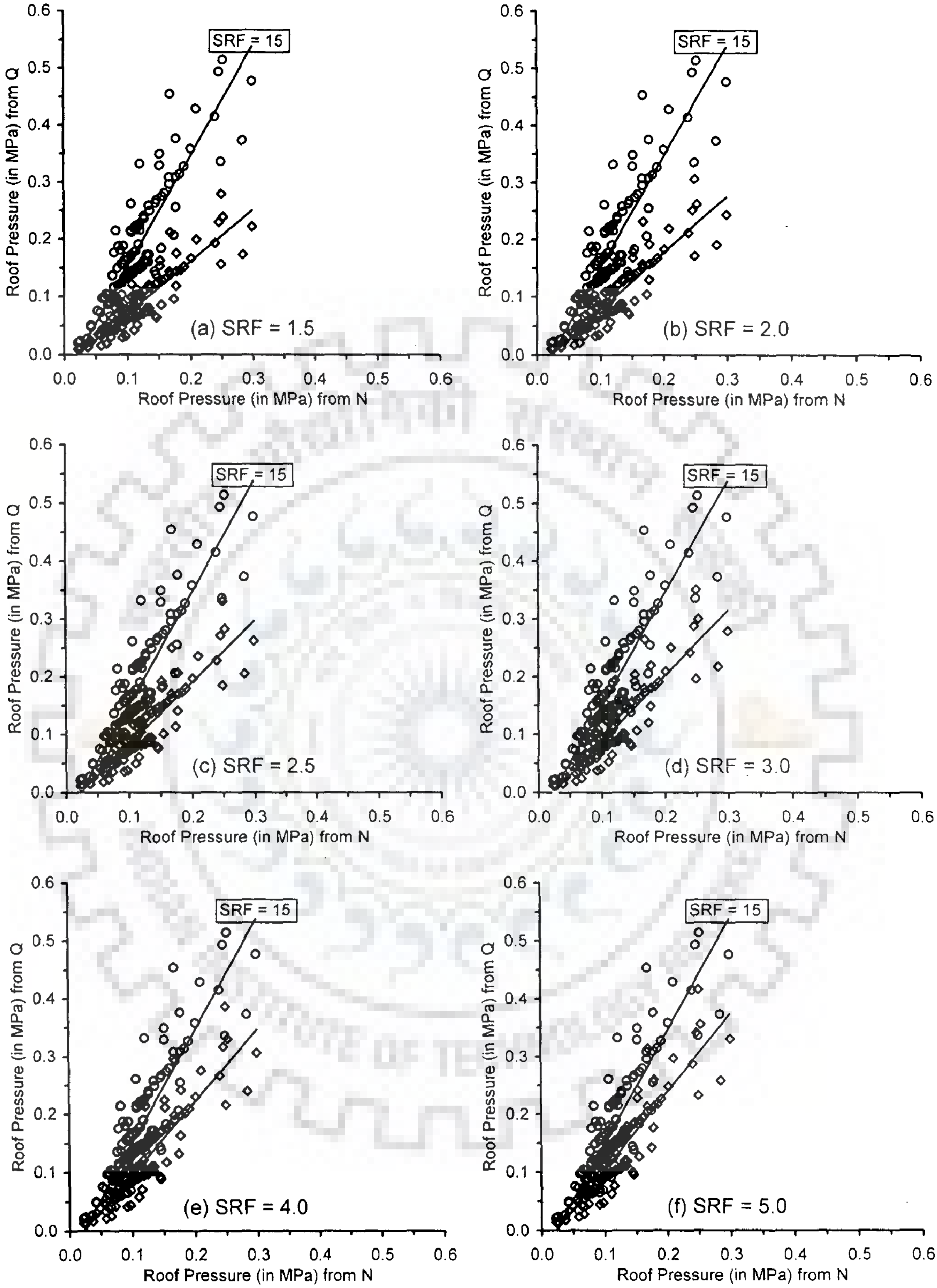


Fig. 9.2 Comparison of roof pressure estimated from N and Q for category M

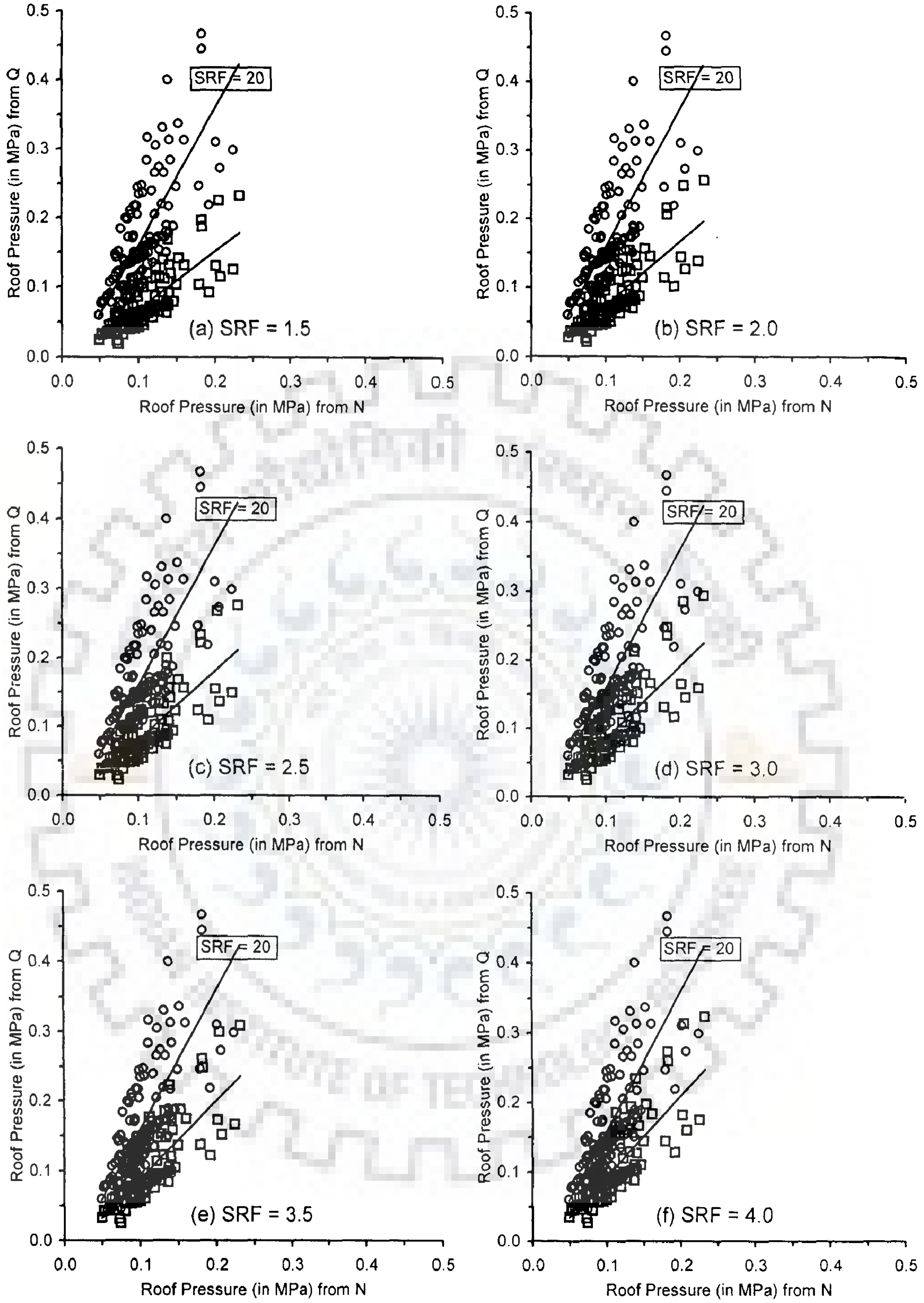


Fig. 9.3 Comparison of roof pressure estimated from N and Q for category N

Table 9.1 Comparison of Roof Pressure Estimated from N and Q

Cate Gory	SRF	Number of Sections (in percent)			Standard Deviation	Correlation
		$P_N < P_Q$	$P_N = P_Q$	$P_N > P_Q$		
L	1.0	1 (0.03)	15 (±0.015)	84 (0.09)	0.045	$P_Q = 0.9\ P_N - 0.025$ (r = 0.85)
	1.5	2 (0.05)	23 (±0.015)	75 (0.08)	0.038	$P_Q = 1.0\ P_N - 0.028$ (r = 0.85)
	2.0	15 (0.07)	15 (±0.015)	70 (0.07)	0.035	$P_Q = 1.1\ P_N - 0.032$ (r = 0.85)
	2.5	16 (0.10)	22 (±0.015)	62 (0.06)	0.033	$P_Q = 1.2\ P_N - 0.034$ (r = 0.85)
	3.5	21 (0.15)	32 (±0.015)	47 (0.05)	0.035	$P_Q = 1.3\ P_N - 0.038$ (r = 0.85)
	4.5	27 (0.20)	43 (±0.015)	30 (0.04)	0.040	$P_Q = 1.4\ P_N - 0.042$ (r = 0.85)
M	1.5	3 (0.04)	16 (±0.015)	81 (0.08)	0.039	$P_Q = 0.90\ P_N - 0.020$ (r = 0.90)
	2.0	5 (0.06)	29 (±0.015)	66 (0.08)	0.033	$P_Q = 1.00\ P_N - 0.023$ (r = 0.90)
	2.5	10 (0.08)	36 (±0.015)	54 (0.08)	0.031	$P_Q = 1.10\ P_N - 0.024$ (r = 0.90)
	3.0	14 (0.10)	33 (±0.015)	53 (0.07)	0.030	$P_Q = 1.14\ P_N - 0.026$ (r = 0.90)
	4.0	29 (0.15)	35 (±0.015)	36 (0.06)	0.033	$P_Q = 1.26\ P_N - 0.030$ (r = 0.90)
	5.0	34 (0.20)	46 (±0.015)	20 (0.05)	0.038	$P_Q = 1.35\ P_N - 0.031$ (r = 0.90)
N	1.5	2 (0.03)	20 (±0.015)	78 (0.10)	0.040	$P_Q = 0.84\ P_N - 0.02$ (r = 0.78)
	2.0	6 (0.05)	21 (±0.015)	73 (0.09)	0.036	$P_Q = 0.93\ P_N - 0.02$ (r = 0.78)
	2.5	11 (0.06)	23 (±0.015)	66 (0.08)	0.033	$P_Q = 1.00\ P_N - 0.02$ (r = 0.78)
	3.0	18 (0.08)	21 (±0.015)	61 (0.07)	0.032	$P_Q = 1.06\ P_N - 0.02$ (r = 0.78)
	3.5	20 (0.10)	25 (±0.015)	55 (0.07)	0.032	$P_Q = 1.12\ P_N - 0.02$ (r = 0.78)
	4.0	22 (0.10)	28 (±0.015)	50 (0.06)	0.032	$P_Q = 1.20\ P_N - 0.02$ (r = 0.78)

Considering standard deviation, maximum deviation, correlation and number of sections, SRF-values proposed for overstressed and moderately jointed rock mass may be as follows:

(a) Category L: 1.5 - 2.0

(b) Category M: 2.0 - 2.5

(c) Category N: 2.5 - 3.0

Barton's Table may therefore be modified slightly by incorporating SRF-values for overstressed and moderately jointed rock comprising two or more joint sets typical in Himalaya as indicated in Table 9.2.

Table 9.2 Stress Reduction Factor for Jointed Rock for Categories L, M and N

Category	Rock Stress Problems		q_c/σ_1	σ_θ/q_c	SRF (Old)	SRF (New)
H	Low stress, near surface open joints		> 200	< 0.01	2.5	2.5
J	Medium stress, favourable stress condition		200 - 10	0.01 - 0.3	1.0	1.0
K	High stress, very tight structure (usually favourable to stability, may be unfavourable to wall stability)		10 - 5	0.3 - 0.4	0.5 - 2.0	0.5 - 2.0
L	Moderate slabbing after >1hr in:	massive rock	5 - 3	0.5 - 0.65	5 - 9	5 - 50
		moderately jointed rock			--	**1.5 - 2.0**
M	Slabbing and rock burst after a few minutes in:	massive rock	3 - 2	0.65 - 1.0	9 - 15	50 - 200
		moderately jointed rock			--	**2.0 - 2.5**
N	Heavy rock burst (strain-burst) and immediate deformations in:	massive rock	< 2	> 1	15 - 20	200 - 400
		moderately jointed rock			--	**2.5 - 3.0**

9.1.2 Proposed Correlation for SRF for Moderately Jointed Rock

For 592 tunnel sections falling in 'competent rock, rock stress problem' of Barton's Table, SRF has been computed by equating Eqs. 6.23 and 6.29 as follows:

$$SRF = \frac{3375\, J_r^{\,3}}{J_n^{\,3/2}} . (0.12\, H^{0.1}\, a^{0.1} - 0.038\, N^{0.33})^3 \qquad (9.1)$$

SRF obtained form Eq. 9.1 is then correlated from the present study as:

$$SRF = 5.84 \left(\frac{q_c}{H}\right)^{0.001} . \frac{J_r^{\,3}}{J_n} + 2.58 \qquad (9.2)$$

where, q_c is in MPa and H in m.

Coefficient of correlation of Eq. 9.2 is 0.90.

Therefore, for moderately jointed rocks falling in 'competent rock, rock stress problem' of Barton's Table, SRF may be obtained from Eq. 9.2 to be used in computing Q-value. The pressure estimated from Eq. 6.23 using Q then would be almost same as estimated from Eq. 6.29 using N.

9.1.3 Comparison of Mohr's and Singh's Criteria of Strength of Rock Mass

Singh et al. (1997, ref. 153) observed considerable enhancement of rock mass strength around tunnels. Rock masses surrounding the tunnel perform much better than theoretical expectations, except near thick and plastic shear zones, faults, thrusts, intra-thrust zones and in water charged rock masses. On the basis of analysis of data collected from 60 tunnels, they recommended that the mobilized crushing strength of the rock mass is as follows:

$$q_{cmass} = 7\, \gamma\, Q^{1/3} \quad \text{MPa} \qquad (9.3)$$

(for $Q < 10$, $100 > q_c > 2$ MPa, $J_w = 1$ and $J_r / J_a < 0.5$)

$$q_{cmass} = \frac{5.5\, \gamma\, N^{1/3}}{B^{0.1}} \quad \text{MPa} \qquad (9.4)$$

where, γ is in gm/cc and B in m.

Grimstad and Bhasin (1996) have modified Eq. 9.3 as Eq. 9.5, which has been found suitable for good and massive hard rock masses (Singh et al., 1997, ref. 153).

$$q_{cmass} = 7\,\gamma\, f_c . Q^{1/3} \quad \text{MPa} \tag{9.5}$$

(for $Q > 10$ and $q_c > 100$ MPa)

where, $f_c = q_c / 100$

$= 1$ for $q_c < 100$ MPa.

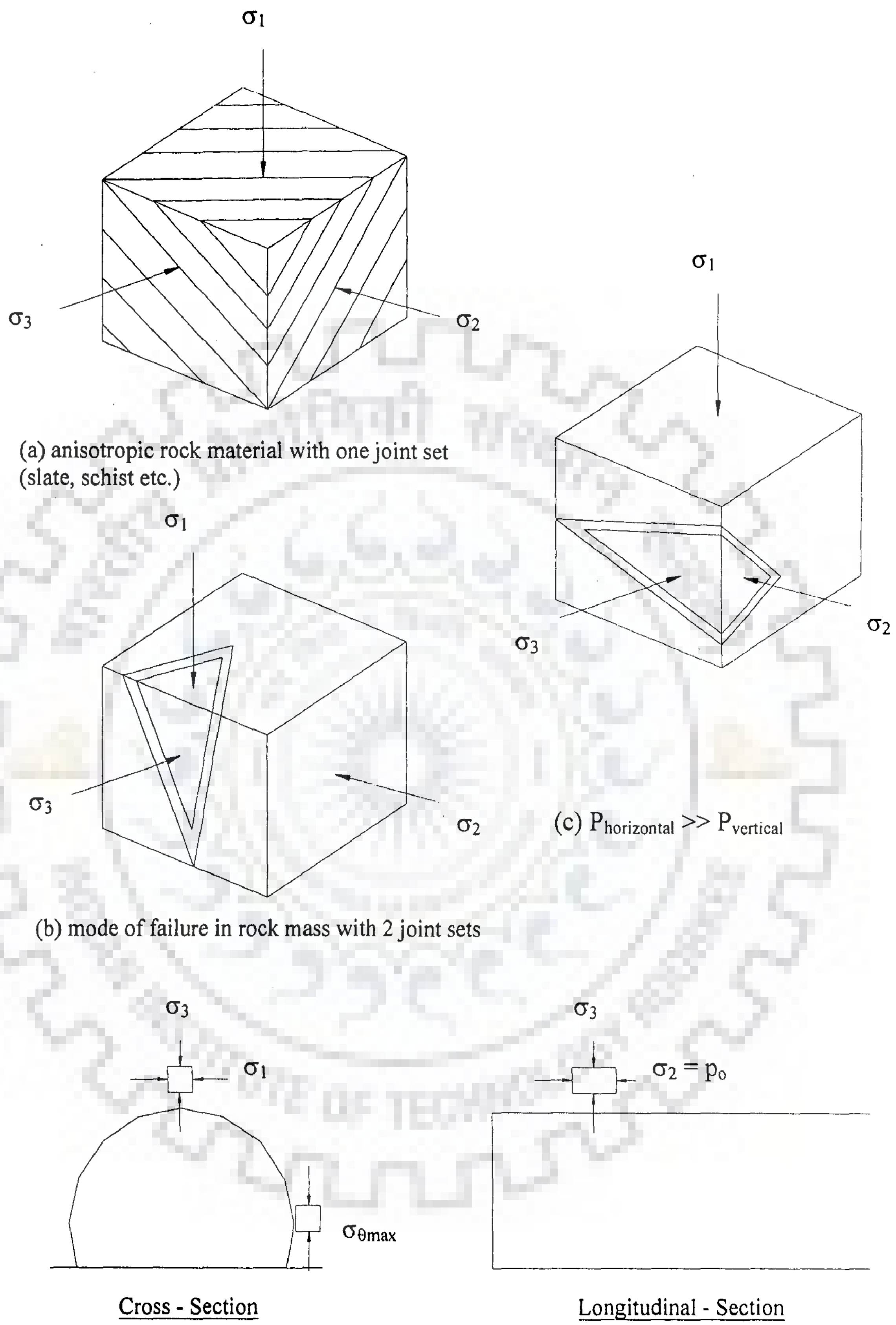
Singh et al. (1998) have suggested a new failure theory for jointed rock masses. In the rock mass with two or more joint sets as shown in Fig. 9.4, if high intermediate principal stress is applied on the two opposite faces then the chances of wedge failure are more than the chances of planar failure. The shear stress along the line of intersection of joint planes will be proportional to $\sigma_1 - \sigma_3$ because σ_3 will try to reduce shear stress. The normal stress on both the joint planes will be proportional to $(\sigma_2 + \sigma_3)/2$. The criterion for peak failure at low confining stresses is as follows:

$$\sigma_1 - \sigma_3 = q_{cmass} + \frac{A\,(\sigma_2 + \sigma_3)}{2} \tag{9.6}$$

where, $$A = \frac{2\sin\phi_p}{(1 - \sin\phi_p)} \tag{9.7}$$

According to Singh et al. (1998), the significant strength enhancement is due to σ_2 or in-situ stress along tunnels which pre-stresses rock wedges and prevents their failure both in the roof and in the walls. Another cause of strength enhancement is higher UCS of rock mass due to higher E_d because of constrained dilatancy and restrained fracture propagation near excavation face only in the underground structures.

The strength criterion by Singh et al. (1998) is different from Mohr's strength theory, which works well for soils and isotropic materials. According to them, there is a basic difference in the structure of soil and rock masses. Soils generally have no pre-existing planes of weaknesses and so



(a) anisotropic rock material with one joint set (slate, schist etc.)

(b) mode of failure in rock mass with 2 joint sets

(c) $P_{horizontal} >> P_{vertical}$

(d) direction of σ_1, σ_2 and σ_3 in the tunnel

Fig. 9.4 Mode of failure of anisotropic rock mass under polyaxial stress conditions failure can occur on a typical plane with dip direction towards σ_3. However, rocks have pre-existing planes of weaknesses like joints and bedding planes etc. As such, failure occurs mostly along these planes of weaknesses. In the triaxial test on rock masses, planar failure takes place along the weakest joint plane. In polyaxial stress field, a wedge type of failure may be dominant mode of failure, if $\sigma_2 >> \sigma_3$. Therefore, according to Singh et al. (1998), Mohr's theory is not applicable for anisotropic and jointed rock masses.

In the present work, an attempt has been made to see the effect of strength enhancement in rock mass and then compare it with Mohr's criterion. The tunnel sections chosen for this purpose encounter greatest rock covers in whole of the length of tunnel. In section 4.3, it has been mentioned that the HRT from Ch. 10900 m to Ch. 12700 m lies under a rock cover of more than 1000 m with a maximum value of 1430 m at Ch. 11435 m. Therefore, this stretch of the tunnel has been selected for the present study. Table 9.3 lists tunnel sections where rock cover is more than 1000 m. In this table, apart from rock cover and q_c, Q-parameters and Q-values have also been indicated in all the sections considered. From the value of J_n, it is clear that is all the sections considered, rock mass contains two or more joint sets. Therefore, it is quite clear that this study pertains to jointed rock mass only. It is understood that there are sections between Ch. 10900 m and Ch. 11435 m, where rock might be massive but since the Project Authorities were not able to provide data for this stretch, the same could not be included in the present study.

Equation 9.6 suggests the following strength enhancement represented by q'_{cmass}, by putting $\sigma_1 = q'_{cmass}$, $\sigma_3 = 0$, $\sigma_2 = p_o$ along tunnel axis in Fig. 9.4 d:

$$q'_{cmass} = q_{cmass} + 0.5\, A\, p_o \tag{9.8}$$

In Eq. 9.8, q_{cmass} has been computed from Eq. 9.3, A from Eq. 9.7. SRF-value selected for estimating Q is 2.5 instead of 20 since rock mass is moderately jointed.

Table 9.3 Comparison of Mohr's and Singh's Criteria of Strength of Rock Mass

S. No	Chainage (in m)	Rock Cover (in m)	UCS (in MPa)	Q Parameters						Q Value	ϕ_p deg	p_o MPa	σ_θ MPa	q_{cmass} MPa	q'_{cmass} MPa	$\frac{\sigma_\theta}{q_c}$	$\frac{\sigma_\theta}{q'_{cmass}}$	Rock Behaviour ($Q_{original}$)	Rock Behaviour (Observed)
				RQD	Jn	Jr	Ja	Jw	SRF										
1	11435-11446	1430	50	70	6	2	2	1	2.5	4.7	45	38.6	77.2	31.6	124.8	1.5	0.62	Heavy burst	Mod. slabbing with noise
2	11446-11459	1420	32	60	6	2	2	1	2.5	4.0	37	38.3	76.7	30.0	87.9	2.4	0.87	Heavy burst	Mod. slabbing with noise
3	11459-11525	1420	50	67	6	2	2	1	2.5	4.5	45	38.3	76.7	31.1	123.7	1.5	0.62	Heavy burst	Mod. slabbing with noise
4	11621-11631	1320	32	55	9	1.5	2	1	2.5	1.8	37	35.6	71.3	23.1	77.0	2.2	0.93	Heavy burst	Mod. slabbing with noise
5	11634-11643	1300	50	70	6	1.5	2	1	2.5	3.5	45	35.1	70.2	28.7	113.4	1.4	0.62	Heavy burst	Mod. slabbing with noise
6	11643-11650	1300	60	60	6	1.5	3	1	2.5	2.0	45	35.1	70.2	23.8	108.6	1.2	0.65	Heavy burst	Mod. slabbing with noise
7	11656-11662	1300	55	55	6	1.5	3	1	2.5	1.8	45	35.1	70.2	23.1	107.9	1.3	0.65	Heavy burst	Mod. slabbing with noise
8	11662-11796	1300	50	65	6	1.5	2	1	2.5	3.3	45	35.1	70.2	28.0	112.7	1.4	0.62	Heavy burst	Mod. slabbing with noise
9	11860-11917	1230	50	67	6	1.5	3	1	2.5	2.2	45	33.2	66.4	24.7	104.9	1.3	0.63	Heavy burst	Mod. slabbing with noise
10	12044-12070	1180	42	70	6	2	2	1	2.5	4.7	55	31.9	63.7	31.6	175.9	1.5	0.36	Heavy burst	Mod. slabbing with noise
11	12070-12077	1180	34	60	6	1.5	3	1	2.5	2.0	30	31.9	63.7	23.8	55.7	1.9	1.14	Heavy burst	Mod. slabbing with noise
12	12087-12223	1180	42	67	6	1.5	2	1	2.5	3.4	45	31.9	63.7	28.3	105.2	1.5	0.61	Heavy burst	Mod. slabbing with noise
13	12223-12267	1100	42	75	4	2	2	1	2.5	7.5	45	29.7	59.4	37.0	108.7	1.4	0.55	Heavy burst	Mod. slabbing with noise
14	12273-12322	1090	50	70	4	3	3	1	2.5	7.0	45	29.4	58.9	36.2	107.2	1.2	0.55	Heavy burst	Mod. slabbing with noise
15	12359-12428	1060	50	75	6	1.5	2	1	2.5	3.8	45	28.6	57.2	29.4	98.5	1.1	0.58	Heavy burst	Mod. slabbing with noise

σ_θ/q_c in Table 9.3 in all the sections is more than 1 predicting heavy rock burst as per category N of Barton's Table (APPENDIX – IV, section 6b) which is indicated in Table 9.3 also in the column named 'Rock Behaviour ($Q_{Original}$)'.

Enhanced strength of rock mass has also been computed from Eq. 9.8 and ratios σ_θ/q'_{cmass} determined as shown in Table 9.3. Effect of σ_θ/q'_{cmass} is also indicated in Table 9.3 in the column named 'Rock Behaviour (Observed)'. Comparison of the two ratios σ_θ/q_c and σ_θ/q'_{cmass} has been presented in Fig. 9.5 also.

From Fig. 9.5, it is clear that ratio (stress/strength) as per Singh's criterion is far less than that as per Mohr's criterion. Also from Table 9.3, it is clear that heavy rock burst predicted by Mohr's criterion is moderated to a great extent if Singh's criterion is adopted. During the excavation of these tunnel reaches also, it has been observed that behaviour of rocks actually follows Singh's criterion. At none of the sections, there were heavy or moderate rock burst since rock mass is jointed and also due to the enhancement of strength in rock masses, there was no stress related problem in the tunnel. Only an instance of rock bolt plates breaking away was observed at Ch. 11467-11483 m where σ_θ/q'_{cmass} is 0.62 (Table 9.3). Slabbing occurred after more than one hour of blasting. Only light supports (Class II, III and IV) have been installed in this tunnel reach. According to Grimstad and Barton (1993) also, in areas affected by high stresses, no real stress problems were observed in areas with heavily jointed or crushed rock (i.e. low ratios of RQD/J_n).

Therefore, it may be concluded from this study that for strength of rock mass, Singh's criterion is more appropriate compared to Mohr's criterion in moderately jointed rocks.

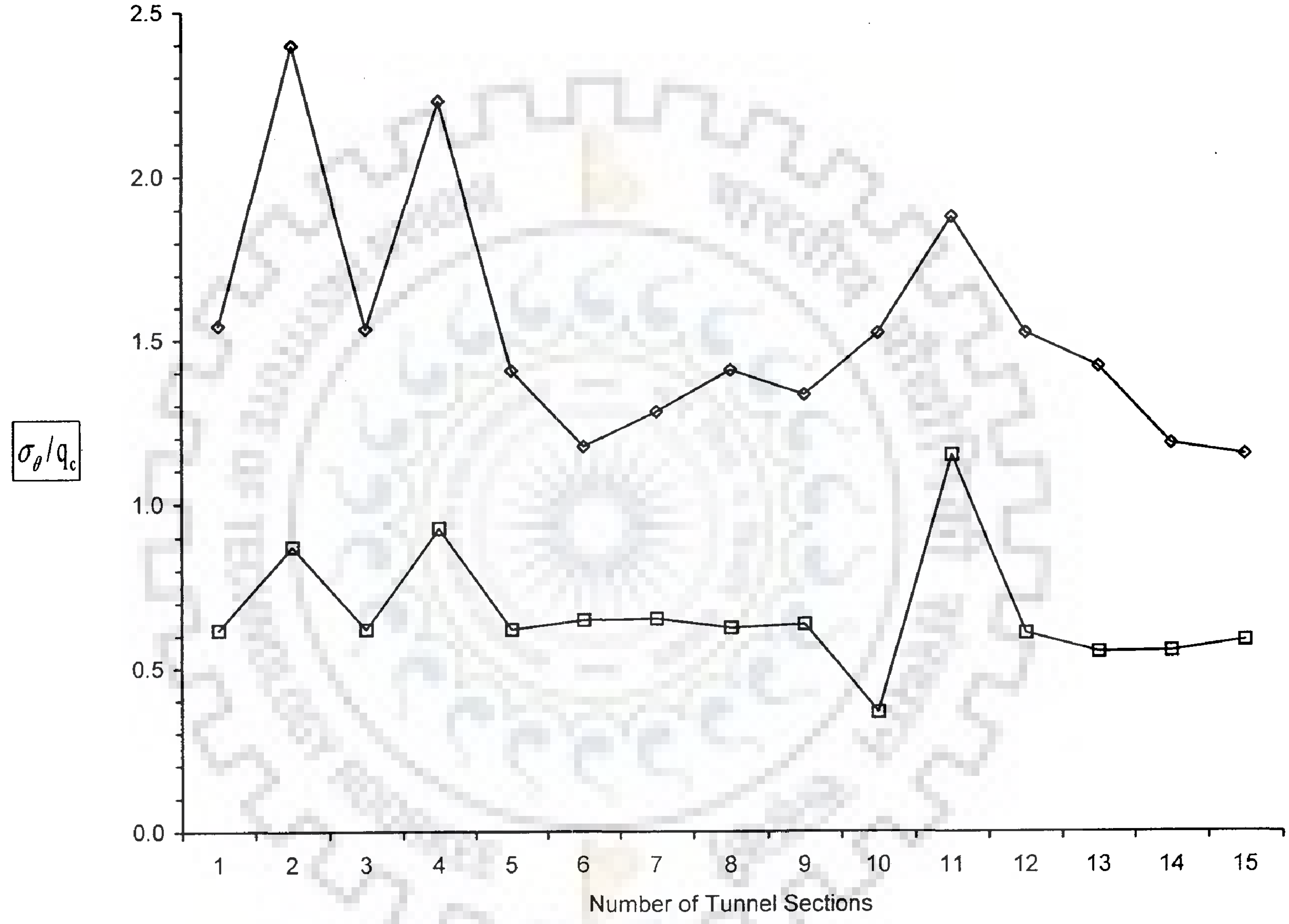


Fig. 9.5 Comparison between Mohr's and Singh's criteria of strength of rock mass

9.1.4 Prediction of Ground Condition

(i) Based on σ_θ/q'_{cmass}

In the previous section i.e. 9.1.3, it has been clearly observed that there is considerable strength enhancement in rock mass in a confined state due to intermediate principal stress. For prediction of ground condition, therefore it is better to estimate ratio σ_θ/q'_{cmass} rather than σ_θ/q_c. The present work contains 50 sections covering a length of 1133 m where squeezing ground conditions were encountered. The ratio σ_θ/q'_{cmass} in these sections has been estimated and following results have been obtained regarding degree of squeezing as indicated in Table 9.4.

Table 9.4 Prediction of Squeezing Ground Condition based on σ_θ/q'_{cmass}

S.No.	σ_θ/q'_{cmass}	Squeezing Condition (Observed)
1.	2 - 3	Mild
2.	3 - 4	Moderate
3.	> 4	High

(ii) Based on Q and N

Correlations for squeezing ground conditions based on Q and N have been obtained as shown in Figs. 9.6 a and b by plotting data for 50 sections encountering squeezing ground conditions. Singh's criterion (Eq. 2.14) has also been indicated in Fig. 9.6 a. On the basis of Figs. 9.6 a and b, Tables 9.5 a and b have been prepared for prediction of squeezing ground condition based on Q and N respectively.

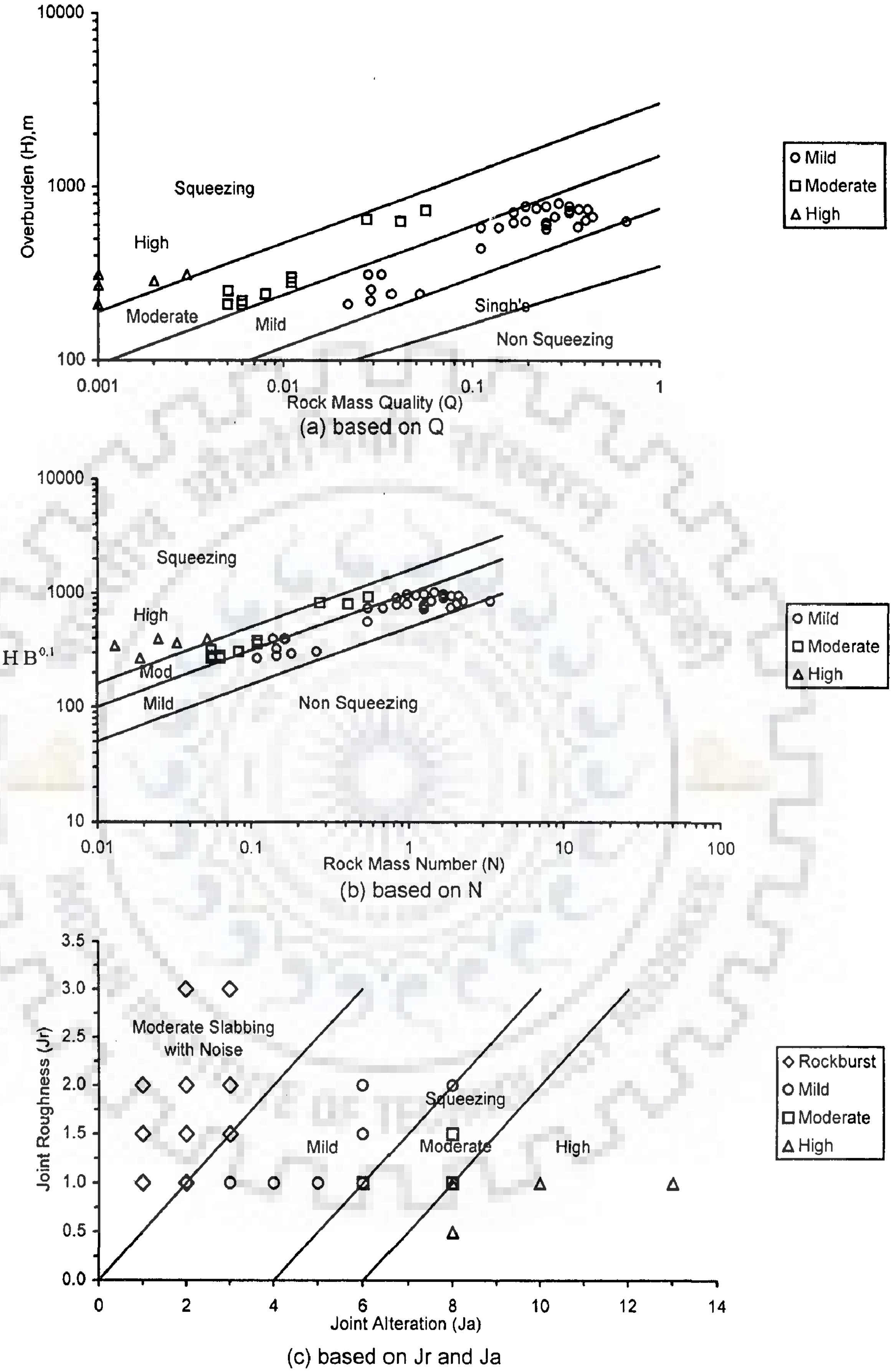


Fig. 9.6 Prediction of ground condition

Table 9.5 a Prediction of Squeezing Ground Condition based on Q

S. No.	Ground Conditions		Correlations	J_r and J_a
1.	Squeezing	Mild	$1500Q^{0.4} \geq H \geq 750Q^{0.4}$	$0.5J_a \geq J_r \geq 0.5J_a - 2$
2.		Moderate	$3000Q^{0.4} \geq H \geq 1500Q^{0.4}$	$0.5J_a - 2 \geq J_r \geq 0.5J_a - 3$
3.		High	$H \geq 3000Q^{0.4}$	$J_r \leq 0.5J_a - 3$

Table 9.5 b Prediction of Squeezing Ground Condition based on N

S. No.	Ground Conditions		Correlations	J_r and J_a
1.	Squeezing	Mild	$1000N^{0.5} \geq HB^{0.1} \geq 500N^{0.5}$	$0.5J_a \geq J_r \geq 0.5J_a - 2$
2.		Moderate	$1600N^{0.5} \geq HB^{0.1} \geq 1000N^{0.5}$	$0.5J_a - 2 \geq J_r \geq 0.5J_a - 3$
3.		High	$HB^{0.1} \geq 1600N^{0.5}$	$J_r \leq 0.5J_a - 3$

(iii) Based on J_r and J_a

Correlations between J_r and J_a have been presented in Fig. 9.6 c where overstress conditions existed in the tunnel alignment. On the basis of Fig. 9.6 c, Table 9.5 c has been prepared for prediction of ground condition based on J_r and J_a. It may be noted that both criteria (H and J_r or J_a) must be satisfied in the squeezing ground conditions.

Table 9.5 c Prediction of Ground Condition based on J_r and J_a

S. No.	Ground Conditions		Correlations
1.	Moderate slabbing with noise		$J_r \geq 0.5J_a$
2.	Squeezing	Mild	$0.5J_a \geq J_r \geq 0.5J_a - 2$
3.		Moderate	$0.5J_a - 2 \geq J_r \geq 0.5J_a - 3$
4.		High	$J_r \leq 0.5J_a - 3$

9.2 ROCK MASS INDEX (RMi) – SYSTEM

9.2.1 Applicability in Jointed Rock

In estimation of RMi, V_b plays a very significant role. It has been found in the present work that evaluation of V_b is not a simple job. Hoek et al. (1992) are of the opinion that strength characteristics for jointed rock masses are controlled by the block shape and size. On the other hand, according to Pandey (1997), due to constrained dilatancy and interlocking of rock blocks the strength of a rock mass in tunnels tends to be equal to the strength of a rock material. In the present work, V_b has been estimated by number of joint sets and their spacing (Eqs. 5.13 through 5.16).

Palmstrom (1995a, b, c, d, 1996a) developed RMi to characterize the strength of rock mass and according to him RMi, is a volumetric parameter indicating the approximate UCS of a rock mass.

A comparison has therefore been made between q_{cmass} estimated from Eq. 9.3 and RMi as shown in Fig. 9.7. From this figure, it is clear that RMi-values (= rock mass strength) are far less than q_{cmass} and no correlation could be established between the two. Reason for this may be that V_b is very low in the rock mass considered in the present study. RMi consists of q_c and JP which is

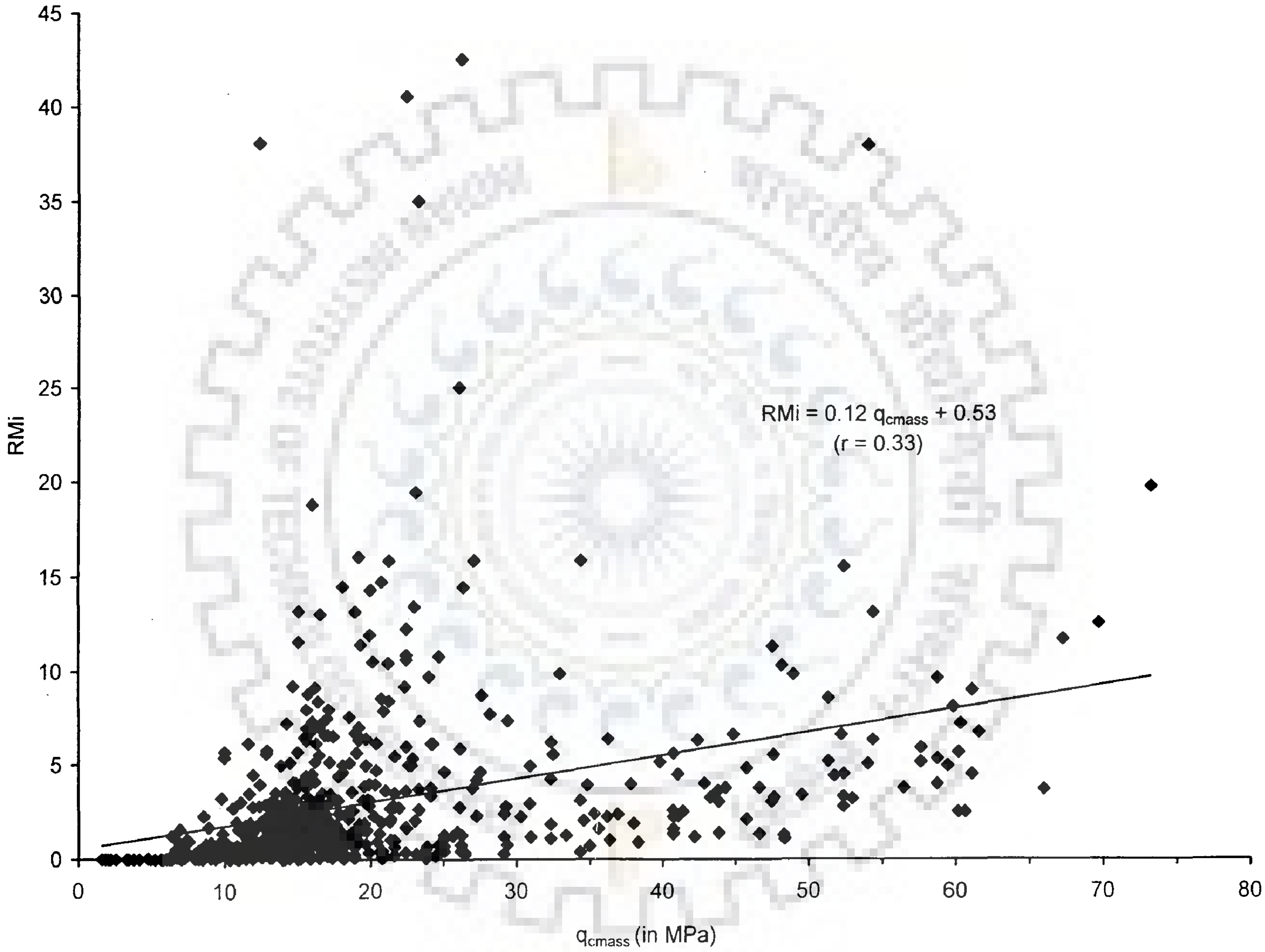


Fig. 9.7 Comparison between q_{cmass} and RMi

dependent on V_b and jC. In jC roughness and alteration are almost similar to those considered by Barton (1974). Therefore, V_b is the only parameter, which might have influenced estimation of RMi. It means that RMi is very sensitive in jointed rocks or in other words its proper estimation is heavily dependent on the correct evaluation of V_b.

9.2.2 Correlation for Roof Pressure

Palmstrom (2000a) proposed charts for the design of support in blocky and continuous grounds separately (APPENDIX - VI) without any correlation for estimation of roof pressure. In the present work, a correlation has been developed for estimation roof pressure in blocky ground by estimating capacities of supports recommended by the RMi-system. Capacities of supports have already been estimated by modified semi - empirical method (section 7.2). The proposed correlation is as follows:

$$p_v = \frac{0.01\, S_r^{\,1.15}}{RMi^{0.33} . H^{0.1}} - 0.05 \quad \text{MPa} \tag{9.9}$$

The coefficient of correlation of Eq. 9.9 is 0.94. The size ratio S_r is defined by Eq. 6.32.

CONCLUSION

10.1 SUMMARY OF PRESENT WORK

In the present study, different RMC systems such as RSR, RMR, Q and RMi have been applied for the HRT of Nathpa Jhakri Project. Other methods of RMC such as N and RCR have also been applied. The supports recommended by these systems have been evaluated and compared with those actually installed at the site. Highlights of this study are as follows:

- Classification parameters and other geotechnical data for 685 tunnel sections covering a length of about 22.2 km out of a total length of 27.4 km have been collected and compiled. Typical data is presented in Table 5.19.
- A new interactive computer program **ROMAC** has been developed in C++ language for classification using RSR, RMR, Q and RMi systems (APPENDIX - VIIA).
- Correlations between different classification systems have been developed as shown in Figs. 5.6 through 5.23 and Tables 5.24 and 5.25.
- On the basis of recommendations given by Bieniawski (1989) (APPENDIX - IVF), Eqs. 6.7 through 6.11 have been developed to interpolate support details for a particular RMR-value.
- The support capacity estimated from the semi - empirical method suggested by Singh et al. (1995a, b) may not be applicable beyond 0.25 MPa as indicated in Fig. 7.2. A correlation has therefore been established as indicated in Eq. 7.12.

- The interactive computer programs have been developed in C++ language for determination of: (i) GRC as per the formulations of Ladanyi (1974) and Brown et al. (1983) and (ii) SRC for shotcrete/rock bolt and steel rib/concrete supports.
- Wickham et al. (1974) have given recommendations for estimation of spacing of steel ribs for different tunnel diameters for datum condition. Moreover, these recommendations have been made for tunnel diameters up to 9 m only. Therefore, to facilitate use of steel ribs manufactured in India as per Indian Standards, Fig. III.1 and Table III.1 (APPENDIX - IIID) have been developed to get the spacing of various steel ribs for datum conditions beyond 9 m also.
- Support charts (Fig. 7.4) for estimating capacity of shotcrete/SFRS and rock bolts systems have been developed from the database contained in this thesis.

10.2 CONCLUSIONS

(A) Classification Systems

(i) In the Q-system, Barton's recommendations do not suggest SRF-values for 'competent rock, rock stress problems' category for moderately jointed rocks in categories L, M and N. On the basis of present work, new SRF-values have been proposed for moderately jointed rocks. A new correlation has also been developed for this purpose (Eq. 9.2).

(ii) Singh's criterion is more appropriate for strength of rock mass (Fig. 9.5 and Table 9.3) on the basis of observed tunnelling conditions as compared to Mohr's criterion for the moderately jointed rocks.

(iii) RMi uses a number of parameters and computations involved in its estimation are complex. RMi is very sensitive in jointed rock masses and its proper estimation is heavily dependent on the correct evaluation of V_b.

(B) Support Pressure

(i) The conclusion of Goel and Jethwa (1991), that the estimated support pressures from Eq. 6.12 by Unal (1983) were unsafe under squeezing ground conditions, has been corroborated from the present study.

(ii) Pressures estimated using RMR from Eq. 6.13 as per Goel and Jethwa (1991) are slightly on lower side compared to that from N (Fig. 6.2 a). A new correlation (Eq. 6.16) with better coefficient of correlation has been proposed (Fig. 6.2 b).

(iii) Pressures estimated using RCR from Eq. 6.14 as per Goel (1994) for non-squeezing ground conditions are on lower side compared to that from N (Fig. 6.3 a). A new correlation (Eq. 6.17) with better coefficient of correlation has been proposed (Fig. 6.3 b).

(iv) Support pressures estimated from Q after corrections as per Singh et al. (1992) and N are nearly same for squeezing ground conditions (Fig. 6.4).

(v) A correlation for estimation of roof pressure (Eq. 9.9) has been developed for blocky ground using the concept proposed by Palmstrom (1995a).

(vi) A general trend with regard to pressure estimation may be given as follows:

Support Pressure (Non-Squeezing): $P_{RCR} < P_Q \leq P_N < P_{RMR} < P_{RSR}$

Support Pressure (Squeezing): $P_{RMR} < P_{RCR} < P_Q < P_N$

(C) Support System Recommended by Classification Systems

(i) RSR overestimates support requirements in non-squeezing ground conditions (Table 7.9). RMR is unsafe in both non-squeezing and squeezing ground conditions (Tables 7.9 and 7.10).

(ii) Q is inadequate in squeezing ground conditions (Table 7.10).

(iii) The capacity of support recommended by the RMi-system has been highest of all in both non-squeezing and squeezing ground conditions. In about 18 percent sections, RMi has recommended SDSC support indicating that in these sections support requirements are very high.

(iv) A general trend with regard to support capacities may be given as follows:

<u>**Support Capacity (Non-Squeezing):**</u> $C_{RMR} < C_Q < C_{RSR} << C_{RMi}$

<u>**Support Capacity (Squeezing):**</u> $C_{RMR} < C_Q < C_{RMi}$

It may be mentioned that similar site-specific correlations need to be developed for other projects.

(D) Support System Installed by the Project Authorities

(i) Support pressures after corrections as per Singh et al. (1992) are more than the support capacity rendering majority of supports actually unstable (Table 7.8 and Fig. 7.21).

(ii) In moderate and high squeezing sections, pressures are excessive and even Class VI and VI+ supports may prove unstable, as bending/twisting of ribs has been observed.

(iii) A general trend with regard to position of project support may be given as follows (Tables 7.9 and 7.10):

<u>**Support Capacity (Non-Squeezing):**</u> $C_{RMR} < P_{RCR} < \mathbf{C_{PROJECT}} < P_N = C_Q < C_{RSR} < C_{RMi}$

<u>**Support Capacity (Squeezing):**</u> $C_{RMR} < P_{RCR} < C_Q < \mathbf{C_{PROJECT}} < P_N < C_{RMi}$

(E) Rock Mass – Support Interaction Analysis

(i) In 'Good rock' (Figs. 8.1 a, b and c), all the four classification systems recommended supports, whereas through the analysis it has been found that hardly any support is needed, as closures are less than 0.1 percent of tunnel diameter.

(ii) For 'Good' and 'Fair' rocks (using RMR-system), observed and predicted deformations are in close agreement. Deformation modulii have been computed from the correlation of Verman et al. (1993). Therefore, it may be concluded that deformation modulii estimated from RMR for 'Good' and 'Fair' rocks appear to be accurate.

(iii) There are many assumptions in the rock mass – support interaction analysis. Thus, generally good agreement is not found between predicted and observed deformations in the squeezing ground conditions.

(F) Prediction of Ground Condition

(i) The present work contains 50 sections covering a length of 1133 m where squeezing ground conditions were encountered. The ratio σ_θ/q'_{cmass} in these sections has been estimated and observations regarding degree of squeezing is indicated in Table 9.4.

(ii) New empirical criteria for squeezing ground conditions based on Q and N have been obtained as shown in Figs. 9.6 a and b by plotting data for 50 sections encountering squeezing ground conditions. On the basis of Figs. 9.6 a and b, Tables 9.5 a and b have been prepared for prediction of squeezing ground condition based on Q and N respectively.

(iii) Correlations between J_r and J_a have been presented in Fig. 9.6 c where overstress conditions existed in the tunnel. On the basis of Fig. 9.6 c, Table 9.5 c has been prepared for prediction of ground condition based on J_r and J_a. Moderate slabbing with noise is likely to occur if

J_r/J_a is equal to or more than 0.50 otherwise squeezing may take place, subject to the above-mentioned condition.

In overall analysis performed in the present work, it has been found that revised Q-system is best due to its realistic approach.

10.3 SCOPE OF FURTHER RESEARCH

In view of the limitations of the present study, the following areas have been identified for further research:

- Various classification systems should be verified at other projects also and most useful classification system should be searched out. This is specially true for new classification systems such as Q_c, RMi, N, RCR etc.
- The support pressure should be observed carefully in various tunnels so that realistic correlations may be obtained between observed support pressures and various classification systems.
- Construction engineers are now building very deep tunnels with overburden of 2000 m. It is necessary that realistic criteria for rock burst and squeezing conditions be developed on the basis of properly studied case histories.
- The polyaxial strength criterion is promising. But it needs to be validated at various project sites extensively.
- There are many assumptions in the analytical approaches. Therefore, there is need of characterization of rock masses using classification approach.
- The behaviour of rock mass specially in rock burst and squeezing conditions is highly time-dependent. The object of the classification approach should also be to predict the

variation of support pressure and tunnel closures with time for various tunnelling conditions.

- Stability of support systems in rock tunnels in high geothermal gradients should also be studied. New type of shotcrete and rock bolts may have to be evolved for stabilizing unstable rocks in high geothermal areas.

REFERENCES

1. Abad, J., Caleda, B., Chacon, E., Gutierrez, V. and Hidlgo, E. (1984): Application of Geomechanical Classification to Predict the Convergence of Coal Mine Galleries and to Design their Supports, 5th Int. Congress on Rock Mech., Melbourne, (E), pp.15-19.

2. Aydan, O., Akagi, T. and Kawamoto, T. (1993): The Squeezing Potential of Rocks around Tunnels; Theory and Prediction, Jr. Rock Mechanics and Rock Engineering, Springer-Verlag, No.26, pp.137-163.

3. Barton, N. (1998): Quantitative Description of Rock Masses for the Design of NMT Reinforcement, Proc. Int. Conf. on Hydro Power Development in Himalayas, April 20-22, Shimla, India, ed. Prof. Vishnu D. Choubey, pp.379-400.

4. Barton, N. (1999): General Report concerning some 20th Century Lessons and 21st Century Challenges in Applied Rock Mechanics, Safety and Control of the Environment, Gen. Rept., Theme 1, Proc. 9th ISRM Cong., Paris, 3, 21, Balkema, Rotterdam.

5. Barton, Nick (2001): Water and stress are fundamental to rock mass characterization and classification, Letter to the Editor, ISRM News Journal, November, pp.4-5.

6. Barton, N. (2002a): Some New Q-Value Correlations to assist in Site Characterisation and Tunnel Design, Int. J. Rock Mech. & Min. Sci., Pergamon, Vol.39, No.2, February, pp.185-216.

7. Barton, N. (2002b): Discussion "Deformation moduli and rock mass characterization", Tunnelling and Underground Space Technology, Pergamon, Vol.17, No.2, April, pp.221-222.

8. Barton, N., Lien, R. and Lunde, J. (1974): Engineering Classification of Rock Masses for the Design of Tunnel Support, Jr. Rock Mechanics and Rock Engineering, Springer-Verlag, 6, pp.189-236.

9. Barton, N., Lien, R. and Lunde, J. (1975): Estimation of Support Requirements for Underground Excavations, Proc. 16th Symp. Rock Mech., University of Minnesota, Minneapolis, USA, pp.163-177.

10. Barton, N., Loset, F., Lien, R. and Lunde, J. (1980): Application of Q–System in Design Decisions concerning Dimensions and Appropriate Support for Underground Installations, Jr. Subsurface Space, Pergamon, pp.553-561.

11. Benson, R.P., Colon, R.J., Merritt, A.H., Joli-Coeur, P. and Deere, D.U. (1971): Rock Mechanics at Churchill Falls, American Society of Civil Engineers, Symposium on Underground Rock Chambers, Phoenix, Arizona, Proceedings, pp.407-486.

12. Bhasin, R. and Grimstad, E. (1996): The Use of Stress–Strength Relationships in the Assessment of Tunnel stability, Proc. Recent Advance in Tunnelling Technology, CSMRS, New Delhi, pp.183-196.

13. Bieniawski, Z.T. (1973): Engineering Classification of Jointed Rock Masses, Trans. S. Afr. Inst. Civ. Eng. 15, pp.335-344.

14. Bieniawski, Z.T. (1975): Case Studies: Prediction of Rock Mass Behaviour by the Geomechanics Classification, Proc. 2nd Aust. N.Z. Conf. on Geomech., Brisbane, pp.36-41.

15. Bieniawski, Z.T. (1976): Rock Mass Classification in Rock Engineering, Proc. Symp. Exploration for Rock Engineering, ed. Z.T. Bieniawski, A.A.Balkema, Rotterdam, Vol.1, pp.97-106.

16. Bieniawski, Z.T. (1978): Determining Rock Mass Deformability: Experience from Case Histories, Intl. J. Rock Mech. and Min. Sci., Pergamon, 15, pp.237-247.

17. Bieniawski, Z.T. (1979): The Geomechanics Classification in Rock Engineering Applications, Proc. 4th International Congress Rock Mech., ISRM, Montreux, Vol.2, pp.41-48.

18. Bieniawski, Z.T. (1984): Rock Mechanics Design in Mining and Tunneling, Balkema, Rotterdam, 272 p.

19. Bieniawski, Z.T. (1988): Rock Mass Classification as a Design Aid in Tunnelling, Jr. Tunnels and Tunnelling, July, pp.19-22.

20. Bieniawski, Z.T. (1989): Engineering Rock Mass Classifications, John Wiley and Sons, 271 p.

21. Bieniawski, Z.T. (1993): Classification of Rock Masses for Engineering: The RMR System and Future Trends, 'Comprehensive Rock Engineering', Vol.3, Ch.22, Pergamon, pp.553-573.

22. Braun, W.M. (1980): Application of the NATM in Deep Tunnels and Difficult Rock Formations, Jr. Tunnels and Tunnelling, Vol.12, No.2, March, pp.17-20.

23. Brekke, T.L. (1968): Blocky and Seamy Rock in Tunnelling, Bull. Assoc. Engineering Geology, Vol.5, No.1, pp.1-12.

24. Brown, E.T. (1981): Putting the NATM into Perspective, Jr. Tunnels and Tunnelling, pp.13-17.

25. Brown, Edwin T., Bray, John W., Ladanyi, Branko and Hoek, Evert (1983): Ground Response Curves for Rock Tunnels, ASCE, Geotech. Engineering, Vol.109, No.1, pp.15-31.

26. Butler, A.G. and Franklin, J.A. (1990): Classex: an Expert System for Rock Mass Classification, Proc. Int. Symp. Rock Mech., Mbabane, Swaziland, pp 73-80.

27. Cameron–Clarke, I.S. and Budavari, S. (1981): Correlation of Rock Mass Classification Parameters Obtained from Borecore and In-Situ Observations, Jr. Engineering Geology, Elsevier Science, Vol. 17, pp.19-53.

28. CEA (2001): Preliminary Ranking Study of Hydroelectric Schemes, Volume-I, General Report, Government of India, Ministry of Power, October, 35 p.

29. Cecil, O.S. (1970): Correlations of Rock Bolts–Shotcrete Support and Rock Quality Parameters in Scandinavian Tunnels, Ph. D Thesis, University of Illinois, Urbana, 414 p.

30. Chaddha, A.K., Chaubey, J., Chauhan, K.S., Pandey, A.K., Kaushal, V., Acharya, A.K., Chauhan, R.K. and Pundhir, G.S. (1999): Geology of Nathpa – Jhakri Hydroelectric Project, Seminar on Rock Mechanics and Tunnelling Technology, 10 December, New Delhi, pp.13-34.

31. Chen, C.N. and Kuo, G.C. (1997): Rock Mass Classification and Guidelines for Tunnel Convergence, Journal of the Chinese Institute of Civil and Hydraulic Engineering, Vol.9, No.3, pp.359-367.

32. Choubey, V.D. (1986): What are the Most Useful Classification Systems for Rock Masses? Indian Geotechnical Conf., Vol. I, IIT, New Delhi, pp.17-23.

33. Choubey, Vishnu D. (1998): Potential of Rock-Mass Classification for Design of Tunnel Supports–Hydro Electric Projects in Himalayas, Proc. Int. Conf. on Hydro Power Development in Himalayas, April 20-22, Shimla, India, ed. Prof. Vishnu D. Choubey, pp.305-336.

34. Choubey, V.D., Sharma, V.M., Dhawan, A.K. and Dhawan, G. (1995): Recent Experiences of the Application of the Q-System in Tunnel Support Designs in Himalaya, Conf. on Design and Construction of Underground Structures, New Delhi, India, pp.83-111.

35. Cording, E.J. and Deere, D.U. (1972): Rock Tunnel Supports and Field Measurements, Proc. Rapid Excav. Tunneling Conf., AIME, NY, Vol.1, pp.601-622.

36. Corranza-Torres, C. and Fairhurst, C. (2000): Application of the Convergence-Confinement Method of Tunnel Design to Rock Masses that Satisfy the Hoek-Brown Failure Criterion, Tunnelling and Underground Space Technology, Pergamon, Vol.15, No.2, pp.187-213.

37. CSMRS (1994a): Report on In-Situ Stress Measurement by Overcoring using CSIRO HI Cell in the Power House at Nathpa Jhakri H.E. Project (HP), No. RM-II/94/2, 37p.

38. CSMRS (1994b): Report on Determination of In-Situ Stress by Surface Hydraulic Fracturing Test at Desilting Chamber, Nathpa Jhakri H.E. Project (HP), No. RM-II/94-3.

39. CSMRS (1996a): Report on Laboratory Testing of Rock Cores from Nathpa HRT, Nathpa Jhakri H.E. Project (HP), No. RM-I/4/1996, August.

40. CSMRS (1996b): Report on Hydrofacturing Test using Minifrac System Conducted in Steel Liner Portion of HRT at Manglad, Nathpa Jhakri H.E. Project (HP), No. CSMRS/RM-IV, October.

41. CSMRS (1996c): Report on Laboratory Testing of Rock Cores from Sholding HRT, Nathpa Jhakri H.E. Project (HP), No. RM-I/9/1996, December.

42. CSMRS (1996d): Report on Laboratory Testing of Rock Cores from Wadhal HRT, Nathpa Jhakri H.E. Project (HP), No. RM-I/10/1996, December.

43. CSMRS (1996e): Report on Laboratory Testing of Rock Cores from Manglad HRT, Nathpa Jhakri H.E. Project (HP), No. RM-I/10/1996, December.

44. CSMRS (1997a): Report on Hydrofacturing Tests Conducted at Rattanpur Downstream, Nathpa Jhakri H.E. Project (HP), No. CSMRS/RM-I, July.

45. CSMRS (1997b): Report on Hydrofacturing Test using Minifrac System Conducted in Sixth Face of HRT at Nathpa Jhakri H.E. Project (HP), No. CSMRS/RM-I/II, September.

46. Daemen, J.J.K. (1975): Tunnel Support Loading Caused by Rock Failure, Ph.D. Thesis, University of Minnesota, Minneapolis, USA.

47. Daemen, J.J.K. and Fairhurst, C. (1970): Influence of Failed Rock Properties on Tunnel Stability, Twelfth Symposium on Rock Mechanics, Rolla, Missouri, pp.855-875.

48. Deere, D.U. (1968): "Geological Considerations" Rock Mechanics in Engineering Practice, ed. R.G.Stagg and D.C.Zienkiewicz, Wiley, New York, pp.1-20.

49. Deere, D.U. and Miller, R.P. (1966): Engineering Classification and Index Properties for Intact Rock, Techn. Report No. AFNL-TR-65-116, Air Force Weapons Laboratory, New Mexico.

50. Deere, D.U., Hendron, A.J., Patton, F.D. and Cording, E.J. (1967): Design of Surface and Near Surface Construction in Rock, Proc. 8th U.S. Symp. Rock Mech., AIME, New York, pp.237-302.

51. Deere, D.U., Peck, R.B., Monsees, J.E. and Schmidt, B. (1969): Design of Tunnel Liner and Support System, U.S. Department of Transportation, Highway Research Record No.339, Washington, D.C.

52. Deere, D.U., Peck, R.B., Parker, H., Monsees, J.E. and Schmidt, B. (1970): Design of Tunnel Support Systems, High Res. Rec. No.339, pp.26-33.

53. Dhar, Y.R. and Choubey, V.D. (1986): A Critical Geotechnical Evaluation of Himalayan Rock Masses with Reference to Rock Tunnelling Projects, 5th IAEG Congress, Buenos Aires, pp.249-258.

54. Dhawan, A.K. (1986): What are the Most Useful Classification Systems for Rock Masses? Indian Geotechnical Conf., Vol.I, IIT, New Delhi, pp.11-16.

55. Dubey, A.K. (1979): Geomechanical Evaluation of Tunnel Stability under Failing Rock Conditions in a Himalyan Tunnel, Ph.D Thesis, University of Roorkee, Roorkee, India.

56. Dube, A.K., Jethwa, J.L. and Singh, B. (1982): Evaluation of Radius of Broken Zone around a Tunnel in Squeezing Rock Conditions, Proc. IV Cong., IAEG, Vol.IV, New Delhi, pp.IV.223-230.

57. Dubey, A.K., Jethwa, J.L., Singh, B., Singh, Bhawani and Mithal, R.S. (1982): Geo-Engineering Evaluation of Problems of a Large Cavity for Tehri Project (India), ISRM Symp., Rock Mech: Caverns and Pressure Shafts, ed.-W. Wittke, Vol.I, A.A. Balkema, Rotterdam, pp.239-244.

58. Einstein, H.H., Thompson, D.E., Azzouz, A.S., O'Reilly, K.P., Schultz, M.S. and Ordun, S. (1983): Comparison of Five Empirical Tunnel Classification Methods – Accuracy, Effect of Subjectivity and Available Information, 5th Int. Cong. Rock Mech., Vol.C, Melbourne, Balkema, Rotterdam, pp.C303-C313.

59. Enever, J.R. and Walton, R.J. (1987): Rock Stress Measurement Techniques, CSIRO Division of Geomechanics, Melbourne, Australia.

60. Fairhurst, C. (1993): Analysis and Design in Rock Mech.-The General Context, 'Comprehensive Rock Engineering', Vol.1, Ch.1, Pergamon, pp.1-29.

61. Franklin, J.A. (1993): Empirical Design and Rock Mass Characterization, 'Comprehensive Rock Engineering', Vol.2, Ch.32, Pergamon, pp.795-806.

62. Goel, R.K. (1994): Correlations for Predicting Support Pressures and Closures in Tunnels, Ph.D. Thesis, Nagpur University, Nagpur, India, 308p.

63. Goel, R.K. (2001): Status of Tunnelling and Underground Construction Activities and Technologies in India, Tunnelling and Underground Space Technology, Pergamon, Vol.16, No.2, April, pp.63-75.

64. Goel, R.K. (2001): Rock Mass Classification, Appreciation Programme for Middle and Junior Executives on Recent Developments in Rock Mechanics & Tunnelling Technology, Organized by ISRMTT, Sept.6-7, New Delhi.

65. Goel, R.K. and Jethwa, J.L. (1991): Prediction of Support Pressure using RMR, Proc. Indian Geotech. Conf., Surat, India, pp.203-205.

66. Goel, R.K., Jethwa, J.L. and Dhar, B.B. (1996): Effect of Tunnel Size on Support Pressure, Tech. Note, Intl. J. Rock Mech. and Min. Sci, Pergamon, Vol.33, No.7, pp.749-755.

67. Goel, R.K., Jethwa, J.L. and Dube, A.K. (1998): A Comprehensive Approach for Tunnelling through Weak and Jointed Rock Masses, Proc. Int. Conf. on Hydro Power Development in Himalayas, April 20-22, Shimla, India, ed. Prof. Vishnu D. Choubey, pp.356-366.

68. Goel, R.K., Jethwa, J.L. and Paithankar, A.G. (1995a): Indian Experiences with Q and RMR Systems, Jr. Tunnelling and U.G. Space Technology, Pergamon, Vol.10, No.1, pp.97-109.

69. Goel, R.K., Jethwa, J.L. and Paithankar, A.G. (1995b): An Empirical Approach for Predicting Ground Condition for Tunnelling and its Practical Benefits, Proc. 35th U.S. Symp. Rock Mech., Univ. of Nevada, Reno, U.S.A. pp.431-435.

70. Goel, R.K., Jethwa, J.L. and Paithankar, A.G. (1995c): Correlation between Barton's Q and RMR–A New Approach, Technical Note, Intl. Jr. Rock Mech. and Min. Sci., Pergamon, Vol.33, No.2, pp.179-181.

71. Goel, R.K., Jethwa, J.L. and Singh, B. (1986): What are the Most Useful Classification Systems for Rock Masses? Indian Geotechnical Conf., Vol. II, IIT, New Delhi, pp.43-46.

72. Goel, R.K., Singh, Bhawani and Singh, Suneel (1999): Time-Dependent Modulus of Deformation in Tunnels, Jr. Rock Mech. and Tunnelling Technology, Pergamon India, Vol.5, No.5, June, pp.1-12.

73. Grimstad, E. and Barton, N. (1993): Updating of the Q-System for NMT, Proc. Intl. Symp. on Sprayed Conc.-Modern use of Wet Mix Sprayed Concrete for Underground Support, Fagernes, (eds:Kompen, Opsahll and Berg.), Norwegian Concrete Association, Oslo, pp.1-21.

74. Grimstad, E. and Barton, N. (1995): Rock Mass Classification and the use of NMT in India, Proc. Conf. on Design and Construction of Underground Structures, Feb.23-25, New Delhi, pp.63-74.

75. Grimstad, E. and Bhasin, R. (1996): Stress Strength Relationships and Stability in Hard Rock, Proc. Conf. on Recent Advances in Tunnelling Technology, New Delhi, Vol.1, pp.3-8.

76. GSI (1988): A Comprehensive Geotechnical Report on the Studies carried out for Nathpa Jhakri Hydroelectric Project, Shimla and Kinnaur Districts, Himachal Pradesh by Z. Ashraf and A.K. Chowdhary, May.

77. GSI (1991): Comprehensive Geotechnical Report No.2 on the Studies carried out for Nathpa Jhakri Hydroelectric Project, Shimla and Kinnaur Districts, Himachal Pradesh by S.P. Jalote, A.K. Chowdhary, P.M. Jalote, Ashok Kumar and S.V. Kumar, March, 1988 to January, 1991.

78. Gupta, L.N., Kumar, Ravindra and Choudhri, Naveen (1996): An Introduction to the Geology of the Northwestern Himalaya, Himachal Pradesh, India, Field Excursion Guide, Himalayan

Field Geology Programme for Institute of Mineralogy, Hannover University, Hannover, Germany, Sept.27–Oct.10, 30p.

79. Haimson, B.C. (1984): Pre-Excavation in In-Situ Stress Measurements in the Design of Large Underground Openings, ISRM, pp.183-190.

80. Hoek, E. (1983): Strength of Jointed Rock Masses, The Rankine Lecture 1983, Jr. Geotechnique, 33, No.3, pp.187-223.

81. Hoek, E. (1994): Strength of Rock and Rock Masses, ISRM News Journal, Vol.2, No.2, pp.4-16.

82. Hoek, E. and Brown, E.T. (1980a): Underground Excavations in Rock, The Institution of Mining and Metallurgy, London, 527p.

83. Hoek, E. and Brown, E.T. (1980b): Empirical Strength Criterion for Rock Masses, ASCE, Geotech. Engineering, Vol.106, No.GT9, pp.1013-1035.

84. Hoek, E. and Brown, E.T. (1988): The Hoek–Brown Failure Criteria – A 1988 Update, 15th Canadian Rock Mech. Sym., pp.31-38.

85. Hoek, E. and Brown, E.T. (1997): Practical Estimation of Rock Mass Strength, Intl. J. Rock Mech. and Min. Sci., Pergamon, Vol.34, No.8, pp.1165-1186.

86. Hoek, E., Wood, D. and Shah, S. (1992): A Modified Hoek–Brown Failure Criteria for Jointed Rock Masses, Intl. Conf. Eurock'92, London, pp.209-214.

87. Hudson, John A. (1993): Rock Properties, Testing Methods and Site Characterization, 'Comprehensive Rock Engineering', Vol.3, Ch.1, Pergamon, pp.1-39.

88. IS 9143 : 1979: Method for Determination of Unconfined Compressive Strength of Rock Materials, Bureau of Indian Standards, New Delhi.

89. IS 13030 : 1991: Method of Test for Laboratory Determination of Water Content, Porosity, Density and Related Properties of Rock material, Bureau of Indian Standards, New Delhi.

90. IS 13047 : 1991: Method for Determination of Strength of Rock Materials in Triaxial Compression, Bureau of Indian Standards, New Delhi.

91. ISRM (1978): Suggested Methods for the Quantitative Description of Discontinuities in Rock Masses, Vol.15, pp.319-368.

92. ISRM (1979): Suggested Methods for Determining In-Situ Deformability of Rock, Int. J. Rock Mech. Min. Sci. & Geomech. Abstracts, Vol.16, No.3, pp.195-214.

93. Jethwa, J.L. (1981): Evaluation of Rock Pressure under Squeezing Rock Conditions for Tunnels in Himalayas, Ph.D. Thesis, University of Roorkee, Roorkee, India.

94. Jethwa, J.L. and Dhar, B.B. (1996): Tunnelling under Squeezing Ground Condition, Proc. Recent Advances in Tunnelling Technology, New Delhi, pp.209-214.

95. Jethwa, J.L., Dubey, A.K. and Singh, B. (1978): Rock Mechanics Instrumentation for Tunnels in India, Sept., pp.63-66.

96. Jethwa, J.L., Dube, A.K., Singh, B. and Singh, Bhawani (1981): Rock Load Estimation for Tunnel in Squeezing Ground Conditions, RETC, San Francisco, May, 3-7, pp.766-783.

97. Jethwa, J.L., Dubey, A.K., Singh, B., Singh, Bhawani and Mithal, R.S. (1982a): Evaluation of Methods for Tunnel Support Design in Squeezing Rock Conditions, Proc. 4th Int. Cong. Int. Ass. Eng. Geol, New Delhi, Vol.V, pp.V.121-V134.

98. Jethwa, J.L., Dubey, A.K., Singh, B., Singh, Bhawani and Mithal, R.S. (1982b): Evaluation of Classification System for Tunnels in Non-squeezing Ground Conditions, ISRM Symp., Rock Mech.: Caverns and Pressure Shafts, ed.-W.Wittke, A.A. Balkema, Rotterdam, pp.607-612.

99. Jethwa, J.L., Singh, B. and Singh, B. (1984): Estimation of Ultimate Rock Pressure for Tunnel Lining under Squeezing Rock Conditions–A New Approach, Design and Performance of Underground Excavations, ISRM/BGS, Cambridge, pp.231-238.

100. Jethwa, J.L., Singh, B., Singh, Bhawani and Mithal, R.S. (1980): Influence of Geology on Tunnelling Conditions and Deformational Behaviour of Supports in the Faulted Zones–A Case History of Chhibro–Khodri Tunnel in India, Jr. Engineering Geology, Vol.16, No.¾, pp.291-319.

101. John, K.W. (1962): An Approach to Rock Mechanics, Journal Soil Mech. Found. Div., ASCE, Vol.88, pp.1-30.

102. Kaisar, P.K., Mackay, C. and Gale, A.D. (1986): Evaluation of Rock Classification at B.C. Rail Tumbler Ridge Tunnels, Jr. Rock Mechanics and Rock Engineering, Springer-Verlag, 19, pp.205-234.

103. Kirsten, H.A.D. (1983): The Combined Q/NATM System–the Design and Specification of Primary Tunnel Support, Jr. South African Tunnelling, 6(1).

104. Krishnamurthy, K.S. (1986): What are the Most Useful Classification Systems for Rock Masses? Indian Geotechnical Conf., Vol.I, IIT, New Delhi, pp.24-31.

105. Ladanyi, B. (1974): Use of the Long-Term Strength Concept in the Determination of Ground Pressure on Tunnel Linings, Proc. 3rd Congr., Intl. Soc. Rock Mech., Denver, Vol. 2B, pp.1150-1156.

106. Lauffer, H. (1958): "Gebirgsklassifizierung fur den Stollenbau", Geology Bauwesen 74, pp.46-51.

107. Lauffer, H. (1988): Zur Gebirgsklassifizierung bei Frasvortrieben, Felsbau, 6(3), pp.137-149.

108. Lombardi, G. (1970): Influences of Rock Characteristics on the Stability of Rock Cavities, Jr. Tunnels and Tunnelling, Vol.2, No.2, pp.104-109.

109. Lombardi, G. (1977): Long Term Measurements in Underground Openings and their Interpretations with Special Consideration to the Rheological Behaviour of Rock, Field Measurements in Rock Mechanics, K. Kovari, ed., Vol.2, A.A. Balkema, Rotterdam, Holland, pp.835-858.

110. Mande, O.D. and Vohra, D.R. (1998): Nathpa Jhakri Project–A Landmark in Hydropower Development in Himalayan Ranges, Proc. Int. Conf. on Hydro Power Development in Himalayas, April 20-22, Shimla, India, ed. Prof. Vishnu D. Choubey, pp.239-247.

111. Mehrotra, V.K. (1992): Estimation of Engineering Parameters of Rock Mass, Ph.D. Thesis, University of Roorkee, Roorkee, India, 267p.

112. Mehrotra, V.K., Singh, Bhawani and Mitra, Subhash (1998): Influence of Saturation on Modulus of Deformation of Himalayan Rock Masses, Jr. Rock Mech. and Tunnelling Technology, Pergamon India, Vol.4, No.2, Aug., pp.125-137.

113. Merritt, A.H. (1972): Geologic Prediction for Underground Excavations, Proc. Rapid Excav. Tunneling Conf., AIME, New York, pp.115-132.

114. Mitri, H.S., Edrissi, R. and Henning, J. (1994): Finite Element Modelling of Cable Bolted Slopes in Hard Rock Underground Mines, Presented at the SME Annual Meeting, Albuquerque, pp.14-17.

115. Moreno Tallon, E. (1980): Application de Las Classificaciones Geomechnicas a Los Tunele de Parjares, II Cursode Sostenimientos Activosen Galeriasy Tunnels Madrid: Foundation Gomez – Parto [referred in Kaiser et al. (1986)].

116. Muir Wood, A.M. (1979): Ground Behaviour and Support for Mining and Tunnelling, Jr. Tunnels and Tunnelling, Part I (May 1979), pp.43-48 and Part II (June 1979), pp.47-51.

117. Muller, L. (1978): Removing Misconceptions on the NATM, Jr. Tunnels and Tunnelling, Vol.10, No.8, October, pp.29-32.

118. Nicholson, G.A. and Bieniawski, Z.T. (1990): A Non-Linear Deformation Modulus based on Rock Mass Classification, Int. Journal Min. and Geol. Engineering, (8) pp.181-202.

119. Pacher, F., Rabcewicz, L. and Golser, J. (1974): "Zum der seitigen Stand der Gebirgsklassifizierung in Stollen-und Tunnelbau" Proc. XXII Geomech. Colloq., Salzburg, pp.51-58.

120. Palmstrom, A. (1982): The Volumetric Joint Count–A Useful and Simple Measure of the Degree of Rock Mass Jointing, 4th Int. Cong., IAEG, New Delhi, pp.V.221-V.228.

121. Palmstrom, A. (1995a): RMi–A Rock Mass Characterization System for Rock Engineering Purposes, Ph.D. Thesis, University of Oslo, Norway, 400p.

122. Palmstrom, A. (1995b): Characterizing the Strengths of Rock Masses for use in Deign of Underground Structures, Intl. Conf. on Design and Construction of Underground Structures, New Delhi, pp.43-52.

123. Palmstrom, A. (1995c): Characterizing Rock Burst and Squeezing by the Rock Mass index, Intl. Conf. on Design and Construction of Underground Struc., New Delhi, pp.53-62.

124. Palmstrom, A. (1995d): RMi–A System of Characterizing Rock Mass Strength for use in Rock Engineering, Jr. Rock Mech. and Tunnelling Technology, Pergamon India, Vol.1, No.2, pp.69-108.

125. Palmstrom, A. (1996a): Characterization of Rock Masses by the RMi for use in Practical Rock Engineering, Part 1: The Development of the Rock Mass index (RMi), Jr. Tunnelling and U.G. Space Technology, Pergamon, Vol.11, No.2, pp.175-188.

126. Palmstrom, A. (1996b): Characterization of Rock Masses by the RMi for use in Practical Rock Engineering, Part 2: Some Practical Applications of the Rock Mass index (RMi), Jr. Tunnelling and U.G. Space Technology, Pergamon, Vol.11, No.3, pp.287-303.

127. Palmstrom, A. (1996c): The Rock Mass index (RMi) Applied in Rock Mechanics and Rock Engineering, Jr. Rock Mech. and Tunnelling Technology, Pergamon India, Vol.2, No.1, pp.1-40.

128. Palmstrom, A. (2000a): Recent Developments in Rock Support Estimates by the RMi, Jr. Rock Mech. and Tunnelling Technology, Pergamon India, Vol.6, No.1, pp.1-24.

129. Palmstrom, A. (2000b): Personal Communication with the Author, dated 5th October 2000.

130. Palmstrom, Arild, Milne, Doug and Peck, Warren (2000): Discussion during Workshop on "The Reliability of Rock Mass Classification used in Underground Excavation and Support Design", Nov. 18, The University of Melbourne.

131. Pande, G.N. (1997): SQCC Lecture on Application of the Homogenisation Techniques in Soil Mechanics and Structures, Sept. 26, University of Roorkee, Roorkee, India.

132. Rabcewicz, L.V. (1964-65): The New Austrian Tunneling Method, Jr. Water Power, Part one, Part two, Part three, Nov. 64, Dec, 64, Jan 65, pp.453-457, 511-515,19-25.

133. Ramamurthy, T., Rao, G.V. and Rao, K.S. (1986): What are the Most Useful Classification Systems for Rock Masses? Indian Geotechnical Conf., Vol. II, IIT, New Delhi, pp.51-53.

134. Ram, B. and Jethwa, J.L. (1986): What are the Most Useful Classification Systems for Rock Masses? Indian Geotechnical Conf., Vol. II, IIT, New Delhi, pp.47-49.

135. Rose, D. (1982): Revising Terzaghi's Tunnel Rock Load Coefficients, Proc. 23rd U.S. Symposium Rock Mech., AIME, New York, pp.953-960.

136. Russenes, B.F. (1974): Analysis of Rock Spalling for Tunnels in Steep Valley Sides (in Norwegian), M.Sc. Thesis, Norwegian Institute of Technology, Dept. of Geology, 247p.

137. Ruteledge, J.C. and Preston, R.L. (1978): New Zealand Experience with Engineering Classifications of Rock for the Prediction of Tunnel Support, Proc. Int. Tunnelling Symp., Tokyo, pp.23-29.

138. Saini, G.S., Dube, A.K. and Singh, B. (1989): Severe Tunnelling Problems in Young Himalayan Rocks for Deep Underground Opening, Rock at Great Depth, Maury and Fourmaintraux (eds.), Balkema, Rotterdam, ISBN 9061919754, pp.677-685.

139. Sakurai, S (1993): Back Analysis in Rock Engineering, ISRM News Journal, Vol.2, No.2, pp.4-16.

140. Samadhiya, N.K. (1998): Influence of Anisotropy and Shear Zones on Stability of Caverns, Ph.D. Thesis, University of Roorkee, Roorkee, India, 334p.

141. Selmer-Olsen, R. (1964): Geology and Engineering Geology (in Norwegian), Troudheim, Norway: Tapir.

142. Serafim, J.L. and Pereira, J.P. (1983): Considerations of the Geomechanics Classifications of Bieniawski, Int. Sym. Eng. Geol. U.G. Contr. LNEC, Lisbon, Vol.1, pp.II.33-II.42.

143. Sharma, V.M. (1990): Rock Support Interaction, Workshop on Rock Mechanics, Central Board of Irrigation and Power, New Delhi, pp.77-84.

144. Sharma, V.M. (1995): Support Measures, Conf. on Design and Construction of Underground Structures, New Delhi, India, pp.431-445.

145. Sharma, V.M., Ramamurthy, T. and Sudhindra, C. (1987): Prediction of Closures and Rock Loads for Tunnels in Squeezing Grounds, Intl. Cong. on Rock Mech., Vol.2, Canada, pp.1227-1231.

146. Sheorey, P.R. (1985): Support Pressure Estimation in Failed Rock Conditions, Jr. Engineering Geology, 22, pp.127-140.

147. Singh, Bhawani (1988): Recent Experiences on Application of Barton's Q-System in India.

148. Singh, Bhawani and Goel, R.K. (1999): Rock Mass Classification: A Practical Approach in Civil Engineering, Elsevier, 267p.

149. Singh, Bhawani, Goel, R.K., Jethwa, J.L. and Dube, A.K. (1997): Support Pressure Assessment in Arched Underground Openings through Poor Rock Masses, Jr. Engineering Geology, Vol.48, pp.59-81.

150. Singh, Bhawani, Goel, R.K., Mehrotra, V.K., Garg, S.K. and Allu, M.R. (1998): Effect of Intermediate Principal Stress on Strength of Anisotropic Rock Mass, Jr. Tunnelling and U.G. Space Technology, Pergamon, Vol.13, No.1, pp.71-79.

151. Singh, Bhawani, Jethwa, J.L. and Dube, A.K. (1995): A Classification System for Support Pressure in Tunnels and Caverns, Jr. Rock Mech. and Tunnelling Technology, Pergamon India, Vol.1, No.l, India, pp.13-24.

152. Singh, Bhawani, Jethwa, J.L., Dube, A.K. and Singh, B. (1992): Correlation between observed Support Pressure and Rock Mass Quality, Jr. Tunnelling and U.G. Space Technology, Pergamon, Vol.7, No.1, pp.59-74.

153. Singh, Bhawani, Viladkar, M.N., Samadhiya, N.K. and Mehrotra, V.K. (1997): Rock Mass Strength Parameters Mobilized in Tunnels, Jr. Tunnelling and U.G. Space Technology, Pergamon, Vol.12, No.1, pp.47-54.

154. Singh, Bhawani, Viladkar, M.N., Samadhiya, N.K. and Sandeep (1995a): Design Philosophy of Support Systems for Underground Structures, Conf. on Design and Construction of Underground Structures, New Delhi, India, pp.225-242.

155. Singh, Bhawani, Viladkar, M.N., Samadhiya, N.K. and Sandeep (1995b): A Semi - Empirical Method of the Design of Support Systems in Underground Openings, Jr. Tunnelling and U.G. Space Technology, Pergamon, 10, Vol.3, pp.375-383.

156. Singh, Pratap and Mahajan, J.L. (1982): Some Geotechnical Considerations on Tunnelling in Himalayas, Proc. IV Cong., IAEG, Vol.IV, New Delhi, pp.IV.65-IV.74.

157. Singh, Rajbal (1996): Q–System and Deformability of Rock Mass, Proceedings of the Conf. on Recent Advances in Tunnelling Technology, New Delhi, Vol.-I, pp.57-68.

158. Singh, Rajbal and Palmstrom, Arild (2002): Discussion "Response to Dr Nick Barton's comments to our paper on deformation modulus of rock masses in Vol.16, 2001, pp.115-131", Tunnelling and Underground Space Technology, Pergamon, Vol.17, No.2, pp.223-225.

159. Singh, R., Prasad Rao, P.D., Sharma, R.N. and Verma, P.K. (1998): Challenges in Excavating the Headrace for Nathpa Jhakri, Jr. Hydropower & Dams, Issue One, pp.36-39.

160. Singh, Suneel (1997): Time Dependent Deformation Modulus of Rocks in Tunnels, M.E. Thesis, Department of Civil Engineering, University of Roorkee, Roorkee, India, 65p.

161. Sinha, R.S. (1988): Discussion of the RSR Model, Proc. Symp. Rock Class. Engineering Purp., ASTM Special Technical Publication 984, Philadelphia, 50p.

162. Sirkek, J.K., Negi, B.S. and Thakur, D.D. (1999): Tunnelling with Special Drilling Rig Through Bad Rock, Seminar on Rock Mechanics and Tunnelling Technology, 10 December, New Delhi, pp.171-181.

163. Speers, C.R. (1992): Support for Tunnels Subjected to Changing Rock Loads: A Comparison of Design Methods, Jr. Tunnelling and U.G. Space Technology, Pergamon, Vol.7, No.1, pp.25-32.

164. Stini, I. (1950): "Tunnulbaugeologie", Springer Verlaag, Vienna, Austria, pp.1-336.

165. Tanimoto, C. (1980): Tunnelling in Rock with Rock Bolts and Shotcrete, Ph.D. Thesis, Kyoto University, Japan, 243p.

166. Terzaghi, K. (1946): "Rock Defects and Loads on Tunnel Supports", Rock Tunneling with Steel Supports, ed. R.V. Proctor and T. White, Commercial Shearing Co, Youngstown, Ohio, USA, pp. 15-99.

167. Unal, E. (1983): Design Guidelines and Roof Control Standards for Coal Mine Roofs, Ph.D. Thesis, Pennsylvania State University, University Park, 355p.

168. Verman, M.K. (1993): Rock Mass–Tunnel Support Interaction Analysis, Ph.D. Thesis, University of Roorkee, Roorkee, India.

169. Verman, M., Singh, B., Jethwa, J.L., Dube, A.K. and Singh, B. (1997): Estimation of mobilized Cohesion around Underground Openings, Intl. J. Rock Mech. and Min. Sci, Pergamon, Vol.34, No.5, pp.851-858.

170. Verman, M, Singh, B., Viladkar, M.N. and Jethwa, J.L. (1993): Effect of Tunnel Depth on Modulus of Deformation of Rock Mass, Jr. Rock Mechanics and Rock Engineering, Springer-Verlag, 30(3), pp.121-127.

171. Ward, W.H. (1978): Eighteenth Rankine Lecture: Ground Supports for Tunnels in Weak Rocks, Jr. Geotechnique, Vol.28, No.2, June, pp.133-170.

172. Ward, W.H., Tedd, P. and Berry, N.S.M. (1983): The Kielder Experimental Tunnel: Final Results, Jr. Geotechnique, 33, pp.275-291.

173. Wickham, G.E. (1974): Ground Support Prediction Model–RSR Concept, Proc. Rapid Excav. Tunneling Conf., AIME, NY, pp.691-707.

174. Wickham, G.E., Tiedemann, H.R. and Skinner, E.H. (1972): Support Determination based on Geological Predictions, Proc. Rapid Excav. Tunnelling Conf., AIME, NY, pp.43-64.

175. Wood, D. (1991): Estimating Hoek-Brown Rock Mass Strength Parameters from Rock Mass Classifications, Transportation Research Record 1330, pp.22-29.

176. Yudhbir (1986): Geomechanical Classification of Rock Masses for Evaluation of Strength and Deformation Parameters, 5th International IAEG Congress, Buenos Aires, pp.177-184.

APPENDIX – I

(SALIENT FEATURES OF NATHPA JHAKRI PROJECT)

Hydrology

(i)	Catchment area of the river Satluj at the dam site	49820 sq. km.
(ii)	Discharge available for 90% period	71 cumec
(iii)	Dependable yearly run off	7.689 x 10^9 m^3
(iv)	Mean run off	9.596 x 10^9 m^3
(v)	Design discharge	405 cumec
(vi)	Observed maximum flood at the dam site	2300 cumec
(vii)	Design flood	5660 cumec

A. River Diversion

A diversion tunnel of 8.0 m diameter 350 m long with invert at inlet side at El. 1449.0 m and at the outlet end at El. 1444.0 m is provided to pass 350 cumec water. The bed gradient of the diversion tunnel is 1 in 70. The top level of the upstream coffer dam is at El. 1459.0 m. The top level of the downstream coffer dam is at El. 1446.0 m. The upstream coffer dam is located about 110 m upstream of the dam axis, and the downstream coffer dam is located about 160 m downstream of the dam axis.

B. Dam

(i)	Type of dam	Straight concrete gravity
(ii)	Height (above foundation level)	67.5 m
(iii)	Maximum base width	65.0 m
(iv)	Length (at road level)	154.0 m
(v)	Top of dam	El. 1493.5 m
(v)	Full Reservoir Level (FRL)	El. 1490.5 m
(vi)	Minimum Draw Down Level (MDDL)	El. 1474.0 m
(vii)	Live storage	457.0 Hectare metres

C. Spillway

(i)	Crest level	El. 1488.0 m
(ii)	Gates	2 Nos., each of 7.5 x 2.5 m size

	(iii)	River sluices	6 Nos., each of size 7 x 7.35 m with sill at El. 1458.0 m
	(iv)	Energy dissipation system	Ski jump

D. Desilting Arrangement

(i)	Type	Underground chambers, 4 Nos.
(ii)	Inlet structures	4 Nos.
(iii)	Intake level	El. 1469.0 m
(iv)	Size of each chamber	Length 525 m Width 16.31 m Height 27.5 m
(v)	Flow velocity	33.0 cm/sec (this will have 92% efficiency for removal of 0.2 mm size particles)
(vi)	Flushing tunnel	Length 1560 m Diameter 5.0 m (horse shoe) Slope 1 in 450

E. Head Race Tunnel

(i)	Length	27.4 km
(ii)	Shape	Circular
(iii)	Diameter	10.15 m
(iv)	Gradient	1 in 275, 1 in 61 and 1 in 430
(v)	Design discharge	405 cumec
(vi)	Velocity of flow	5.0 m/sec

F. Construction Adits

Nathpa adit	950 m
Sholding adit	850 m
Nigulsari adit	647 m
Wadhal adit	847 m
Manglad right bank adit	689 m
Rattanpur adit	1357 m

VI Face adit	264 m

G. Sholding Works

Weir	Trench type across Sholding khad
Design discharge	6 cumec

H. Surge Tank

Type	Restricted orifice
Diameter	21.6 / 10.2 m
Height	301 m
Tunnel invert at surge shaft	El. 1398.92 m
Maximum upsurge	El. 1530.05 m
Minimum downsurge	El. 1415.60 m
Upper and lower expansion galleries	15 m D-shaped, 150 m long each

I. Pressure Shaft

Type	Underground inclined shafts
Number of penstocks	3, each bifurcating to feed two units
Diameter and length	Main shaft 4.9 m, 571 to 622 m
	Branch 3.45 m, 60 m

J. Power House

Type	Underground
Size	220 x 20 x 49 m
Gross head	488 m
Design head	425 m
No. of caverns	4 parallel caverns, (i) Turbine hall, (ii) Main machine hall, (iii) Transformer hall and (iv) Draft tube gates and surge chamber

K. Tail Race Tunnel

Size	10.15 m, D-shaped
Length	982 m
Velocity	5 m/sec

APPENDIX – II

(TERZAGHI'S THEORY)

A. Terzaghi's (1946) Rock Load in Tunnels within Various Rock Classes

Rock Class	Rock Condition	Rock Load Factor, H_p	Remarks
I	Hard and intact	Zero	Light lining required only if spalling or popping occurs
II	Hard stratified or schistose	0-0.5B	Light support, mainly for protection against spalling. Load may change erratically from point to point
III	Massive, moderately jointed	0-0.25B	No side pressure
IV	Moderately blocky and seamy	0.25B-0.35 ($B+H_t$)	No side pressure
V	Very blocky and seamy	(0.35-1.10) ($B+H_t$)	Little or no side pressure
VI	Completely crushed	1.10 ($B+H_t$)	Considerable side pressure. Softening effects of seepage toward bottom of tunnel require either continuous support for lower ends of ribs or circular ribs
VII	Squeezing rock – moderate depth	(1.10-2.10) ($B+H_t$)	Heavy side pressure, invert struts required. Circular ribs are recommended
VIII	Squeezing rock – great depth	(2.10-4.50) ($B+H_t$)	-do-
IX	Swelling rock	Upto 250 ft. (80m), irrespective of the value of ($B+H_t$)	Circular ribs are required. In extreme cases, use of yielding support recommended

B. Terzaghi's Rock Load Concept as Modified by Deere et al. (1970) and Rose (1982)

Rock Class & Condition		RQD percent	Rock Load Factor, H_p	Remarks
I	Hard and intact	95-100	Zero	same as A
II	Hard stratified or schistose	90-99	0-0.5B	same as A
III	Massive, moderately jointed	85-95	0-0.25B	same as A
IV	Moderately blocky and seamy	75-85	0.25B-0.35 ($B+H_t$)	same as A
V	Very blocky and seamy	30-75	(0.2-0.6) ($B+H_t$)	Types V and VI reduced by about 50% from Terzaghi values because water table has little effect on rock load (Terzaghi, 1946; Brekke, 1968)
VI	Completely crushed	3-30	(0.6-1.10) ($B+H_t$)	
VIa	Sand and gravel	0-3	(1.1-1.4) ($B+H_t$)	
VII	Squeezing rock – moderate depth	NA	(1.10-2.10) ($B+H_t$)	same as A

VIII	Squeezing rock – great depth	NA	(2.10-4.50) $(B+H_t)$	same as A
IX	Swelling rock	NA	Upto 250 ft. (80m), irrespective of the value of $(B+H_t)$	same as A

C. Recommendations of Singh et al. (1995) on Support Pressure for Rock Tunnels

Terzaghi's Classification			Classification of Singh et al., 1995				Remarks
Category	Rock Condition	Rock Load Factor, H_p	Category	Rock Condition	Recommended Support Pressure, MPa		
					p_v	p_h	
I	Hard & intact	0	I	Hard & intact	0	0	--
II	Hard stratified or schistose	0-0.5B	II	Hard stratified or schistose	0.0-0.04	0	--
III	Massive, moderately jointed	0-0.25B	III	Massive, moderately jointed	0.04-0.07	0	--
IV	Moderately blocky seamy & Jointed	0.25B-0.35 $(B+H_t)$	IV	Moderately blocky seamy very Jointed	0.07-0.1	0-0.2 p_v	Inverts may be required
V	Very blocky & seamy, shattered arched	(0.35-1.1) $(B+H_t)$	V	Very blocky & seamy, shattered, highly jointed, thin shear zone or fault	0.1-0.2	0-0.5 p_v	Inverts may be required, arched roof preferred
VI	Completely crushed but chemically intact	1.1 $(B+H_t)$	VI	Completely crushed but chemically unaltered, thick shear and fault zone	0.2-0.3	0.3-1.0 p_v	Inverts essential, arched roof essential
VII	Squeezing rock at moderate depth	(1.1-2.1) $(B+H_t)$	VII	Squeezing rock condition			
				A. mild squeezing (u_a/a upto 3%)	0.3-0.4	Depends on primary stress values p_h may exceed p_v	Inverts essential. In excavation flexible support preferred. Circular section recommended
				B. moderate squeezing (u_a/a = 3 to 5%)	0.4-0.6	-do-	-do-
VIII	Squeezing rock at great depth	(2.1-4.5) $(B+H_t)$	VII	C. high squeezing (u_a/a > 5%)	0.6-1.4	-do-	-do-
IX	Swelling rock	upto 80m	VIII	Swelling rock			
				A. mild swelling	0.3-0.8	Depends on type & content of swelling clays, p_h may exceed p_v	Inverts essential in excavation, arched roof essential
				B. moderate swelling	0.8-1.4	-do-	-do-
				C. high swelling	1.4-2.0	-do-	-do-

APPENDIX – III

(ROCK STRUCTURE RATING SYSTEM)

A. Rock Structure Rating – Parameter A

General Area Geology (Maximum Value 30)

Basic Rock Type

	Hard	Med.	Soft	Decomp.
Igneous	1	2	3	4
Metamorphic	1	2	3	4
Sedimentary	2	3	4	4

Basic Rock Type	Geological Structure			
	Massive	Slightly Faulted or Folded	Moderately Faulted or Folded	Intensely Faulted or Folded
Type 1	30	22	15	9
Type2	27	20	13	8
Type 3	24	18	12	7
Type 4	19	15	10	6

B. Rock Structure Rating – Parameter B

Joint Pattern (Maximum Value 45)								
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
1.Very closely jointed < 2 in	9	11	13	10	12	9	9	7
2.Closely jointed 2-6 in	13	16	19	15	17	14	14	11
3.Moderately jointed 6-12 in	23	24	28	19	22	23	23	19
4.Moderate to Blocky 1-2 ft	30	32	36	25	28	30	28	24
5.Blocky to Massive 2-4 ft	36	38	40	33	35	36	34	28
6.Massive > 4 ft	40	43	45	37	40	40	38	34

Note: Flat: 0-20°; Dipping: 20°-50°; Vertical: 50°-90°

C. Rock Structure Rating – Parameter C

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

D. Theoretical Rib Spacing for Datum Condition

Wickham et al. (1974) have given recommendations for estimation of spacing of steel ribs for different tunnel diameters for datum condition. For a particular rib ratio obtained from Eq. 6.3, spacing of ribs for the selected section can then be computed from Eq. 6.2.

But the problem faced in using this table is that, it uses FPS system of units of measurement and the steel ribs do not conform to the Indian Standard Specifications. Moreover, these recommendations have been made for tunnel diameters up to 9 m only. Therefore to facilitate use of steel ribs manufactured in India as per Indian Standards, Fig.III.1 and Table III.1 have been developed to get the spacing of various steel ribs for datum condition, beyond 9 m also.

In Fig.III.1 and Table III.1, H_t is kept equal to B in Eq. 6.1. Considering B in mm, roof pressure obtained for datum condition is as follows:

$$p_v = 1.38 \times 2B \times 19 \times 10^{-6} \quad \text{MPa} \quad (\gamma = 19\ \text{kN/m}^3) \tag{III.1}$$

Now, for a particular steel rib, spacing should be such that it is capable of resisting the roof pressure computed in Eq. III.1. Therefore,

$$\frac{2A_s.\sigma_{ys}}{B.S_{rib}} = p_v \tag{III.2}$$

Putting p_v from Eq. III.1 and rearranging

$$S_{rib} = 3.8 \times 10^4 \times \frac{A_s.\sigma_{ys}}{B^2} \tag{III.3}$$

where, A_s is in mm^2, B in mm and σ_{ys} in MPa.

σ_{ys} considered is 245 N/mm^2.

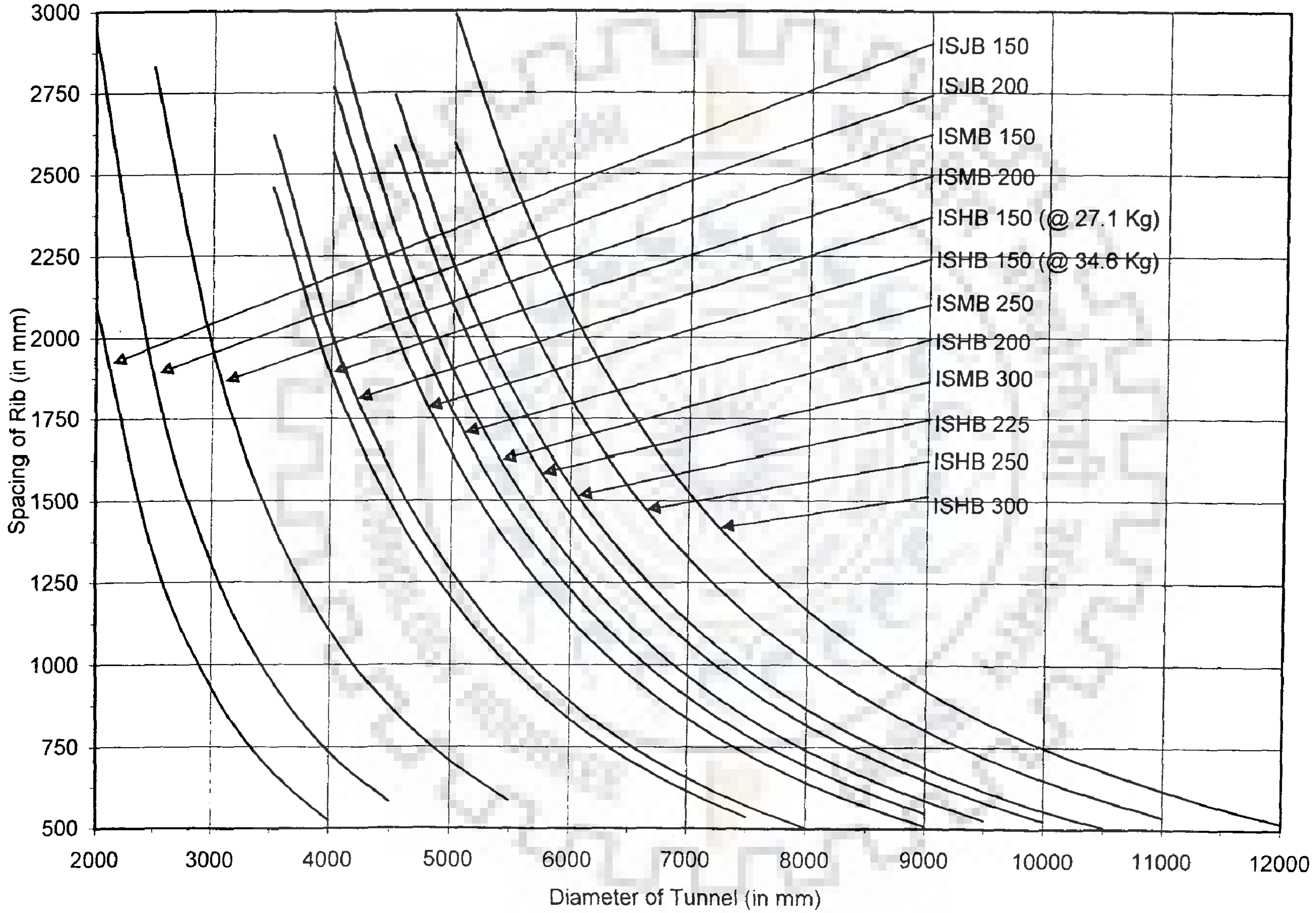


Fig. III.1 Theoretical spacing of typical rib sizes for datum condition

Table III.1 Theoretical Spacing of Typical Rib Sizes for Datum Condition (spacing in mm)

Rib Size	Tunnel Diameter (in mm)																				
	2000	2500	3000	3500	4000	4500	5000	5500	6000	6500	7000	7500	8000	8500	9000	9500	10000	10500	11000	11500	12000
ISJB 150	2097	1342	932	685	524																
ISJB 200	2942	1883	1308	961	735	581															
ISMB 150		2830	1965	1444	1106	874	708	585													
ISMB 200				2457	1881	1486	1204	995	836	712	614	535									
ISHB 150 (@ 27.1 Kg)				2620	2006	1585	1284	1061	892	760	655	571	502								
ISHB 150 (@ 34.6 Kg)					2565	2027	1642	1357	1140	971	838	730	641	568	507						
ISMB 250					2767	2186	1771	1463	1230	1048	903	787	692	613	547						
ISHB 200					2964	2342	1897	1568	1317	1122	968	843	741	656	585	525					
ISMB 300						2587	2095	1732	1455	1240	1069	931	818	725	647	580	524				
ISHB 225						2743	2222	1836	1543	1315	1134	987	868	769	686	615	555	504			
ISHB 250							2596	2145	1803	1536	1324	1154	1014	898	801	719	649	589	536		
ISHB 300							2989	2470	2075	1768	1525	1328	1167	1034	922	828	747	678	617	565	519

APPENDIX - IV
(ROCK MASS RATING SYSTEM)

A. Classification Parameters and their Ratings

	Parameter		Ratings of values						
1	Strength of intact rock material	Point-load strength index (MPa)	> 10	4-10	2-4.	1-2	For this low range, UCS test preferred		
		Uniaxial compressive strength (MPa)	> 250	100-250	50-100	25-50	5-25	1-5	< 1
	Rating		15	12	7	4	2	1	0
2	Drill core quality RQD (%)		90-100	75-100	50-75	25-50	< 25		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		> 2m	0.6-2m	200-600mm	60-200mm	< 60mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities		Very rough surfaces, Not continuous, No separation, Unweathered wall rock	Slightly rough surfaces, Separation <1mm, Slightly weathered walls	Slightly rough surfaces, Separation <1mm, Highly weathered walls	Slickensided surfaces or Gouge <5mm thick or Separation 1-5mm, Continuous	Soft gouge >5mm thick or Separation > 5mm, Continuous		
	Rating		30	25	20	10	0		
5	Ground-water	Inflow per 10 m tunnel length (L/min)	None Or	< 10 or	10-25 or	25-125 or	> 125 or		
		Joint water pressure / Major principal stress	0 Or	< 0.1 or	0.1-0.2 or	0.2-0.5 or	> 0.5 or		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

B. Rating Adjustment for Discontinuity Orientations

Strike & Dip Orientations of Discontinuities		Very Favorable	Favorable	Fair	Unfavorable	Very unfavorable
Ratings	Tunnels and mines	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	20 yr for 15m span	1 yr for 10m span	1 wk for 5m span	10 hr for 2.5m span	30 min for 1m span
Cohesion of the rock mass (kPa)	> 400	300 - 400	200 - 300	100 - 200	< 100
Friction angle of the rock mass (deg)	> 45	35 - 45	25 - 35	15 - 25	< 15

E. Effect of Discontinuity Strike and Dip Orientations in Tunneling

Strike Perpendicular to Tunnel Axis				Strike Parallel to Tunnel Axis		Irrespective of Strike
Drive with Dip		Drive against Dip				
Dip 45 - 90	Dip 20 - 45	Dip 45 - 90	Dip 20 - 45	Dip 20 - 45	Dip 45 - 90	Dip 0 - 20
Very favorable	Favorable	Fair	Unfavorable	Fair	Very unfavorable	Fair

F. Guidelines for Excavation and Support of Rock Tunnels as per the RMR System

Rock Mass Class	Excavation	Support		
		Rock Bolts (20mm Dia.,Fully Grouted)	Shotcrete	Steel Sets
Very good rock I RMR: 81-100	Full face 3-m advance	Generally, no support required except for occasional spot bolting		
Good rock II RMR : 61-80	Full face 1.0-1.5m advance Complete support 20m from face	Locally, bolts in crown 3m long, spaced 2.5m, with occasional wire mesh	50mm in crown where required	None
Fair rock III RMR : 41-60	Top heading and bench, 1.5-3m advance in top heading, Commence support after each blast, Complete support 10m from face	Systematic bolts 4m long, spaced 1.5-2m in crown and walls with wire mesh in crown	50-100mm in crown and 30mm in sides	None
Poor rock IV RMR : 21-40	Top heading and bench, 1-1.5m advance in top heading, Install support concurrently with excavation 10m from face	Systematic bolts 4-5m long, spaced 1-1.5m in crown and walls with wire mesh	100-150mm in crown and 100mm in sides	Light to medium ribs spaced 1.5m where required
Very poor rock V RMR : < 20	Multiple drifts, 0.5-1.5m advance in top heading, Install support concurrently with excavation, Shotcrete as soon as possible after blasting	Systematic bolts 5-6m long, spaced 1-1.5m in crown and walls with wire mesh , Bolt invert	150-200mm in crown,150mm in sides and 50mm on face	Medium to heavy ribs spaced 0.75m with steel lagging & forepoling if required,Close invert

APPENDIX – V (ROCK MASS QUALITY SYSTEM)

1. Rock Quality Designation		RQD
A	Very poor	0 – 25
B	Poor	25 – 50
C	Fair	50 – 75
D	Good	75 – 90
E	Excellent	90 - 100

2. Joint Set Number		J_n
A	Massive, none or few joints	0.5 – 1.0
B	One joint set	2
C	One joint set plus random	3
D	Two joint sets	4
E	Two joint sets plus random	6
F	Three joint sets	9
G	Three joint sets plus random	12
H	Four or more joint sets, random, heavily jointed, "suger cube", etc.	15
I	Crushed rock, earth like	20

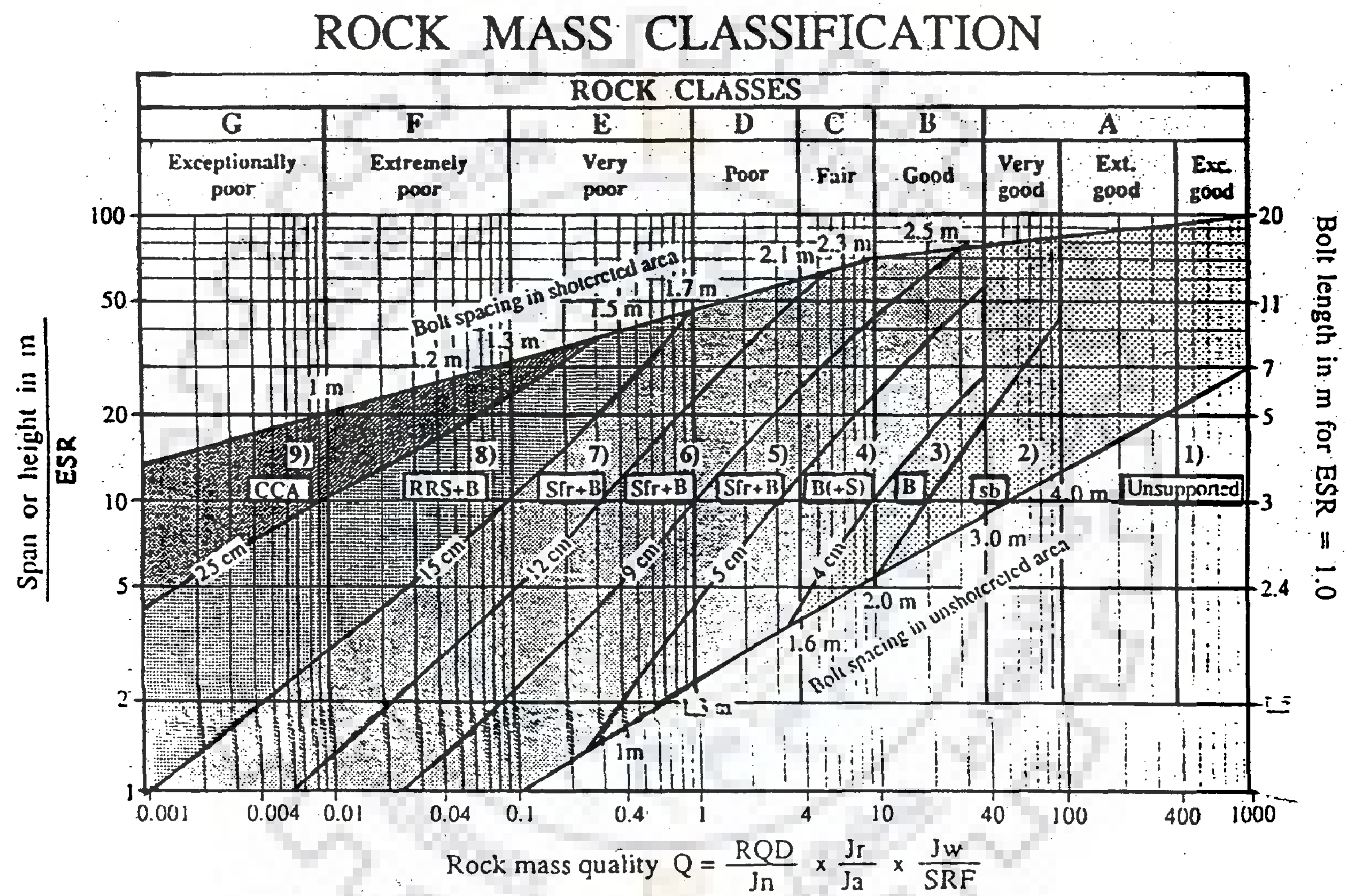
3. Joint Roughness Number		J_r
(a) Rock wall contact and (b) rock wall contact before 10cm shear		
A	Discontinuous joints	4
B	Rough or irregular, undulating	3
C	Smooth, undulating	2
D	Slickensided, undulating	1.5
E	Rough or irregular, planar	1.5
F	Smooth, planar	1.0
G	Slickensided, planar	0.5
(c) No rock wall contact when sheared		
H	Zone containing clay minerals thick enough to prevent rock wall contact	1.0
I	Sandy, gravelly or crushed zone thick enough to prevent rock wall contact	1.0

4. Joint Alteration Number		ϕ(deg.)	J_a
(a) Rock wall contact (no mineral fillings, only coatings)			
A	Tightly healed, hard, non-softening, impermeable filling, i.e. quartz or epidote		0.75
B	Unaltered joint walls, surface staining only	25-35	1.0
C	Slightly altered joint walls. Non-softening mineral coatings, sandy particles, clay-free disintegrated rock etc.	25-30	2.0
D	Silty or sandy clay coatings, small clay fraction (non-softening)	20-25	3.0
E	Softening or low-friction clay mineral coatings, i.e.kaolinite, mica. Also chlorite, talc, gypsum & graphite etc. & small quantities of swelling clays	8-16	4.0
(b) Rock wall contact before 10cm shear (thin mineral fillings)			
F	Sandy particles, clay-free disintegrated rock etc.	25-30	4.0
G	Strongly over-consolidated, non-softening clay mineral fillings (continuous, <5mm in thickness)	16-24	6.0
H	Medium or low over-consolidation, softening clay mineral fillings (continuous, <5mm thick)	12-16	8.0
J	Swelling clay fillings, i.e., montmorillonite (continuous, < 5mm in thickness)	6-12	8-12
(c) No rock wall contact when sheared			
KLM	Zones or bands of disintegrated or crushed rock and clay(see G,H,J for description of clay condition)	6-24	6,8 or 8-12
N	Zones or bands of silty or sandy clay, small clay fraction (non-softening)		5
OPR	Thick, continuous zones or bands of clay (see G,H,J for description of clay condition)	6-24	10,13 or 13-20

5. Joint Water Reduction Factor		Pressure (MPa)	J_w
A	Dry excavations or minor inflow, i.e., 5 lit/min locally	<0.1	1
B	Medium inflow or pressure occasional out-wash of joint fillings	0.1-0.25	0.66
C	Large inflow or high pressure in competent rock with unfilled joints	0.25-1.0	0.5
D	Large inflow or high pressure, considerably out-wash of joint fillings	0.25-1.0	0.33
E	Exceptionally high inflow or water pressure at blasting, decaying with time	>1.0	0.2-0.1
F	Exceptionally high inflow or water pressure continuing without noticeable decay	>1.0	0.1-0.05

6. Stress Reduction Factor					SRF
(a)	Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated				
A	Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)				10.0
B	Single-weakness zones containing clay or chemically disintegrated rock (depth of excavation <=50m)				5.0
C	Single-weakness zones containing clay or chemically disintegrated rock (depth of excavation >=50m)				2.5
D	Multiple-shear zones in competent rock (clay-free), loose surrounding rock (any depth)				7.5
E	Single-shear zones in competent rock (clay-free) (depth of excavation <=50m)				5.0
F	Single-shear zones in competent rock (clay-free) (depth of excavation >=50m)				2.5
G	Loose open joints, heavily jointed or "suger cube", etc. (any depth)				5.0
Note: (i) Reduce these SRF values by 25-50% if the relevant shear zones only influence but do not intersect the excavation					
(b)	Competent rock, rock stress problems	q_c/σ_1	σ_θ/q_c	SRF (old)	SRF (new)
H	Low stress, near surface open joints	>200	<0.01	2.5	2.5
J	Medium stress, favorable stress condition	200-10	0.01-0.3	1	1
K	High stress, very tight structure (favorable to stability, may be unfavorable to wall stability)	10-5	0.3-0.4	0.5-2	0.5-2.0
L	Moderate slabbing after >1hr in massive rock	5-3	0.5-0.65	5-9	5-50
M	Slabbing and rock burst after a few minutes in massive rock	3-2	0.65-1.0	9-15	50-200
N	Heavy rock burst (strain burst) and immediate deformations in massive rock	<2	>1	15-20	200-400
(c)	Squeezing rock; plastic flow of incompetent rock under the influence of high rock pressure			σ_θ/q_c	SRF
O	Mild squeezing rock pressure			1-5	5-10
P	Heavy squeezing rock pressure			>5	10-20
(d)	Swelling rock; chemical swelling activity depending on pressure of water				
Q	Mild swelling rock pressure				5-10
R	Heavy swelling rock pressure				10-15

Note: J_r and J_a classification is applied to the joint set or discontinuity that is least favorable for stability both from the point of view orientation and shear resistance, τ(where $\tau \approx \sigma_n . J_r/J_a$).



REINFORCEMENT CATEGORIES:

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

Fig. V·1 - Permanent support recommendations based on Q.(Grimstad & Barton, 1993)

APPENDIX – VI

(ROCK MASS INDEX SYSTEM)

A. The Values and Ratings of the Input Parameters to RMi

INPUT PARAMETERS TO THE RMi	
UNIAXIAL COMPRESSIVE STRENGTH of intact rock (MPa)	May be found from laboratory tests (or estimated from handbook tables etc.)
BLOCK VOLUME (V_b) (cu.m)	May be measured at site (from observations or from bore hole cores)

THE JOINT ROUGHNESS FACTOR (jR)		Large scale waviness of joint planes				
		Planar	Slightly undulating	Undulating	Strongly undulating	Stepped or interlocking
Small scale smoothness of joint surface	Very rough	2	3	4	6	6
	Rough	1.5	2	3	4.5	6
	Smooth	1	1.5	2	3	4
	Polished or slickensided*	0.5	1	1.5	2	3
	For filled joints jR = 1	For irregular joints a rating of jR = 6 is suggested				

* For slickensided surfaces the ratings given cover possible movement along the lineations. (For movements across lineations, rough or very rough rating should be applied for the surface)

THE JOINT ALTERATION FACTOR (jA) (the ratings are based on J_a in the Q-system)

Contact between joint walls	CLEAN JOINTS	Healed or welded joints	filling of quartz, epidote etc.		jA = 0.75
		Fresh joint walls	no coating or filling, except from staining (rust)		1
		Altered joint walls	- one grade higher alteration than the rock		2
			- two grade higher alteration than the rock		4
	COATING or THIN FILLING OF	Frictional materials	sand, silt, calcite, etc. (non-softening)		3
		Cohesive materials	clay, chlorite, talc, etc.		4
				Thin filling (< 5mm)	Thick filling
Some or no wall contact	THICK FILLING OF	Frictional materials	sand, silt, calcite, etc. (non-softening)	jA = 4	8
		Hard, cohesive materials	clay, chlorite, talc, etc.	6	6 – 10
		Soft, cohesive materials	clay, chlorite, talc, etc.	8	12
		Swelling clay materials		8 – 12	13 - 20

THE JOINT SIZE FACTOR (jL)		continuous	discont*
Bedding or foliation partings etc.	length < 0.5m	jL = 3	jL = 6
Joints	with length 0.1 – 1m	2	4
	with length 1 – 10m	1	2
	with length 10 – 30m	0.75	1.5
(Filled) joint, seam or shear*	length > 30m	0.5	1

* Discontinuous joints end in massive rock;** Often a singularity (special feature) and should in these cases be treated separately

B. Ratings of the Adjustment Factors for Rock Support Estimates in Blocky Ground

STRESS LEVEL		
Very low (in portals, etc.)	(overburden < 10m)	SL = 0.1
Low	(overburden 10 - 35m)	0.5
Moderate	(overburden 35 - 350m)	1
High	(overburden > 350m)	1.5*

* For stability in high walls a high stress level may be unfavourable. Possible rating SL = 0.5 – 0.75

NUMBER OF JOINT SETS*		NUMBER OF JOINT SETS*	
One set	N_j = 3	Three sets	N_j = 1
One set + random	2	Three sets + random	0.85
Two sets	1.5	Four sets	0.75
Two sets + random	1.2	Four sets + random	0.65

* Means the number of joint sets at the actual location

ORIENTATION OF JOINTS (related to the axis of the tunnel)				
IN WALL		IN ROOF	TERM	
for strike > 30°	for strike < 30°	for all strikes		
dip < 30°	dip < 20°	dip > 60°	favourable	C_o = 1
dip = 30 – 60°	dip = 20 – 45°	dip = 45 – 60°	fair	1.5
dip > 60°	dip = 45 – 60°	dip = 20 – 45°	unfavourable	2
-	dip > 60°	dip < 20°	very unfavourable	3

INCLINATION OF ACTUAL TUNNEL SURFACE (ROOF or WALL)	
Horizontal (roof)	C=1
30° inclination (roof in shaft)	1.5
45° inclination (roof in shaft)	2.2
60° inclination (roof in shaft)	3
Vertical (walls)	5

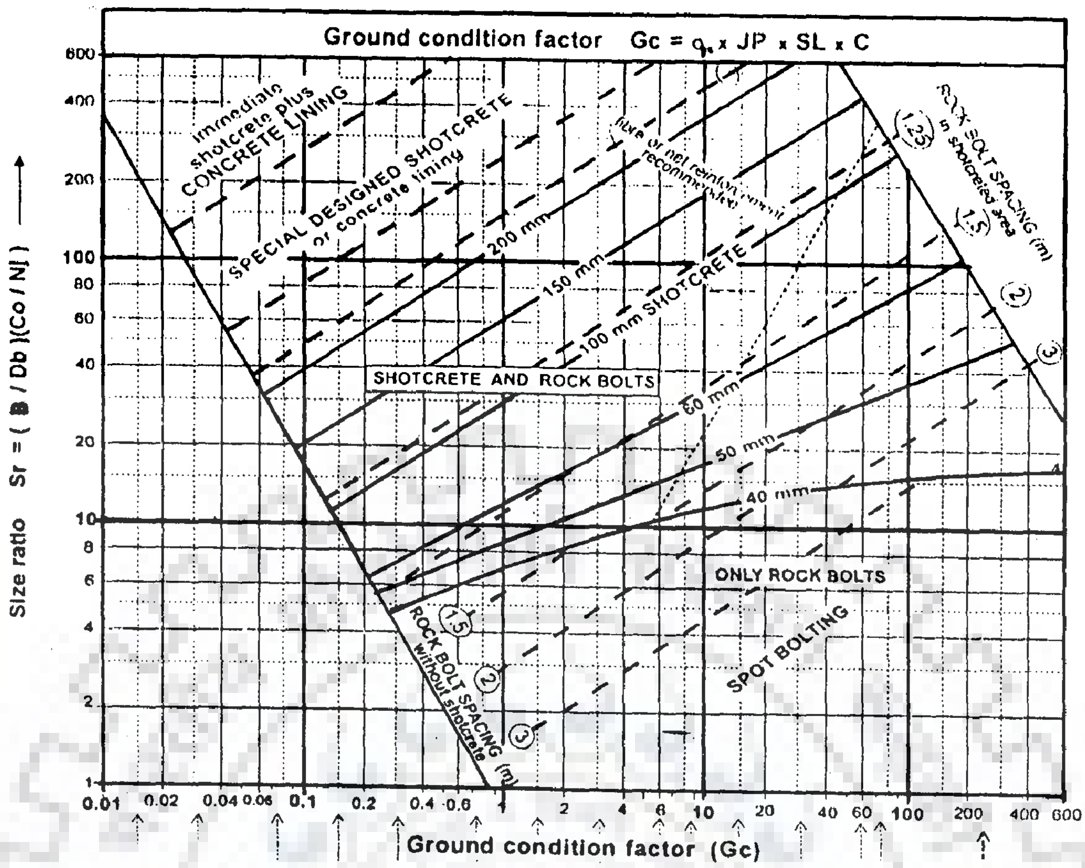


Fig. VI.1 - Rock support for blocky ground including weakness zones

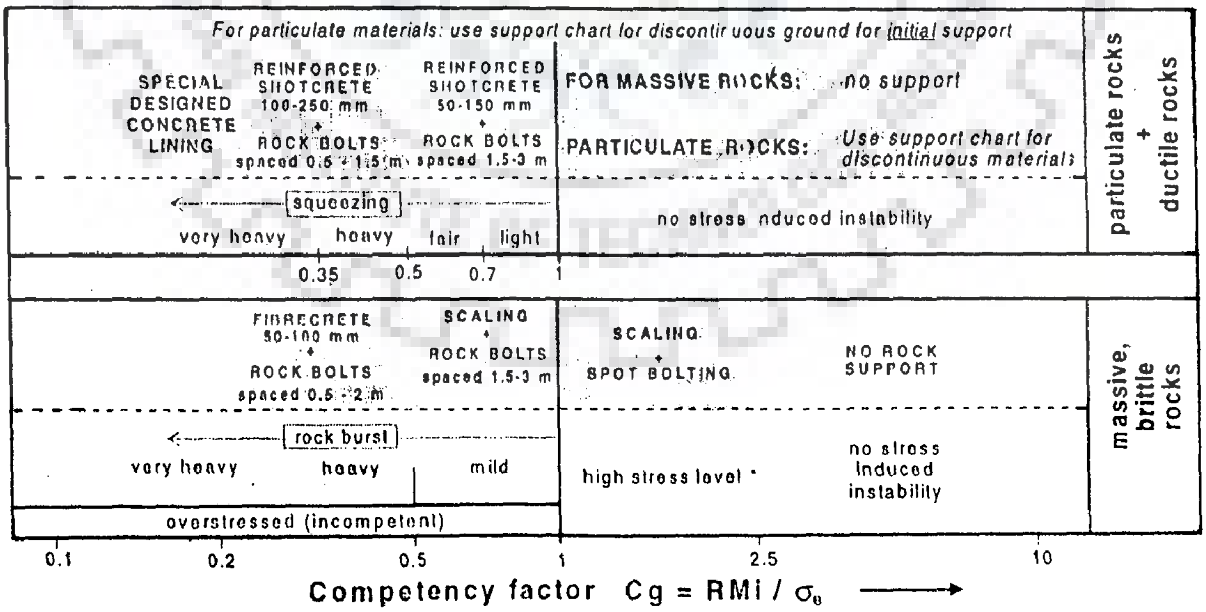


Fig. VI.2 - Chart for estimating support in continuous (massive & particulate) ground

APPENDIX - VIIA
(FLOW CHARTS OF THE COMPUTER PROGRAM FOR ROCK MASS CLASSIFICATION)

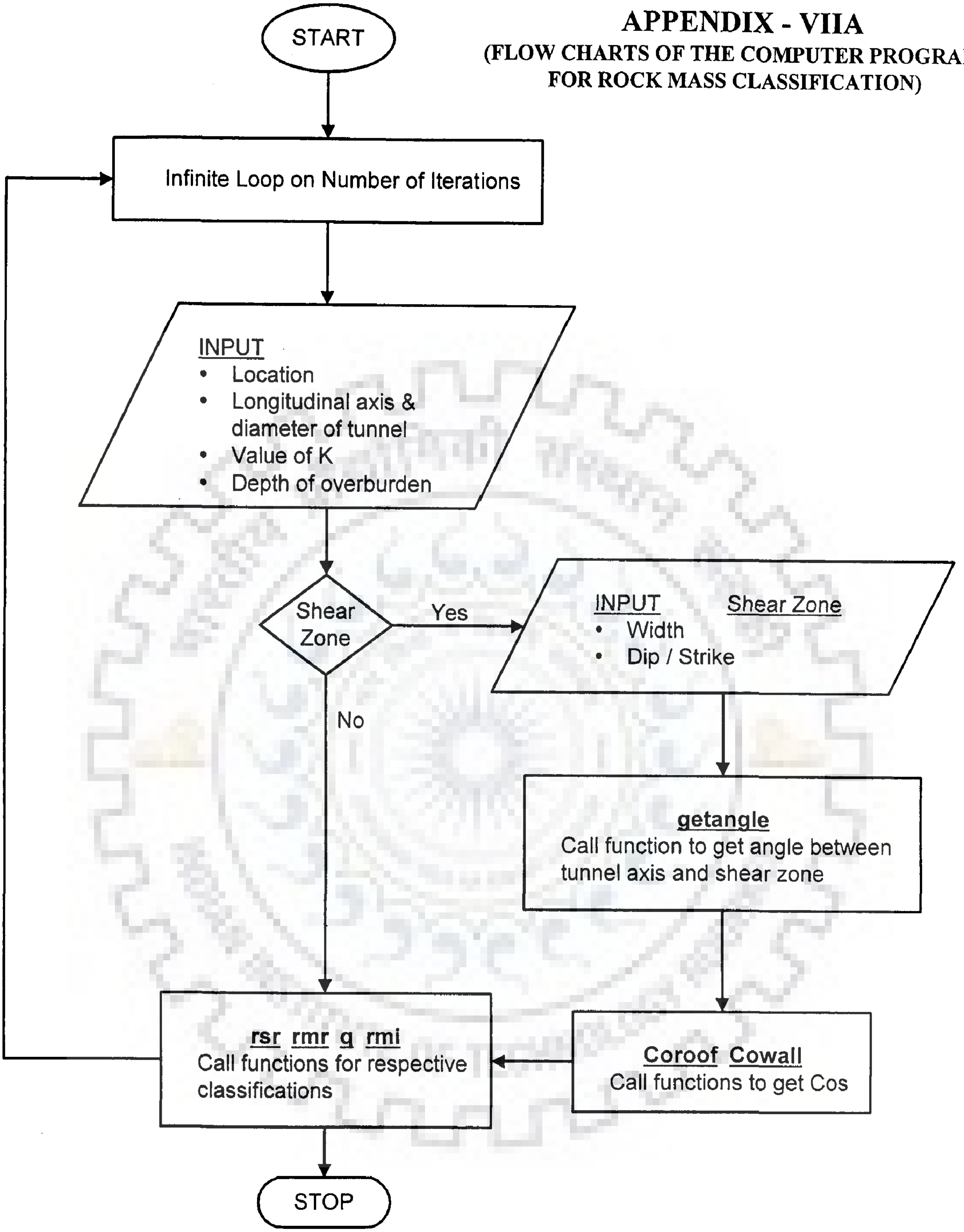


Fig. VII.1 Flow chart for main program (main) for rock mass classification by RSR, RMR, Q and RMi systems

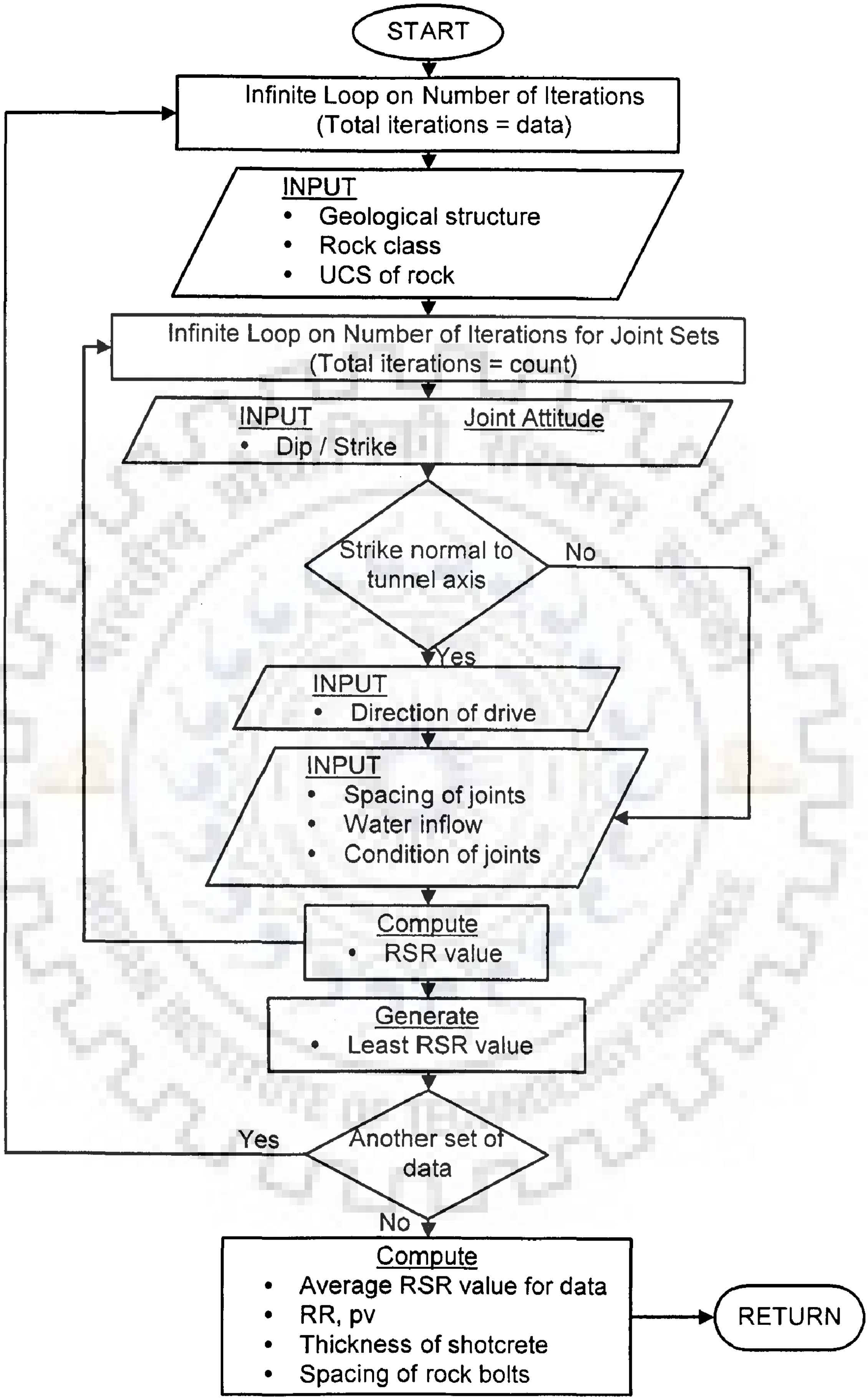


Fig. VII.2 Flow chart for sub program (rsr) for rock mass classification by RSR-system

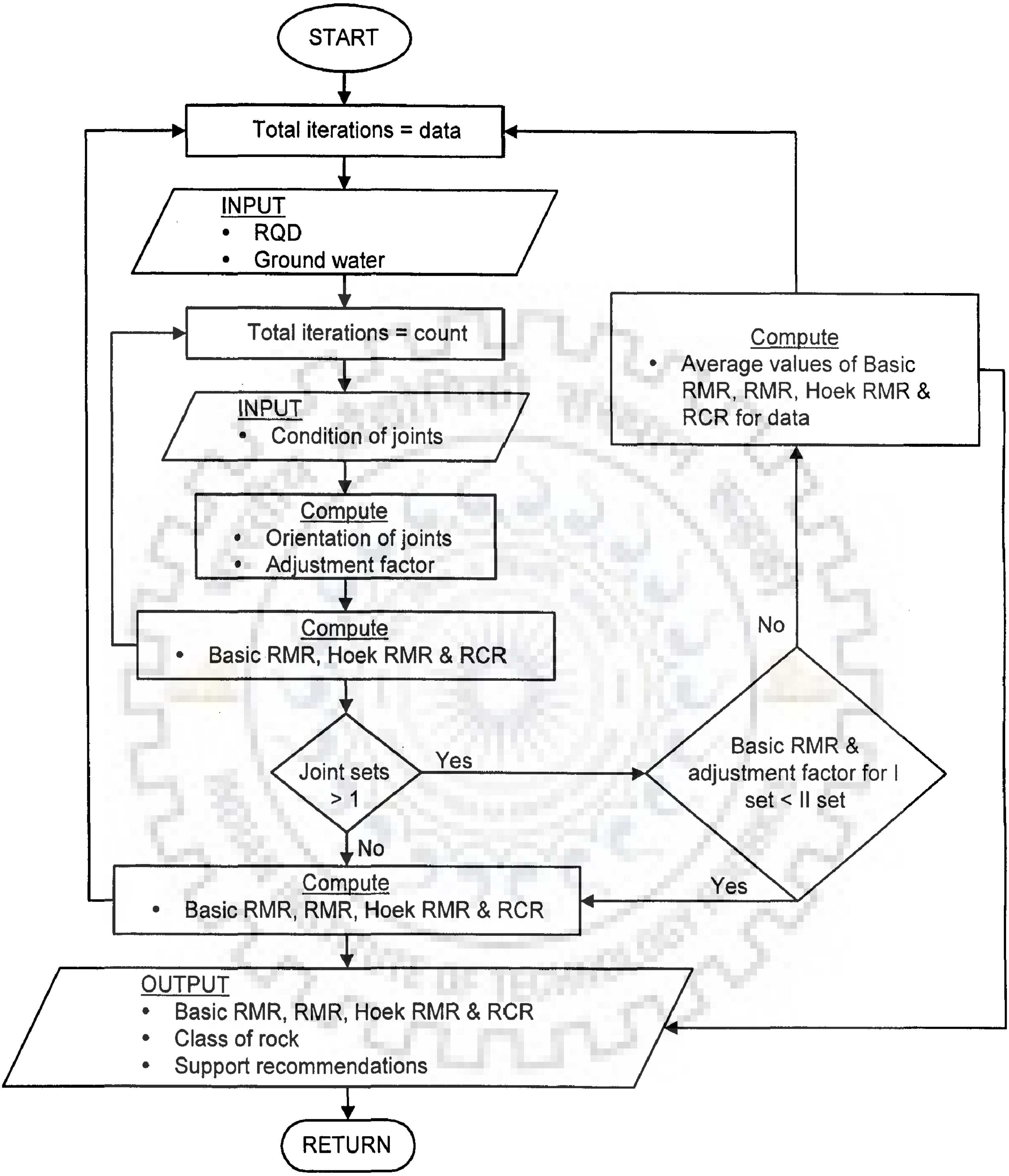
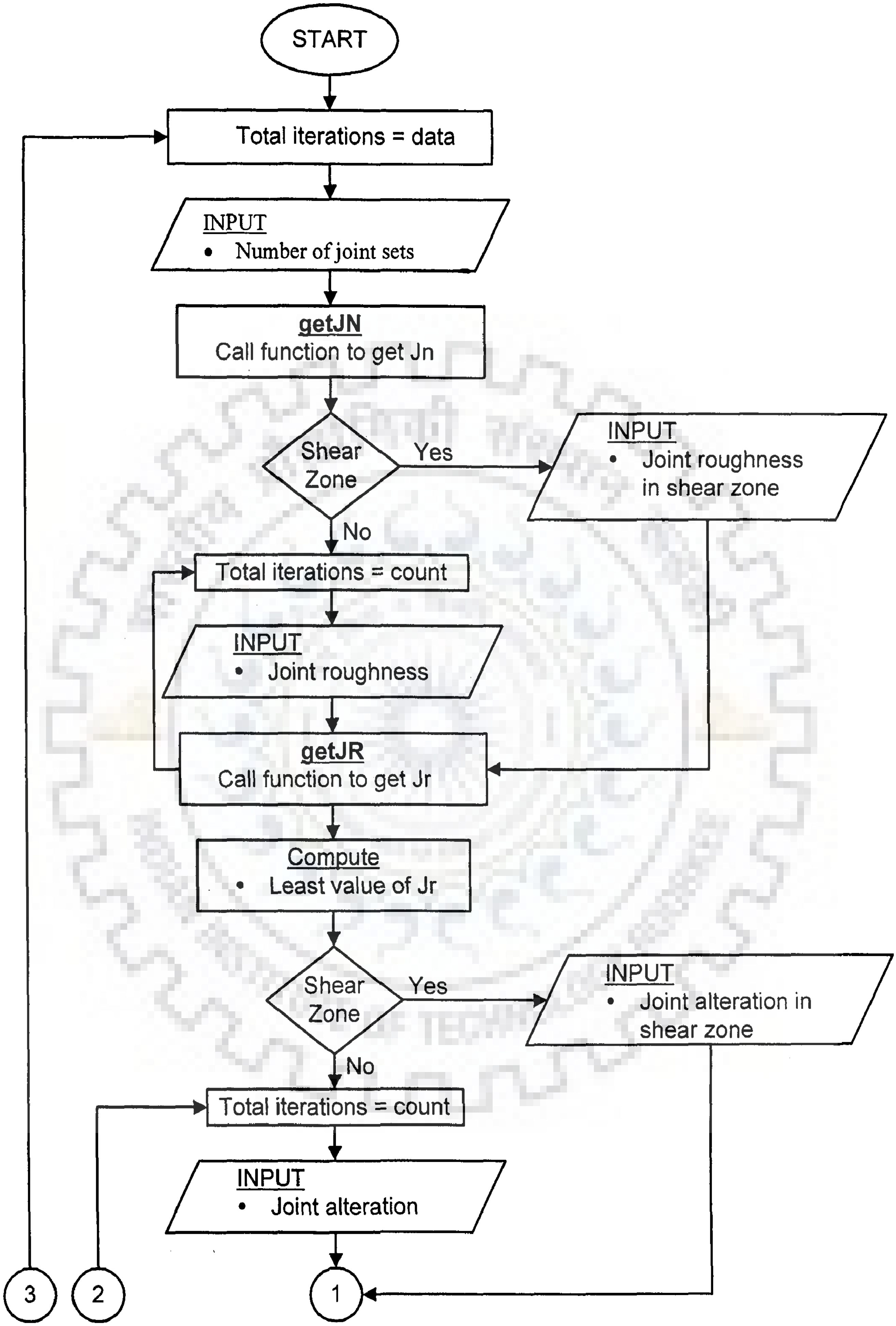


Fig. VII.3 Flow chart for sub program (rmr) for rock mass classification by RMR-system


START
Total iterations = data
INPUT
• Number of joint sets
getJN
Call function to get Jn
Shear Zone
Yes
No
INPUT
• Joint roughness in shear zone
Total iterations = count
INPUT
• Joint roughness
getJR
Call function to get Jr
Compute
• Least value of Jr
Shear Zone
Yes
No
INPUT
• Joint alteration in shear zone
Total iterations = count
INPUT
• Joint alteration
3
2
1

Contd.

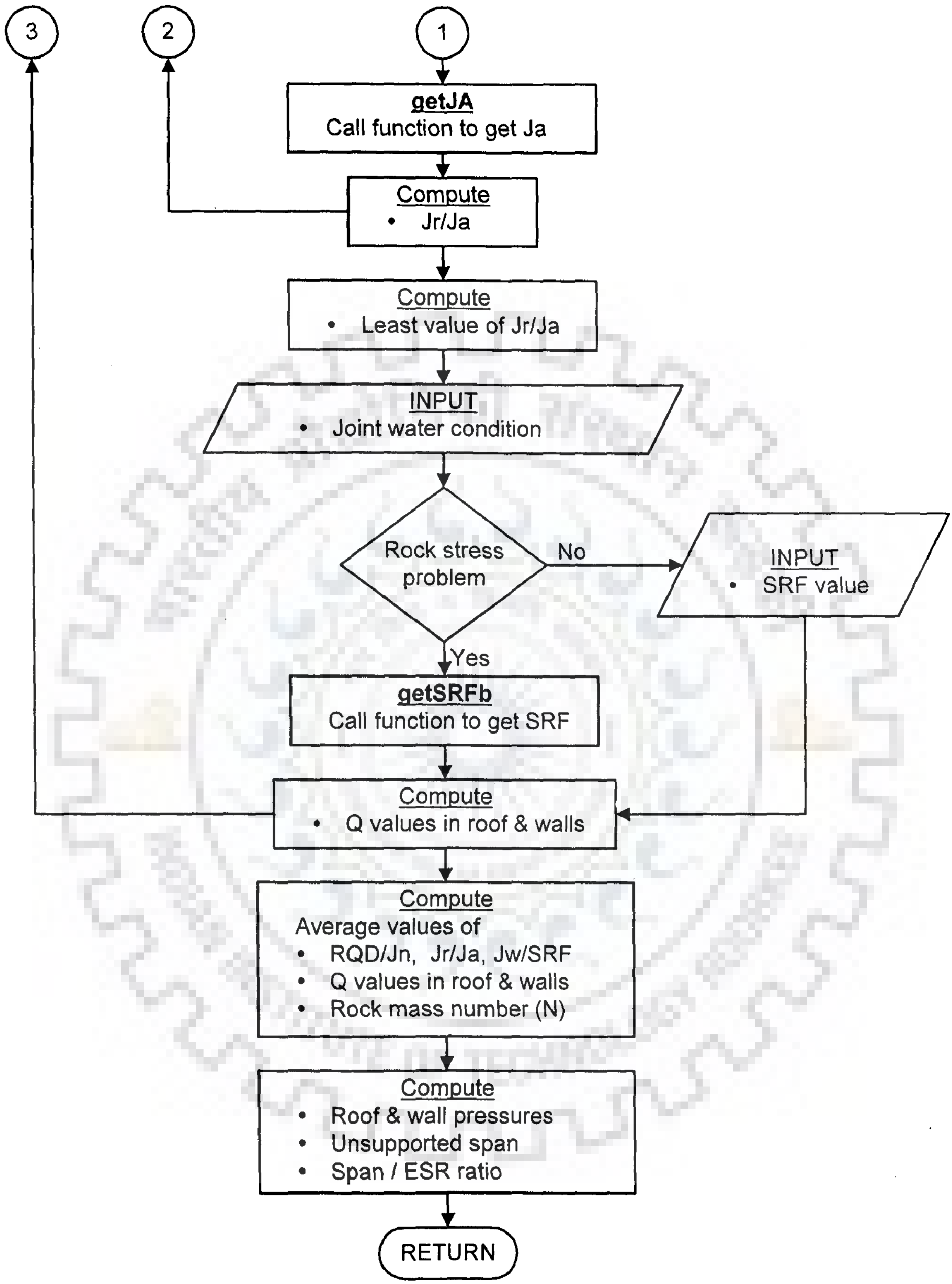
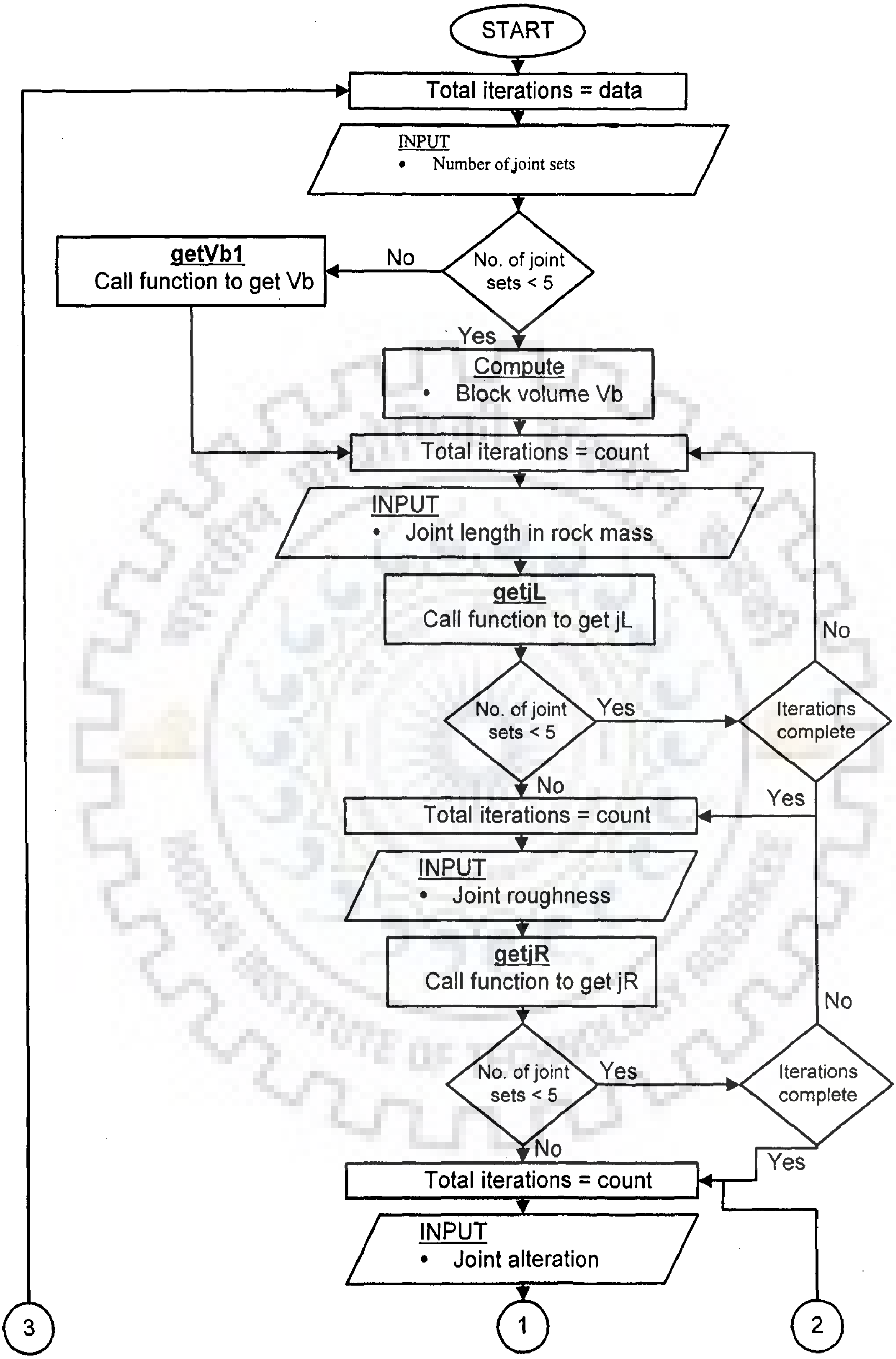


Fig. VII.4 Flow chart for sub program (q) for rock mass classification by Q-system


START
Total iterations = data
INPUT
• Number of joint sets
No. of joint sets < 5
No
getVb1
Call function to get Vb
Yes
Compute
• Block volume Vb
Total iterations = count
INPUT
• Joint length in rock mass
getjL
Call function to get jL
No. of joint sets < 5
Yes
Iterations complete
No
No
Yes
Total iterations = count
INPUT
• Joint roughness
getjR
Call function to get jR
No. of joint sets < 5
Yes
Iterations complete
No
No
Yes
Total iterations = count
INPUT
• Joint alteration
3
1
2

Contd.

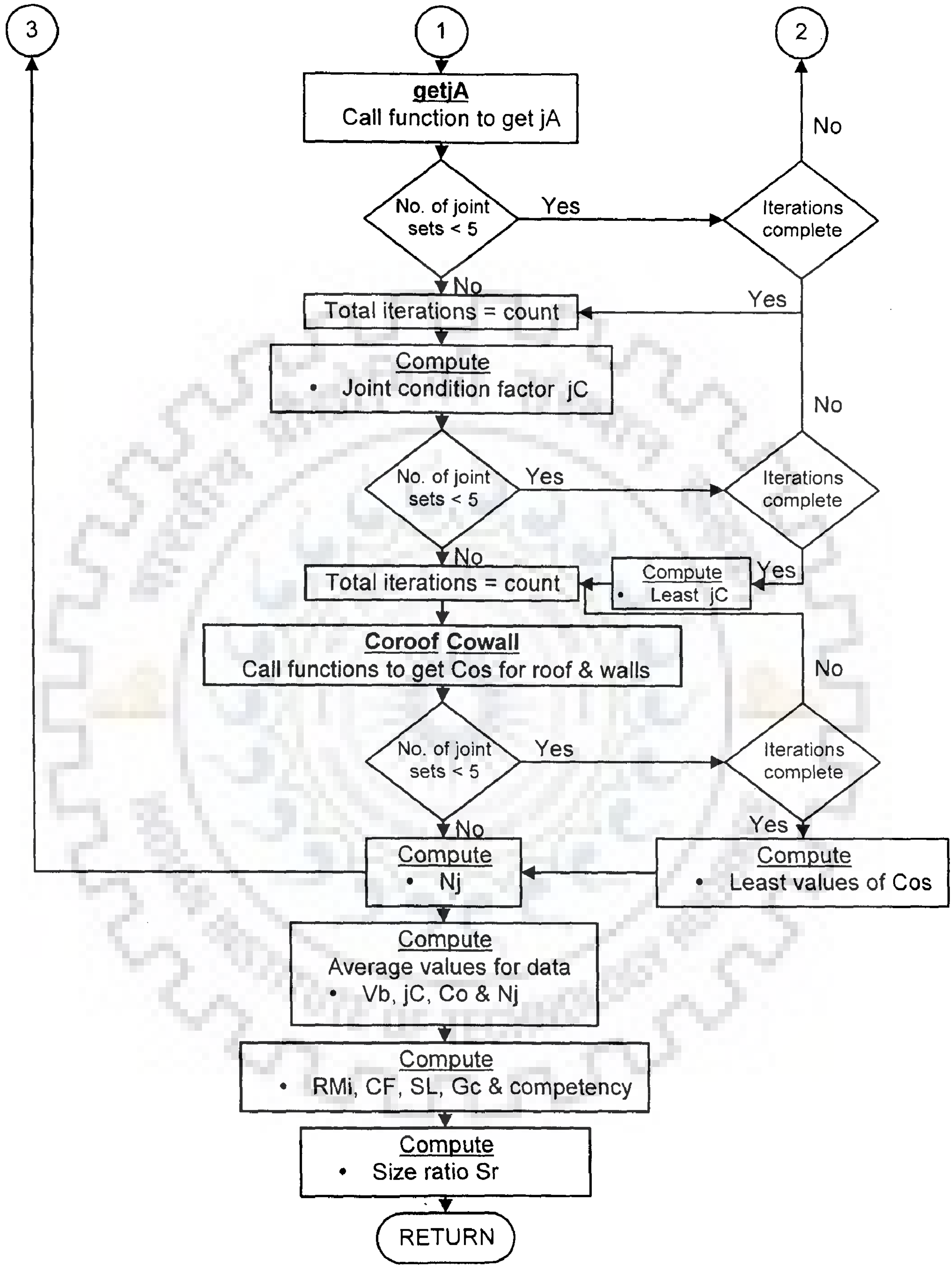


Fig. VII.5 Flow chart for sub program (rmi) for rock mass classification by RMi-system

APPENDIX - VIIB

(OUTPUT OF THE COMPUTER PROGRAM FOR ROCK MASS CLASSIFICATION)

CH 0-50

RSR System: RSR=67 RR=10.72 p_v=64.95 kPa t_{sc}=55 mm S_{bolt}=1.75 m

RMR System: Basic RMR=79 RMR=75 Hoek RMR=79 RCR=72
Good rock(classII)
1. ROCK BOLTS(20 mm dia.,fully grouted):
ROOF: locally,3 m long @ 2.4 m with occasional wire mesh
2. SHOTCRETE:
ROOF: 50 mm where required

Q System: Q(roof)=0.422 p_v=0.177 MPa Q(wall)=1.056 p_h=0.131 MPa
RQD/Jn=8.444 Jr/Ja=0.75 Jw/SRF=0.067
Rock mass number(N)=6.333 Unsupported span(De)=2.25 m

RMi System: ROOF: RMi=7.25 CF=14.24 Cg=0.2 SL=1.5 Sr=28.48 Gc=10.88
WALL: RMi=7.25 CF=14.24 Cg=0.2 SL=1.5 Sr=28.48 Gc=54.38

CH 50-53

RSR System: RSR=59 RR=18.88 p_v=114.35 kPa t_{sc}=75 mm S_{bolt}=1.35 m

RMR System: Basic RMR=59 RMR=55 Hoek RMR=70 RCR=52
Fair rock(classIII)
1. ROCK BOLTS(20 mm dia.,fully grouted):
ROOF: systematic,3.25 m long @ 1.9 m + wire mesh
2. SHOTCRETE:
ROOF: 65 mm
WALL: 30 mm

Q System: Q(roof)=1.167 p_v=0.103 MPa Q(wall)=1.167 p_h=0.103 MPa
RQD/Jn=11.667 Jr/Ja=0.5 Jw/SRF=0.2
Rock mass number(N)=2.917 Unsupported span(De)=3.4 m

RMi System: ROOF: RMi=3.97 CF=14.18 Cg=0.11 SL=1.5 Sr=1.01 Gc=5.96
WALL: RMi=3.97 CF=14.18 Cg=0.11 SL=1.5 Sr=2.03 Gc=29.81

CH 53-110

RSR System: RSR=71 RR=10.72 p_v=64.95 kPa t_{sc}=55 mm S_{bolt}=1.75 m

RMR System: Basic RMR=67 RMR=64 Hoek RMR=72 RCR=60
Good rock(classII)
1. ROCK BOLTS(20 mm dia.,fully grouted):
ROOF: locally,3 m long @ 2.1 m with occasional wire mesh
2. SHOTCRETE:
ROOF: 50 mm where required

Q System: Q(roof)=0.625 p_v=0.127 MPa Q(wall)=1.563 p_h=0.094 MPa
RQD/Jn=12.5 Jr/Ja=0.75 Jw/SRF=0.067
Rock mass number(N)=9.375 Unsupported span(De)=2.65 m

RMi System: ROOF: RMi=9.16 CF=11.67 Cg=0.24 SL=1.5 Sr=19.45 Gc=13.74
WALL: RMi=9.16 CF=11.67 Cg=0.24 SL=1.5 Sr=19.45 Gc=68.71

APPENDIX – VIII

(SUPPORT SYSTEMS INSTALLED BY THE PROJECT AUTHORITIES)

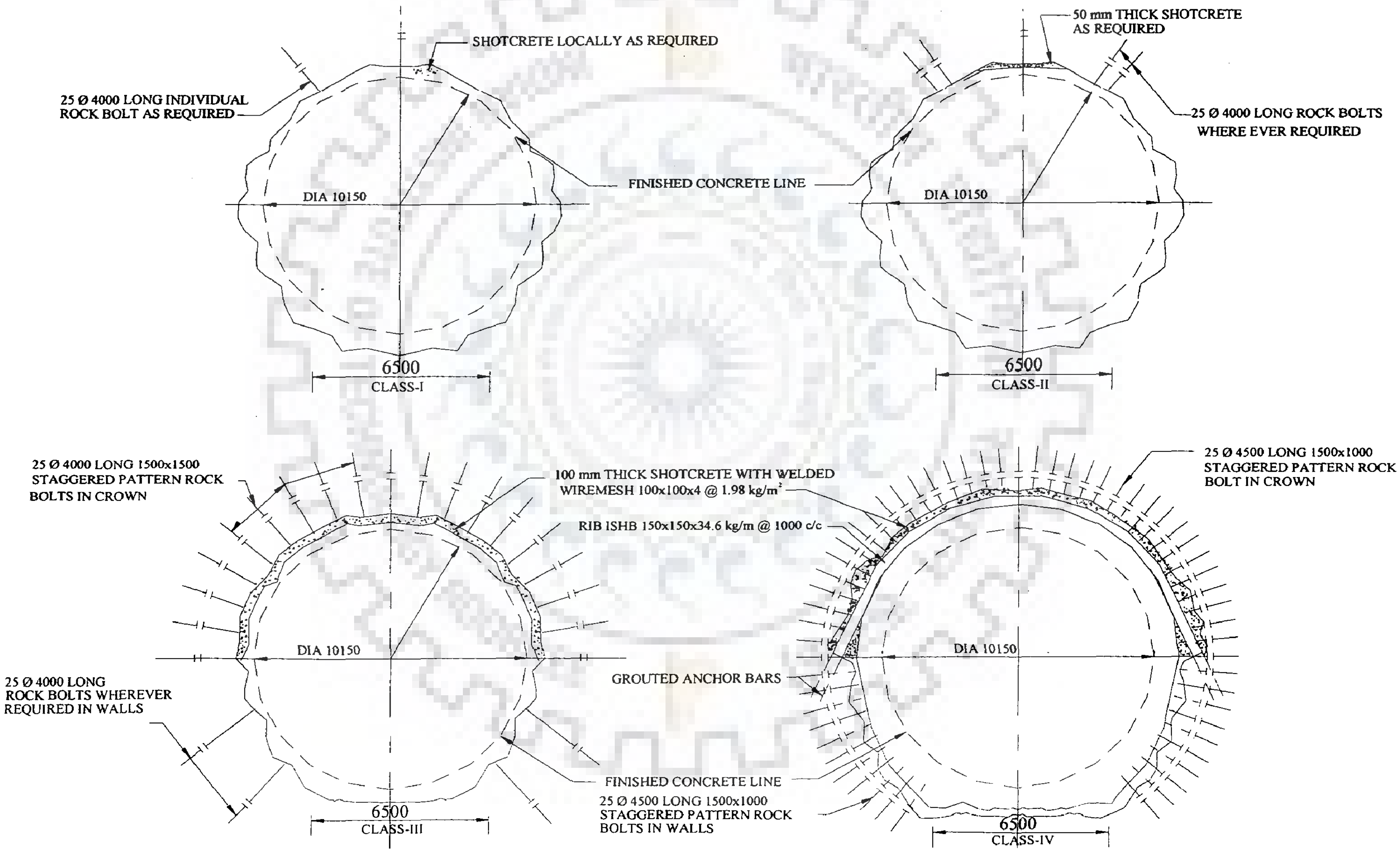


Fig. VIII.1a Supporting system (Class I to IV)

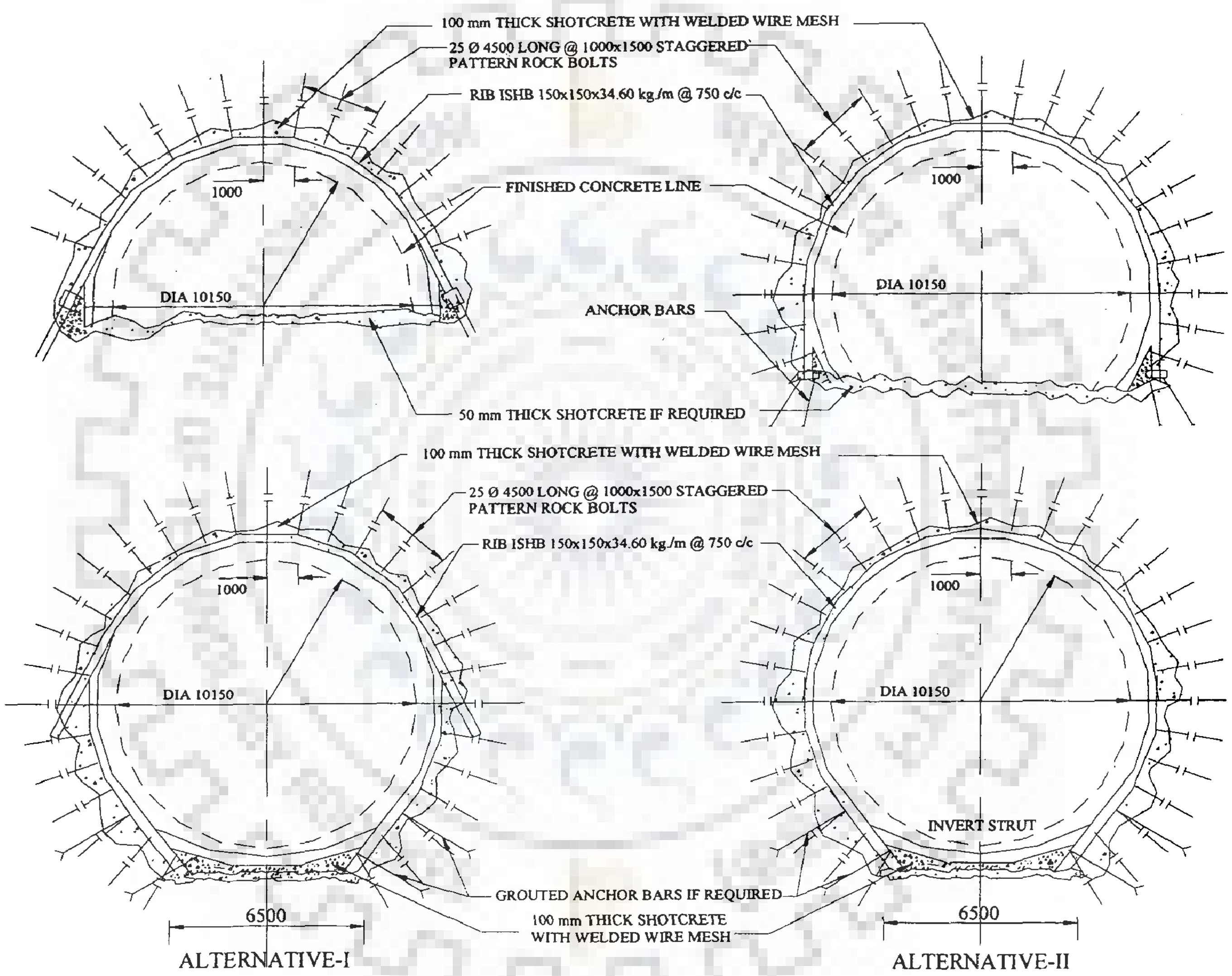


Fig. VIII.1b Supporting system (Heading & Benching) Class V

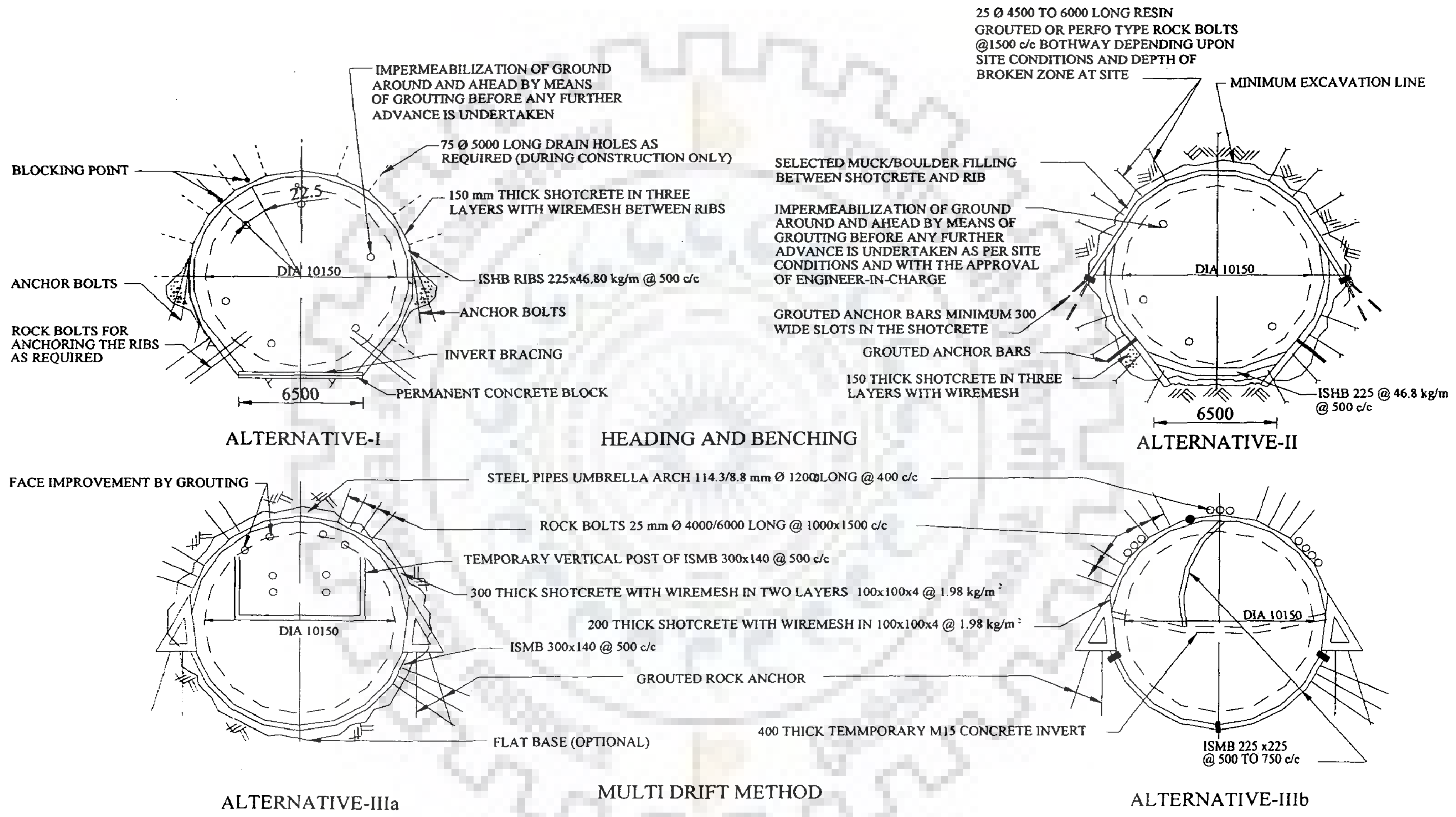


Fig. VIII.1c Supporting system (Class VI)

APPENDIX – IX
(FORMULATIONS FOR SUPPORT REACTION CURVE)

The SRCs have been determined following procedures given by Hoek and Brown (1980a) and Sharma (1995). The initial convergence is denoted by u_{io}.

The stiffness of the support installed within the tunnel is characterized by a stiffness constant k. The radial support pressure p_i provided by the support is give by

$$p_i = k.\frac{u_{ie}}{a} \tag{IX.1}$$

where, u_{ie} is the elastic part of the total deformation u_i.

$$\text{Hence, } u_i = u_{io} + \frac{p_i\, a}{k} \tag{IX.2}$$

This Eq. IX.2 is applicable up to the point where the strength of the support system is reached. The plastic failure of the support system occurs at this point and further deformation occurs at a constant support pressure.

Concrete or Shotcrete Lining

For concrete or a shotcrete lining, placed inside the tunnel, stiffness k_c is given by

$$k_c = \frac{E_c\left[a^2 - (a - t_{sc})^2\right]}{(1+\nu_c)\left[(1-2\nu_c)\,a^2 + (a - t_{sc})^2\right]} \tag{IX.3}$$

The maximum support pressure (p_{scmax}), generated by a shotcrete or concrete, is given by

$$p_{scmax} = \frac{1}{2} q_{c.conc}\left[1 - \frac{(a - t_{sc})^2}{a^2}\right] \tag{IX.4}$$

Blocked Steel Sets

The stiffness of a blocked steel set is defined by

$$\frac{1}{k_s} = \frac{S_{rib}.a}{E_s A_s} + \frac{S_{rib}.a^3}{E_s I_s}\left[\frac{\alpha\,(\alpha + \sin\alpha\cos\alpha)}{2\sin^2\alpha} - 1\right] + \frac{2S_{rib}.\alpha\, t_B}{E_b W^2} \tag{IX.5}$$

The maximum support pressure (p_{ssmax}), which can be accommodated by the steel set is

$$p_{ssmax} = \frac{3A_s I_s \sigma_{ys}}{2S_{rib}.a.\alpha\left[3I_s + XA_s\left\{a - \left(t_B + \frac{1}{2}X\right)\right\}(1-\cos\alpha)\right]} \tag{IX.6}$$

Ungrouted Mechanically / Chemically Anchored Rock Bolts

The support pressure for these rock bolts depend upon the deformation characteristics of the anchor, washer plate and bolt head as well as on the deformation of the bolt shank.

The stiffness of the ungrouted mechanically/chemically anchored rock bolt is given by

$$\frac{1}{k_b} = \frac{S_{bolt}.S_l}{a}\left[\frac{4l_{bolt}}{\pi d_b^2 E_s} + R\right] \tag{IX.7}$$

The maximum support pressure (p_{sbmax}) is given by

$$p_{sbmax} = \frac{T_{bf}}{S_{bolt}.S_l} \tag{IX.8}$$

Combined Support Systems

When two support systems are combined in a single application, it is assumed that the stiffness of the combined support system (k') is equal to the sum of the stiffness of the individual components.

$$k' = k_1 + k_2 \tag{IX.9}$$

where, k_1 is the stiffness of first system and

k_2 is the stiffness of second system.

The maximum deformation of the combined support system will be equal to lesser of the maximum deformation of the individual support systems. Since at this point the remaining support system will be required to carry most of the load and its response will be unpredictable.

APPENDIX – IX
(FORMULATIONS FOR SUPPORT REACTION CURVE)

The SRCs have been determined following procedures given by Hoek and Brown (1980a) and Sharma (1995). The initial convergence is denoted by u_{io}.

The stiffness of the support installed within the tunnel is characterized by a stiffness constant k. The radial support pressure p_i provided by the support is give by

$$p_i = k.\frac{u_{ie}}{a} \tag{IX.1}$$

where, u_{ie} is the elastic part of the total deformation u_i.

Hence, $$u_i = u_{io} + \frac{p_i\, a}{k} \tag{IX.2}$$

This Eq. IX.2 is applicable up to the point where the strength of the support system is reached. The plastic failure of the support system occurs at this point and further deformation occurs at a constant support pressure.

Concrete or Shotcrete Lining

For concrete or a shotcrete lining, placed inside the tunnel, stiffness k_c is given by

$$k_c = \frac{E_c\left[a^2 - (a - t_{sc})^2\right]}{(1 + v_c)\left[(1 - 2v_c)\, a^2 + (a - t_{sc})^2\right]} \tag{IX.3}$$

The maximum support pressure (p_{scmax}), generated by a shotcrete or concrete, is given by

$$p_{scmax} = \frac{1}{2} q_{c.conc}\left[1 - \frac{(a - t_{sc})^2}{a^2}\right] \tag{IX.4}$$

Blocked Steel Sets

The stiffness of a blocked steel set is defined by

$$\frac{1}{k_s} = \frac{S_{rib}.a}{E_s A_s} + \frac{S_{rib}.a^3}{E_s I_s}\left[\frac{\alpha\,(\alpha + \sin\alpha\,\cos\alpha)}{2\sin^2\alpha} - 1\right] + \frac{2S_{rib}.\alpha\, t_B}{E_b W^2} \tag{IX.5}$$

The maximum support pressure (p_{ssmax}), which can be accommodated by the steel set is

$$p_{ssmax} = \frac{3A_s I_s \sigma_{ys}}{2S_{rib}.a.\alpha\left[3I_s + XA_s\left\{a - \left(t_B + \frac{1}{2}X\right)\right\}(1-\cos\alpha)\right]} \tag{IX.6}$$

Ungrouted Mechanically / Chemically Anchored Rock Bolts

The support pressure for these rock bolts depend upon the deformation characteristics of the anchor, washer plate and bolt head as well as on the deformation of the bolt shank.

The stiffness of the ungrouted mechanically/chemically anchored rock bolt is given by

$$\frac{1}{k_b} = \frac{S_{bolt}.S_l}{a}\left[\frac{4l_{bolt}}{\pi d_b^2 E_s} + R\right] \tag{IX.7}$$

The maximum support pressure (p_{sbmax}) is given by

$$p_{sbmax} = \frac{T_{bf}}{S_{bolt}.S_l} \tag{IX.8}$$

Combined Support Systems

When two support systems are combined in a single application, it is assumed that the stiffness of the combined support system (k′) is equal to the sum of the stiffness of the individual components.

$$k' = k_1 + k_2 \tag{IX.9}$$

where, k_1 is the stiffness of first system and

k_2 is the stiffness of second system.

The maximum deformation of the combined support system will be equal to lesser of the maximum deformation of the individual support systems. Since at this point the remaining support system will be required to carry most of the load and its response will be unpredictable.

APPENDIX – IX
(FORMULATIONS FOR SUPPORT REACTION CURVE)

The SRCs have been determined following procedures given by Hoek and Brown (1980a) and Sharma (1995). The initial convergence is denoted by u_{io}.

The stiffness of the support installed within the tunnel is characterized by a stiffness constant k. The radial support pressure p_i provided by the support is give by

$$p_i = k.\frac{u_{ie}}{a} \tag{IX.1}$$

where, u_{ie} is the elastic part of the total deformation u_i.

$$\text{Hence, } u_i = u_{io} + \frac{p_i\, a}{k} \tag{IX.2}$$

This Eq. IX.2 is applicable up to the point where the strength of the support system is reached. The plastic failure of the support system occurs at this point and further deformation occurs at a constant support pressure.

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For concrete or a shotcrete lining, placed inside the tunnel, stiffness k_c is given by

$$k_c = \frac{E_c\left[a^2 - (a - t_{sc})^2\right]}{(1 + v_c)\left[(1 - 2v_c)\, a^2 + (a - t_{sc})^2\right]} \tag{IX.3}$$

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The maximum support pressure (p_{ssmax}), which can be accommodated by the steel set is

$$p_{ssmax} = \frac{3A_s I_s \sigma_{ys}}{2S_{rib}.a.\alpha\left[3I_s + XA_s\left\{a - \left(t_B + \frac{1}{2}X\right)\right\}(1-\cos\alpha)\right]} \qquad \text{(IX.6)}$$

Ungrouted Mechanically / Chemically Anchored Rock Bolts

The support pressure for these rock bolts depend upon the deformation characteristics of the anchor, washer plate and bolt head as well as on the deformation of the bolt shank.

The stiffness of the ungrouted mechanically/chemically anchored rock bolt is given by

$$\frac{1}{k_b} = \frac{S_{bolt}.S_l}{a}\left[\frac{4l_{bolt}}{\pi d_b^2 E_s} + R\right] \qquad \text{(IX.7)}$$

The maximum support pressure (p_{sbmax}) is given by

$$p_{sbmax} = \frac{T_{bf}}{S_{bolt}.S_l} \qquad \text{(IX.8)}$$

Combined Support Systems

When two support systems are combined in a single application, it is assumed that the stiffness of the combined support system (k′) is equal to the sum of the stiffness of the individual components.

$$k' = k_1 + k_2 \qquad \text{(IX.9)}$$

where, k_1 is the stiffness of first system and

k_2 is the stiffness of second system.

The maximum deformation of the combined support system will be equal to lesser of the maximum deformation of the individual support systems. Since at this point the remaining support system will be required to carry most of the load and its response will be unpredictable.