5. Geotechnical problems and treatment of underground structures

5.1 Location and layout

The Sardar Sarovar underground powerhouse is located at 160m downstream of the dam on the Right Bank in basalt flows intruded by dolerite dykes and sill. The location of powerhouse cavern has been selected in between vertical and inclined dolerite dykes so as they would act as seepage barrier for Rock fill dam-I reservoir water in the north and for Main River water in the south, respectively (Fig. 28 & 29).

The machine hall (cavern) is oriented in N10°E -S10° W direction i.e. longer axis of the powerhouse cavern is nearly aligned parallel to the direction of maximum horizontal stress ±N5°E. Major shear zones are cutting across the longer axis of the cavern. Tunnels (Access tunnel and draft tube tunnels) are aligned across the major discontinuities except exit tunnels, which are aligned, parallel to Akkalbar fault (Fig. 28).

5.2 Salient features

The size of the underground powerhouse (Cavern) is 23 m (wide) x 58m (high) x 212m (long) and installed capacity is1200 MW (6 x 200 MW). The height of the underground powerhouse is about 58m. The crown level of the powerhouse is at El.45m and bottom level at El. –12.6m. It will operate with head varying from 77.5 m to 117.8m.Other components include bus shafts and bus galleries, lift well, control room, ventilation shaft and ventilation room. As the cavern will be surrounded by water, a drainage and grouting gallery is provided around the cavern with an average El. of 36.0m.

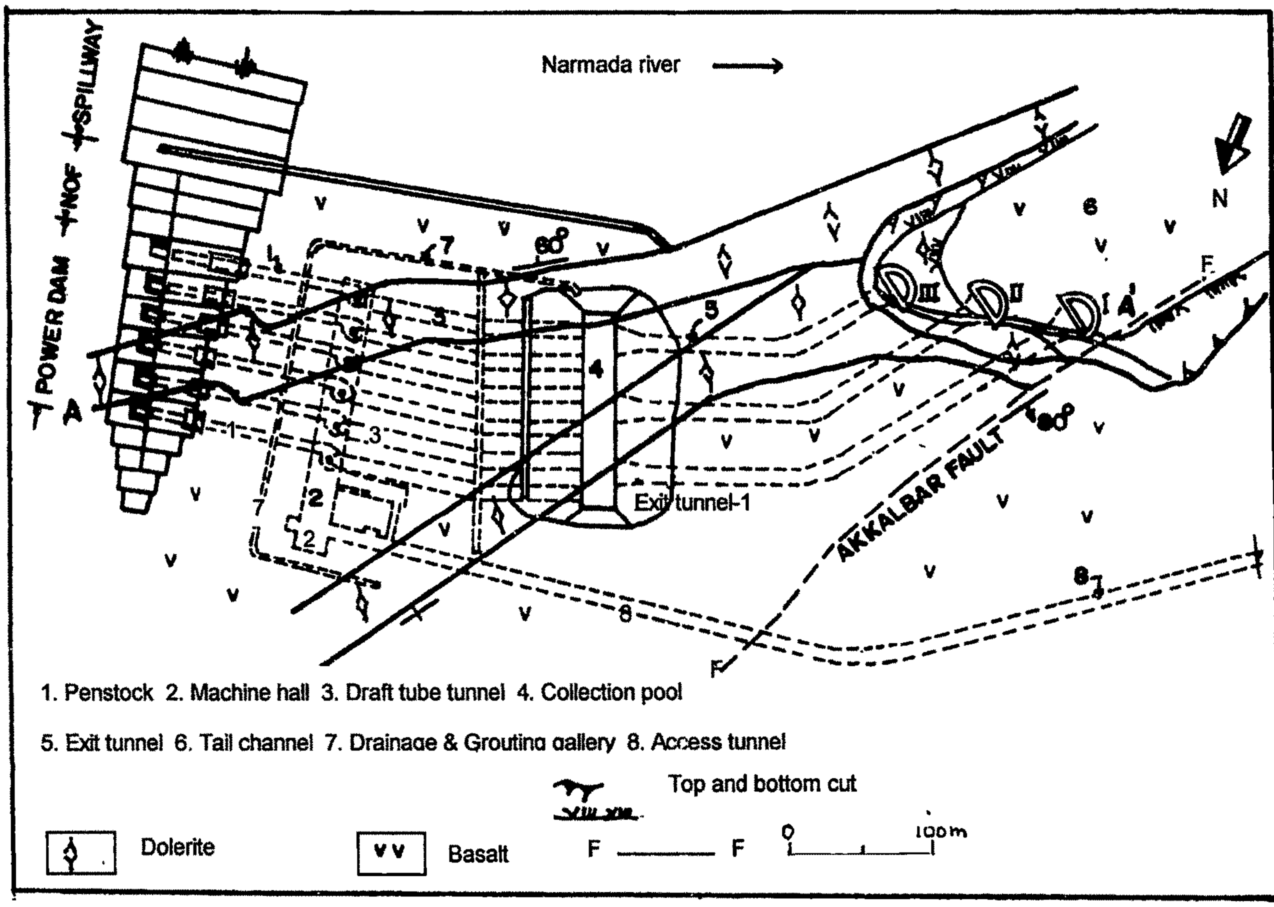
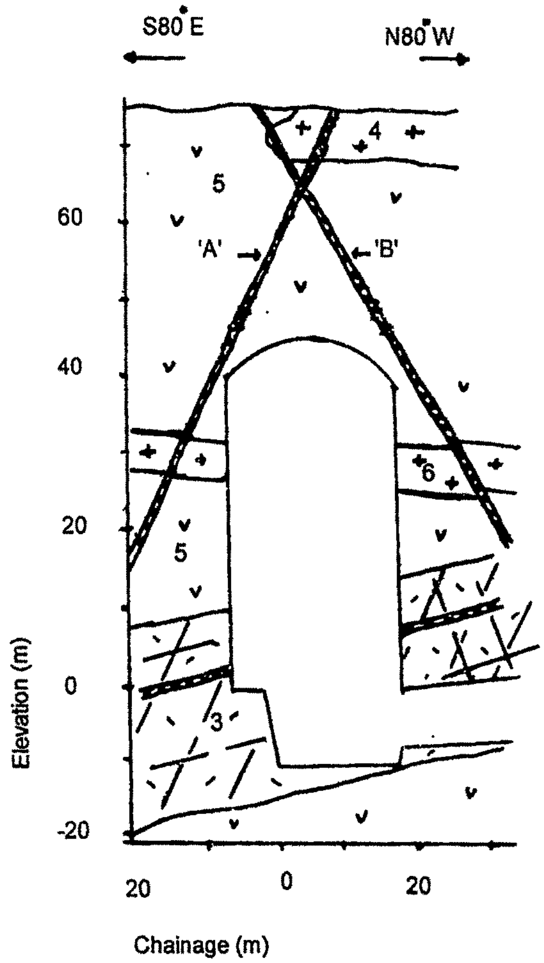
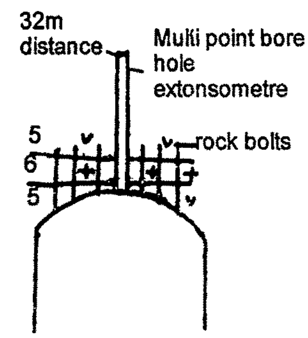


Fig. 28: Geology and layout of underground powerhouse

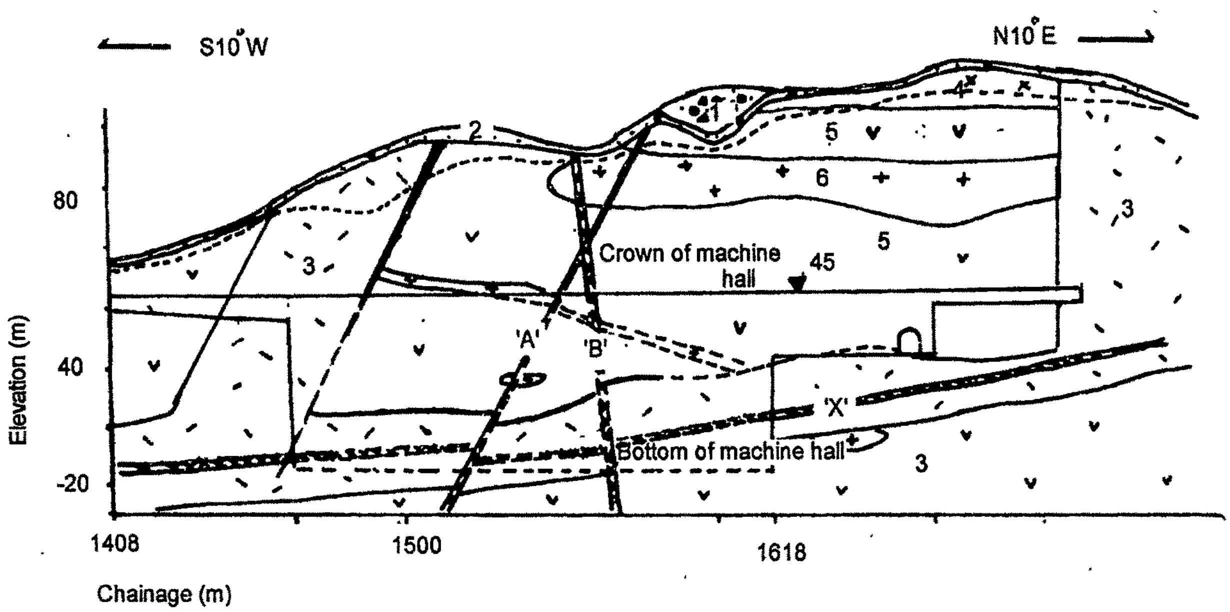


I. Geological cross-section of machine hall



II. section showing spot rock bolts for agglomerate layer

1. Fill material 2. Over-burden 3. Dolerite 4. Amygdaloidal basalt 5. Porphyritic basalt 6. Agglomerate



II. Geological longitudinal section of the machine hall

Fig. 29: Geological sections of the machine hall, Sardar Sarovar (Narmada) Project

The D-shaped, 860m long access tunnel (8.5m X 9m) provides the principal access to the powerhouse. The intake arrangement of powerhouse comprises six inclined steel lined penstocks of 7.61m internal diameter (9.0m excavated diameter). The tailrace system comprises six draft tube tunnels of 10m finished diameter (10.5m excavated) tailing into an open collection pool (surge pool) to accommodate surges during various operating conditions. Three exit tunnels of 12m, finished diameter (excavated 13m), horse shoe shaped, off take from the collection pool to connect them to the open tail pool and the tail race channel to join the parent river about 730m downstream of the dam axis.

The powerhouse is having reversible pump turbines. A concrete wier of 30m (maximum) height is proposed to be constructed at a distance of 12 km on the downstream of main dam to create tail pond for the operation of reversible turbine.

5.3 Machine hall

The machine hall (powerhouse cavern) is having shallow rock cover (varying from 35m to 60m) consisting of basalt flows. Geotechnical assessment of the machine hall is given below:

5.3.1 Geology and rock mass characteristics

The rocks in which cavern is excavated consists of sub-horizontal flows of amygdaloidal and porphyritic basalt separated by pockets of agglomerate and intruded by ENE-WSW trending, 25 to 30m thick, vertical and inclined (60°-65° towards SSE) dolerite dykes and low dipping (20° -25° SE) dolerite sill aligned in NE-SW direction (Fig.29). All of these rocks are strong (>60 MPa compressive strength) but are well jointed with typical block size of approximately 1 to 2 cubic metre. Flows are sub-horizontal and there are three well-developed joint sets (NNW/60°-vertical, ENE/60°-vertical and ENE/30°-45° NW) and some random joints (Fig. 30).

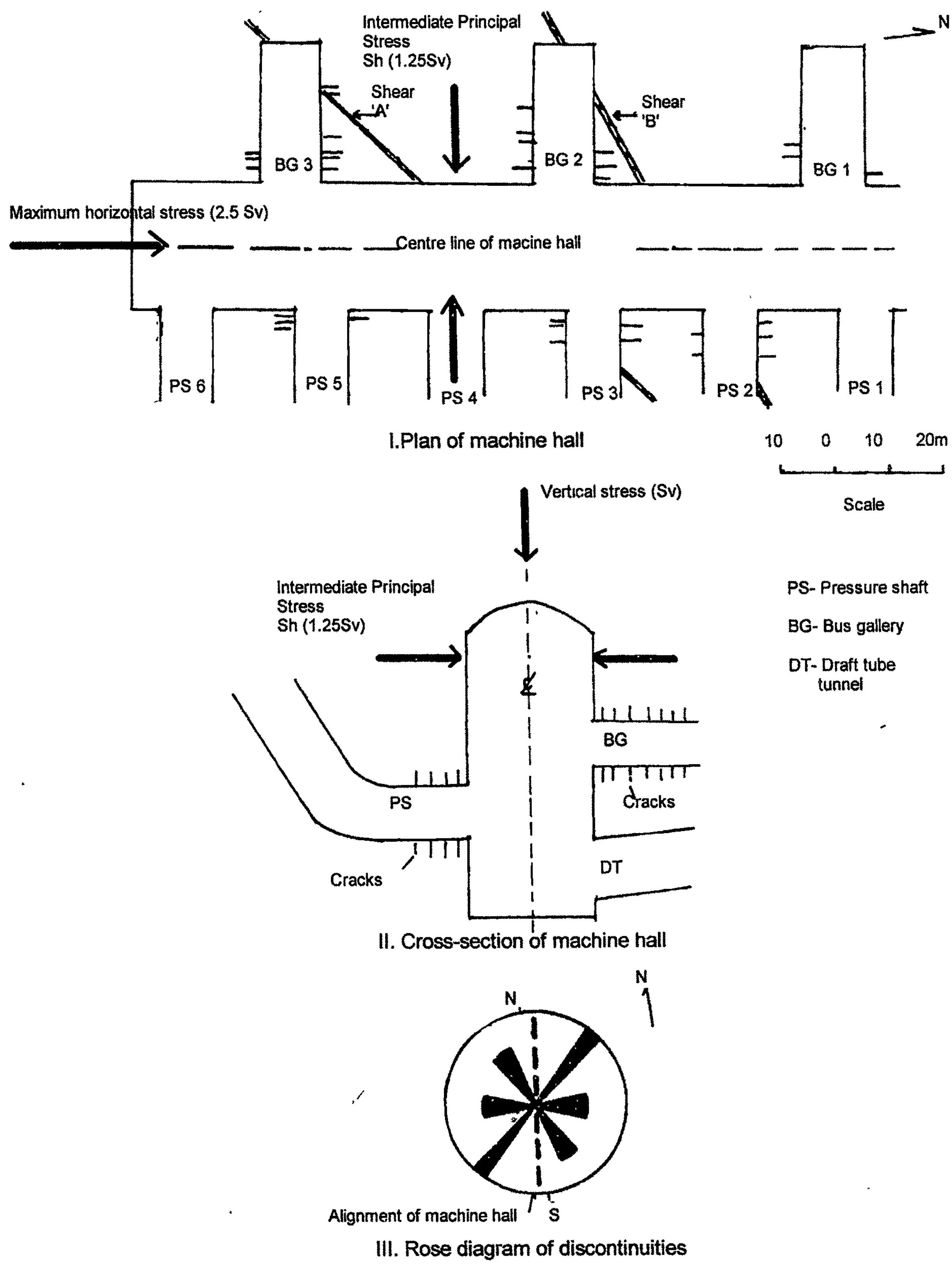


Fig. 30: Plan and section of machine hall of underground powerhouse showing disposition of cracks in the Pressure shafts and Bus galleries

Rock mass inside the underground structure is in general fresh to slightly weathered. Basalt and dolerite above machine hall are weathered about 0.5 to 5m and 7 to 23m in depth, respectively from the surface. Rock mass in the powerhouse is of poor to good category (Table 29).

Table 29: Rock mass description and classification of underground powerhouse

Rock mass description	Rock mass characteristic	Barton's Q	RMR Rating (Bieniawiski 1976)	Class No.	Class Description
Jointed basalt	Three joint set plus random, rough or irregular planar joints, non-softening infilling, dry rock mass, medium stress.	9.16	60	III	Fair rock
Jointed inclined dolerite	-do-	10.00	63	II	Good rock
Jointed Vertical dolerite rock	Moderately altered dolerite, Joints infilled with chloritic material	1.5	45	III	Moderately to highly altered dolerite (slaking of the rock observed on exposure to air)
Jointed dolerite sill	Moderately to highly altered dolerite (slaking of the rock observed on exposure to air)	0.6 to 1.25	30 to 40	IV	Poor rock
Shear zone	Sandy, gravely crushed zone thick enough to prevent rock wall contact, softening or low friction clay mineral coating, dry rock mass, single shear zone containing clay or disintegrated rock.	1.25	35	IV	Poor rock

Major discontinuities traversing the machine hall are viz. sheared contact of the inclined dolerite dyke with host rocks (basalt and dolerite sill) designated as main shear, ramified steeply dipping shear zones 'A' (N50° E-S50° W/ 70° S40° E) and

'B' (N70° E-S70° W/ 70° -80° N20° W). Low dipping (20° -25° SE) shears 'X', 'Y' and 'Z' are traversing dolerite sill (Fig. 31 & 32)).

Dolerite dyke (Vertical) and sill are dissected by chlorite-coated joints, shears and slaked zones. The dolerite sill is forming foundations of major part of the turbo-generator units.

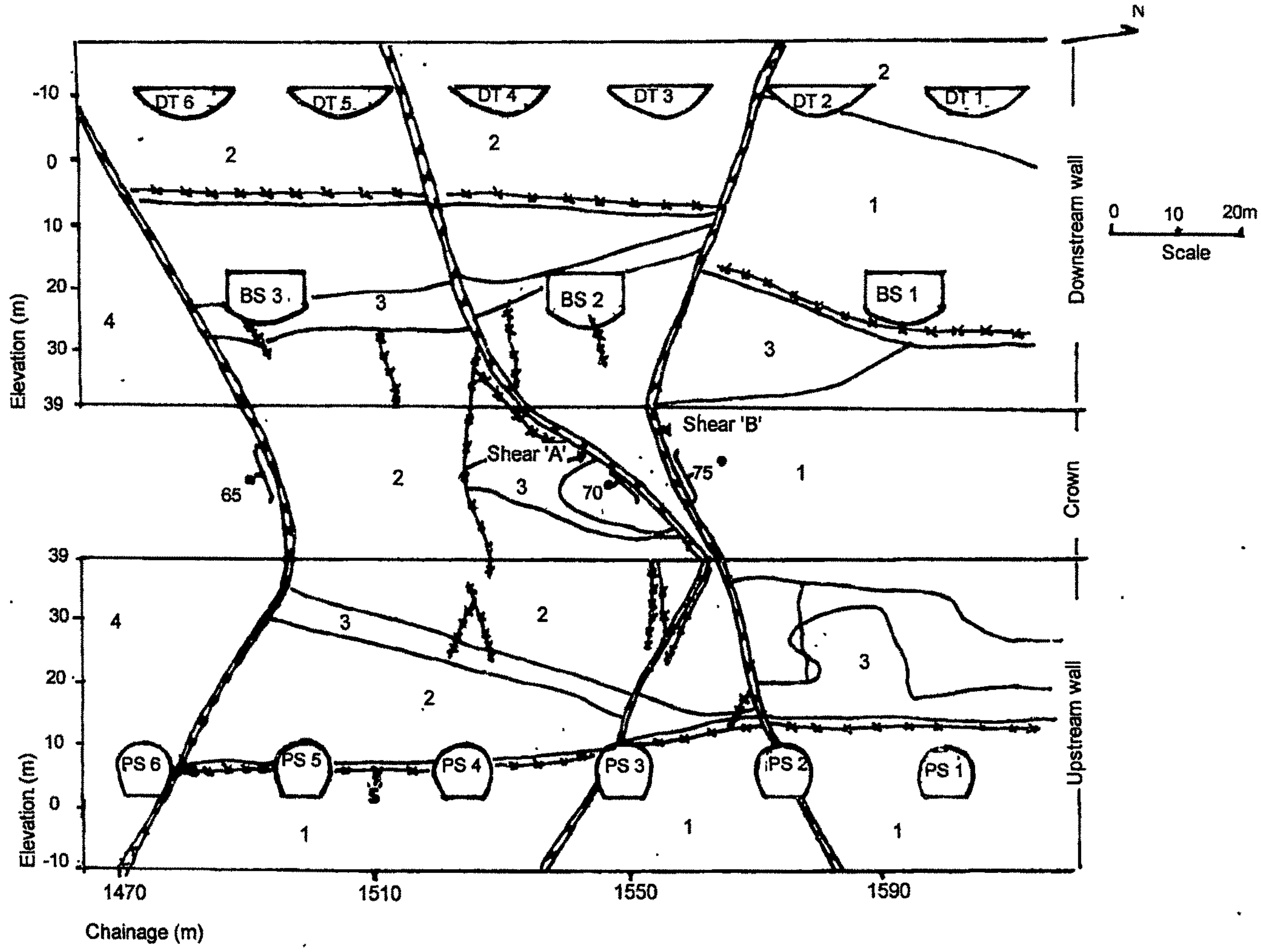
In-situ tests indicted large variations of rock mass permeability from 0-30 lugeons. Major part of the underground rock mass is free from water seepage. Oozing and water dripping is confined in the reaches affected by shear zones/ fault.

5.3.2 Design support

The original support provided for this cavern consisted of pattern rock bolts and two 38mm layers of shotcrete with a sandwiched layer of welded wire mesh. Roof supports included tensioned rock bolts of 25mm dia., 6m long and 1.75m center to center (c/c) pretension to 14 tonnes load and two layers of 38mm thick shotcrete with wire mesh in between. Wall supports include tensioned rock bolts of 25mm dia., 6m long and 2.5m c/c and two layers of 38mm thick shotcrete with wire mesh in between. In the middle third height of the wall (El. 13 to 33m), additional rock bolts of 7.5m length are added to make the overall spacing of 1.52m c/c. These supports were designed by the Central Water Commission, New Delhi (Divatia and Trivedi 1990).

5.3.3 Geotechnical problems

Machine hall and other underground structures were excavated by heading and benching method by adopting New Austrian tunneling method (NATM). The basic principal of NATM is to utilise rock itself as structural material. Six numbers of cross drifts from the central exploratory drifts were excavated from El. 45m to



3-D Geological log of the machine hall

	Bieniawiski's RMR	Barton's Q
1. Jointed/ sheared dolerite sill	30-40	0.6-1.25
2. Jointed basalt	60	9.16
3. Agglomerate	50	-
4. Jointed Inclined dolerite dyke	63	10
5. Shear zone	35	1.25

PS Pressure shaft
BS Bus gallery
DT Draft tube tunnel

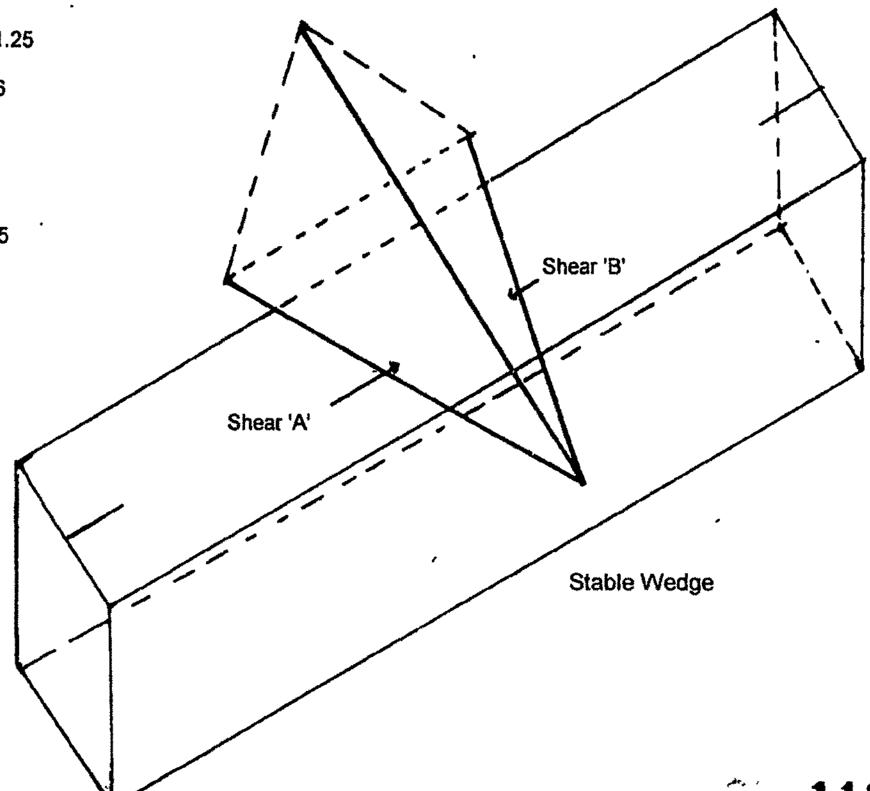
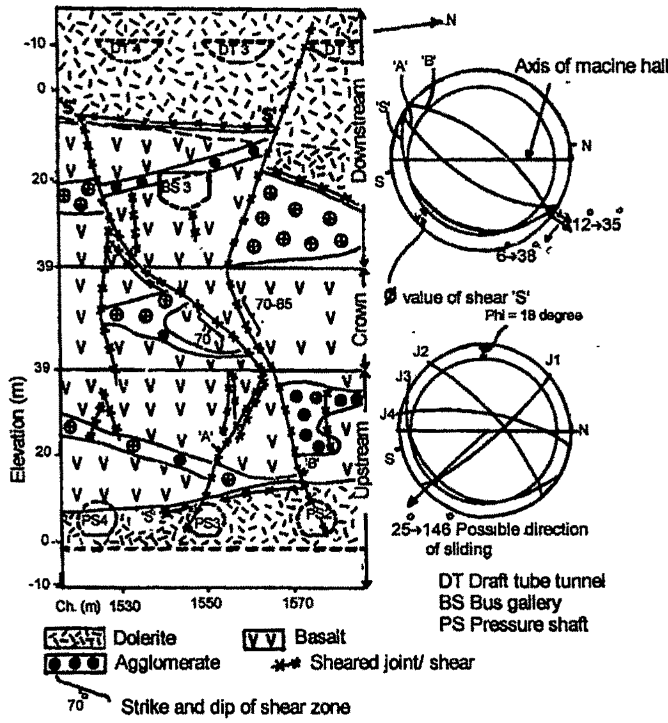
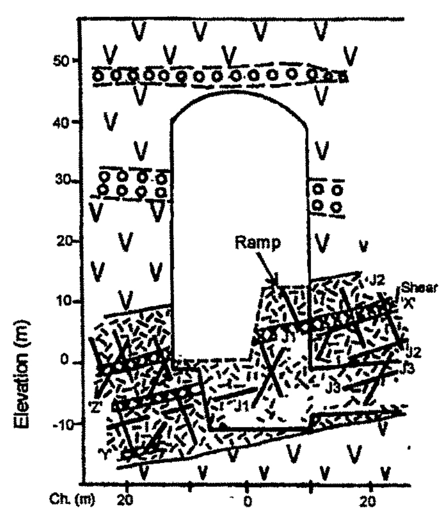


Fig. 31: Rock mass characteristics and disposition of shear zones in the machine hall



I. 3-D Geological map of part of machine hall



II. Geological cross section of macine hall

Fig. 32: Geological map of part of machine hall showing disposition of major discontinuities

39m. After assessing and observing the behaviour of rock mass, powerhouse cavern was excavated to 5m, 9m and to its full width of 23m. The bench height varied from 2.5 to 4.0m.

Main geotechnical problem observed in the machine hall was the development of cracks on the shotcrete of upstream and downstream walls as well as inside the walls including pressure shafts and bus galleries. Minor rock falls in the crown were also observed during the excavation (Appendix-I and VI).

a. Rock falls in the crown: Rock fall occurred in February 1988 between R.D. 1540 and 1556m involving about 125 cubic meter of the rock mass. Three point bore hole extensometer installed at R.D. 1504m to study the behaviour of contact of agglomerate with basalt revealed opening of contact at a very small but constant rate of 0.024 mm/ month prior to the rock fall. Total opening prior to the rock fall was 3.03 mm during the period August 1984 to February 1988 (Goel and Jethwa, 1991). Overbreaks of the order of 1.5 to 2m occurred in the upstream of the roof arch between shear zones 'A' and 'B'. As a remedial measure additional rock bolts in between the pattern rock bolts were provided besides two additional shotcrete layers. No further opening of the contact and rock fall occurred since then in the treated area.

b. Development of Cracks (fissures) in the upstream and downstream walls: The excavation of the roof with pattern rock bolt support system was completed in December 1989. The excavation of the walls was completed upto El. 20m by January 1992.The crown level of the cavern is at El. 45m and bottom level at El. (-) 12.6m. Further benching was taken up from the ramp along downstream wall approximately half the width of the cavern from service bay end (El. 20.0m) to riverside end of the cavern (El. 4.0m). The benching in the remaining half width along upstream wall completed with the pattern rock bolt support system upto El. (-) 1.9m by June 92. The excavation of six numbers of pressure shafts on upstream wall was also completed.

Cracks in the upstream wall were observed between Ch. 1545 and 1585m below El. 14.0m. These cracks were stitched by 4 to 6m long inclined criss-cross rock bolts. Additional layer of shotcrete with wiremesh was provided in this reach. These Cracks (fissures) were observed propagated upto El. 13.50m and El. 36m respectively, when bench excavation reached upto El.10m. The fissures already treated earlier and covered with shotcrete reappeared when bench excavation reached to El. 1.9m besides additional peripheral fissures were observed around pressure shafts 2, 3 and 5. Fissures were also observed on the downstream wall between El.39m (i.e. spring level) and El. 9.0m extending inside the bus galleries. Popping of shotcrete between ch.1505m and ch.1520m and between El. 9.0m and El. 27.0m along shear zone 'A' was noticed.

5.3.4 Nature of cracks in the upstream and downstream walls

Cracks (fissures) developed in the pressure shafts and bus galleries are aligned parallel to the longer axis of the machine hall (Fig. 30). These cracks are not following geological discontinuities. Sub-horizontal to low dipping cracks developed in the downstream wall in en echelon pattern parallel to then excavated profile of the ramp. A few cracks were also observed near the major shear zones 'A' and 'B'. These cracks were observed extending inside the rock mass by opening windows in the shotcrete.

5.3.5 Three Dimension Numerical (FEM and DEC) Analysis

Three Dimension Finite element (3-D FEM) and Three Dimensional Distinct Element Code (3-DEC) back analysis were conducted after the development of cracks in the walls of the machine hall to know the present and future behaviour of the underground powerhouse cavern (Fig. 33)(Table 30).

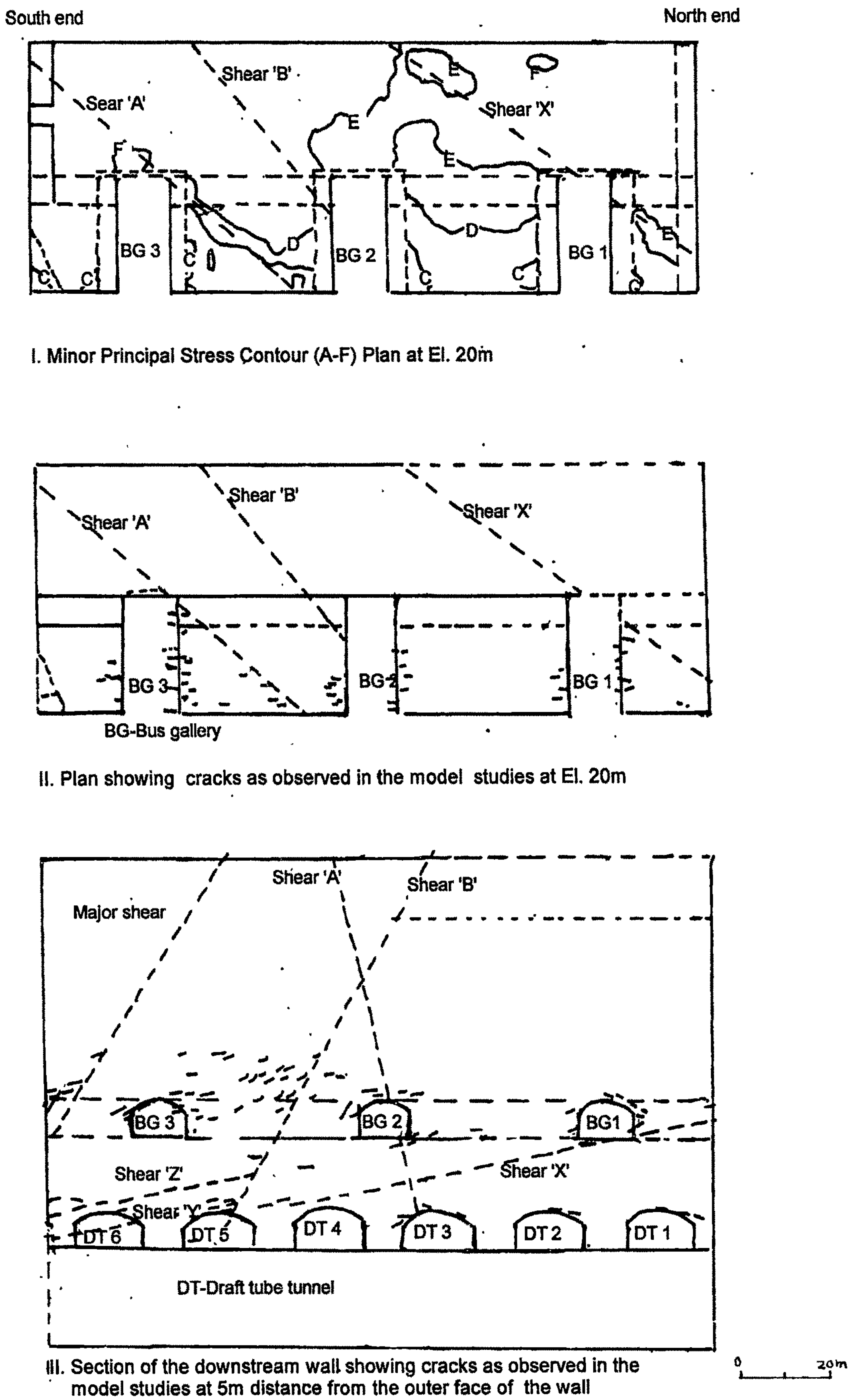


Fig. 33: Plot of 3-DEC (3-Dimensional Distinct Element Code) Discontinuum analysis model study results of downstream wall of Powerhouse cavern (Machine hall) prior to the excavation of ramp

Table 30: Result of 3-D FEM and 3-DEC analysis

Displacement/ Stresses/ FOS	3-D FEM analysis	3-DEC analysis
Displacement	The maximum displacement on the wall was 7.4mm without dam loading and 7.6mm with dam loading. Displacement of upstream wall was less in comparison to downstream wall without dam loading while displacements of both the walls became almost equal with dam loading.	Rock mass movement is continuous in the continuum analysis. The horizontal displacement contours, compared to continuum analysis, are not symmetric about the longitudinal axis of cavern and the continuity of contours is disturbed at the shear zones. The contour shows the movement of wall towards the cavern. The shear movements primarily along shear zone "A" and "B" are higher near the excavation face and reduce inside the rock mass. The shear movements is more pronounced on the upstream wall relatively at higher elevations i.e. El.35, 30 and 25m whereas on the downstream wall more movement is at lower elevations i.e. El.20 and 15m. Displacement of the walls as observed(at El. 20m): 1. During continuum analysis 1.6 to 1.8 cm with ramp and 1.9 to 2.2cm without ramp. 2. During discontinuous analysis 2.75 cm with ramp and 3.25 cm without ramp.
Stresses: Major principal stress	The major principal stress 11.5 MPa (115kg/cm^2) occurred at El. 6.4m at section through pressure shaft.	The major principal stresses are oriented along the longitudinal axis of the cavern. Major principal stress contours show that the stress concentration area lies at the junctions of bus galleries and pressure shafts with powerhouse cavern.
Minor principal stress	A tensile minor stress zone developed at sections through rock pillar between pressure shafts at upstream wall but stresses in the downstream wall were all compressive. The maximum depth of the tensile zone is 6.5m. The direction of the minor principal stress is nearly right angle to the wall.	The minimum principal stress contours show the tensile regions at the junction between the bus galleries and pressure shafts with cavern. Higher magnitude of tensile stress observed upto a distance of 10m in the pressure shafts, 12m to 20m in the bus galleries. The rock mass in the bus gallery-3 is affected most where shear zone A is forming wedge with northern side of the gallery wall where ramp support is almost negligible.
Factor of safety (FOS)	The dam loading has little influence on the factor of safety contours. The maximum depth of the 1.0 FOS contour is 10m in the upstream wall and 6.5m in the downstream wall. The factor of safety of the pillars in between Draft Tube Tunnels is sufficient but between the pressure shafts it is less than the 1.5. In the area affected by cracks safety factor contour of 1.5 extends upto 25m in depth.	Contour of FOS of 1.5 in general is about 16 to 17m away from the face of the cavern wall inside the rock mass. However, around bus gallery-3 it is at a distance of 20m where maximum displacement has been observed.

5.3.6 Geological analysis of the prominent discontinuities and stresses

The geological stability analysis was done to find out the reasons for the development of cracks in the machine hall.

a. Wedge failure analysis: The most common types of failure in jointed rock masses at relatively shallow depth are those involving wedges falling from the roof or sliding out of the side walls of the openings (Stacey and Page 1986). Major shear zones 'A' and 'B' traversing the machine hall are forming stable wedge in the upstream wall as the plunge of the intersection of these shear zones is about 22° towards northeast (i.e. inside upstream wall). Sliding of wedge can only occur along the line of intersection of two planar discontinuities if it "day light" in the open space (i.e. free face). In the downstream wall these shears are diverging by virtue of their orientation and thus they are also not creating any problem of wedging (Fig. 31 and 32).

Sliding wedges are formed at the lower part of downstream wall of the machine hall in the dolerite sill with intersection of joints J_1 and J_3. These wedges are having 25° plunge towards S34° E (i.e. towards free face) (Fig. 32). Similar minor rock wedges are formed with the intersection of joints J_1 and low dipping shears 'X', 'Y' and 'Z'. These wedges were stitched by 12m long rock bolts during progressive excavation.

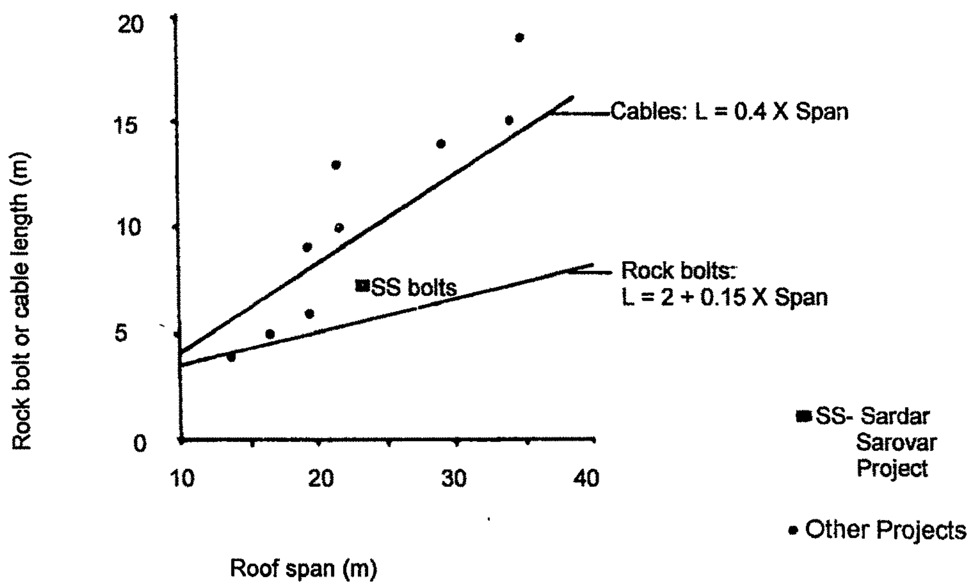
b. Plane failure analysis: One of the primary condition of the plane failure is that the plane on which sliding is to occur must strike parallel or nearly parallel (within approximately $\pm$ 20°) to the slope face (Hoek and Brown 1980). In the machine hall, prominent shear zones and joints are striking at an angle more than 30° to the longer axis of the cavern and thus posing no problem of plane failure (Fig. 30).

c. In-situ stresses: In-situ horizontal stresses in the machine hall perpendicular to the longer axis of the cavern is low (1.5 MPa). The average compressive strength of the rocks surrounding powerhouse cavern is more than 60 MPa. Therefore, possibility of development of cracks due to in-situ stresses is not there.

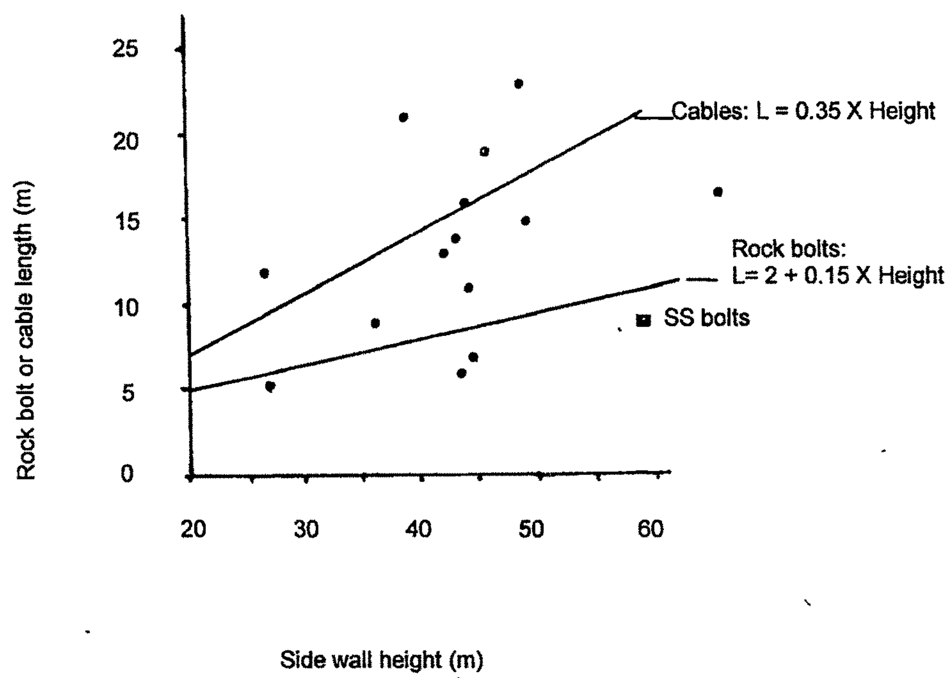
5.3.7 Review of Supports

Review of the design supports from the various approaches (empirical approaches of Cording et. al. (1971), United States Corps of Engineers (1980), Hoek and Brown (1980), Barton et.al. (1980)) and plot of rock bolt and cable lengths for arch support in various hydroelectric projects around the world indicated that the 6m long rock bolts used in the arch of the Sardar Sarovar cavern fall within the precedent range (Fig. 34). Thus they are considered adequate for permanent arch support. None of the above approaches except Barton's method provides criterion for estimating support pressure. The available roof support capacity of 1.19 kg/cm^2 in the jointed basalt and the shear zone and 1.06 kg/cm^2 in the jointed dolerite of the cavern against the estimated ultimate roof support pressure (p_v) of 0.88 kg/cm^{2}in the shear zone, 0.73 kg/cm^2 in jointed basalt and 0.69 kg/cm^2 in jointed dolerite show that the roof support is adequate (Goel and Jethwa 1992). Performance monitoring for fourteen years also established that the roof of the cavern has remained stable.

Similar approaches and plot for side wall support for a 57-58m high cavern gives the average length for rock bolts and cable 10-11m and 20m, respectively (Fig. 34). It appears that the 6 to 7.5m long rock bolts provided for the side walls of the Sardar Sarovar powerhouse cavern were too short (Goel and Jethwa 1992, Prakash and Srikarni 1998). Thus, they could not provide adequate restraint to prevent development of cracks in both upstream and downstream walls including pressure shafts and bus galleries.



I. Roof Support



II. Wall Support

Fig. 34: Plot of rock bolt and cable lengths for roof (arch) and side wall support of underground powerhouse cavern

5.3.8 Mechanism of the rock mass behaviour and development of cracks

Major problem associated with excavation of this powerhouse cavern, in jointed rock mass having shallow rock cover, is that of *stress relief* due to low confining stress. This is analogous to the situation, which arises when excavating very steep slopes in hard but jointed rock masses. The stress relief caused by removal of the 'cut' can induce movement and/or failure in the rock mass. The vertical cavern walls can be thought of as steep rock slopes and unless adequate support would have been provided during excavation, deformation and cracking would occur. Thus in the absence of adequate supports, vertical tension cracks (which are very common in steep rock slopes) can form parallel to the walls. Symmetry of pattern of cracks parallel to the longer axis of the cavern in the pressure shafts and bus galleries also suggest that these cracks are developed due to tensile stresses acting on inadequately supported rock mass (Fig. 30).

Increase in the shotcrete cracking in both the upstream and downstream walls at places was observed during the installation of additional supports. This cracking in the shotcrete was probably manifestation of gradual adjustment of loosened rock mass due to earlier inadequate supports. It is expected that rate of change of deformation of the rock mass would become negligible after the placement of concrete in the foundation of turbo-generator units and installation of adequate wall supports. Numerical model studies indicated that changes of stresses induced by the raising of the reservoir level (from present level El. 80m to full reservoir level El. 140m) would not be very significant.

5.3.9 Remedial measures adopted to stabilise the rock mass in the machine hall

The remedial support in the upstream wall consisted of 10.5 to 32m long 80-ton capacity cables tensioned to 50 tons and then fully grouted. In addition, 12m long 32mm diameter rock bolts, tensioned to 20 tons, were installed at various locations. In the downstream wall, a large number of 12m long 32mm-diameter rock bolts, tensioned to 20 tons before grouting, were installed. These cables were tensioned to 5 tons before grouting. Remaining excavation in the lower part of the cavern was done by providing 12m long tensioned rock bolt support. Low pressure grouting has been done in the upstream and downstream walls to stabilise the loosened rock mass.

5.4 Tunnels

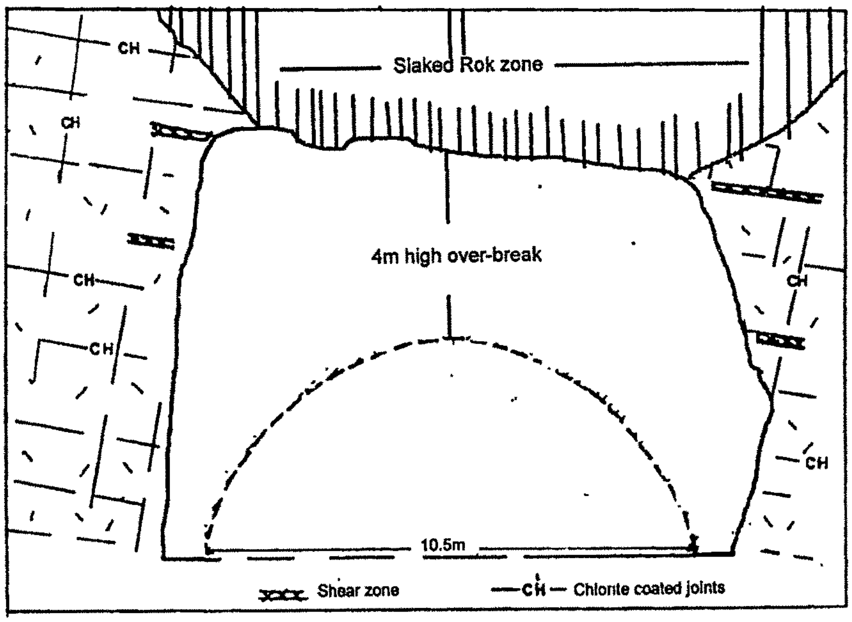
The tunnel excavation has been done by New Austrian Tunneling Method (NATM) to utilise the rock itself as principal structural material (Subramanian et. al. 1992). Major parts of the draft tube tunnels and exit tunnels and small part of the access tunnel are passing through chloritised and slaked jointed dolerite dyke and sill posing problems of rock falls and roof collapses (Prakash 1994) (Fig. 35).

The dolerite sill and vertical dolerite dyke are dissected by three sets of chlorite coated joints. Steeply dipping to vertical joints are prominent in the vertical dyke and sub-horizontal joints in the dolerite sill. Joints in the doleite sill are closely to moderately spaced and in the dolerite sill moderately to widely spaced. Most of the joints are slickensided. Sub-horizontal to low dipping shears are traversing the dolerite sill (Appendix-VII).

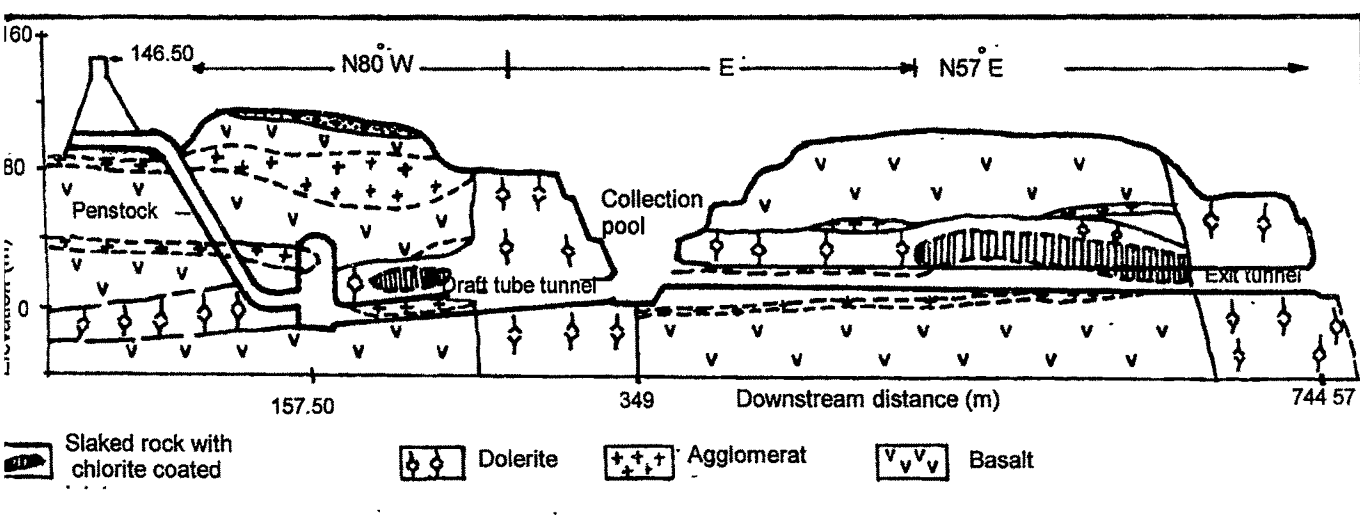
RMR value of dolerite rock dissected by chlorite coated joints varies from 30 to 45 and Q value from 0.6 to 1.5 (Table 29).

5.4.1 Design support

The initial support system comprised 25mm diameter, 4 to 6m long pattern rock bolts at 1.75m c/c with two layers of 38mm thick shotcrete with wire mesh in between. This support system was designed by Central Water Commission, New Delhi (Divatia and Trivedi 1990).



I. Geological cross-section of part of draft tube tunnel-2 showing over-break



II. Geological section along water conductor system

Fig. 35: Geological conditions of water conductor system of underground powerhouse

Dolerite rocks traversing the tunnels are closely to highly jointed. In highly discontinuous rock formations, the validity of empirical design methods based upon general rock classification is questionable (Goodman and Hatzor 1990).

5.4.2 Geotechnical problems and treatment

Geotechnical problems and treatment of the access tunnels, draft tube tunnels and exit tunnels are summarised below:

a. Access Tunnel: Roof of the tunnel passing through dolerite sill in 67m length became flat due to presence of sub-horizontal shear near the crown. Step like profile at haunches is formed due to intersection of vertical and sub-horizontal joints. As a remedial measure spacing of the pattern rock bolt was reduced from 1.5 to 0.75m in this reach. Tunnel section occupied by chlorite coated, closely spaced vertical dolerite dyke in 10m length was provided rib supports.

b. Draft Tube Tunnels: Overbreaks of the order of 4.5m in height occurred in the draft tube tunnel-2 & 3 in the reaches occupied by sub-horizontal shears and slaked rock zones even after the installation of pattern rock bolt supports (Fig. 35). Rib supports were installed in all the tunnels after the roof falls and collapses except in selected reaches (about 80m length in the D.T.1 and 14m length in D.T.2) where chlorite coating along joints was almost negligible.

c. Exit tunnels: Adverse effect of the Akkalbar fault was noticed in the exit tunnel-1 where it is running parallel and close to the alignment in about 200m length (Plate 21 & Fig.26). Joints sympathetic to the fault in the E.T.-1 are forming removable/ detachable blocks of size varying from $1m^3$ to $6m^3$ resulting in major block falls and roof collapses. The rock falls exposed the basic characteristic of crumbling of the rock mass due to chlorite infilled joints and slaking nature of dolerite rock (Plate 22 & 23). About 50% length of the all the exit tunnels is passing through dolerite rock/ dissected by chlorite coated joints and about 50%

(a) Rock mass condition at the outlet portal face of the tunnels

(b) Shotcreted (with wire mesh) outlet portal face of the tunnels

Plate 21: Outlet portal face of the Exit Tunnels prior and after shotcreting the surface, Sardar Sarovar (Narmada) Project (a & b)

(a) Partly slaked vertical dolerite dyke inside the tunnel

(b) Slaked dolerite sill at the outlet portal site of the tunnel

Plate 22: Slaked nature of vertical dolerite dyke and sill in and around the Exit Tunnel-1, Sardar Sarovar (Narmada) Project (a & b)

Plate 23: Slaking of dolerite rock at the portal site of Exit Tunnel, Sardar Sarovar (Narmada) Project

rock bolts were noticed slipped in these zones during tensioning (Fig. 35). Collapses in part of the tunnel sections traversed by chlorite coated joints and slaked rock zones occurred despite installation of design rock bolt supports. Even change of the excavation pattern by excavating the central 7m wide portion in heading and then excavating the remaining portion after providing shotcrete and longer rock bolts in the central drift could not prevent the rock fall. Therefore, rib supports were introduced in major part of all the exit tunnels (Prakash and Sanganeria 1993).

5.4.3 Behaviour of the rock mass and installation of rib supports in the tunnels

Tunneling through vertical dolerite dyke and sub-horizontal dolerite sill dissected by chlorite coated joints, shears and slaked zones experienced problems of rock falls and roof falls. Tunneling in dolerite rock exposed blocks of various size that were released from the opening periphery resulting in large overbreaks and safety hazards. Failure occurred where removable blocks were formed by the introduction of a free face of the tunnel or due to intersection of three sets of joints, especially chlorite coated joints having low shear parameters. Pattern rock bolt supports based on the general rock mass classification could not prevent the roof falls/ collapses in the tunnel sections occupied by the chloritised, slaked and jointed dolerite rock necessitating installation of rib supports.