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Geotechnical Problems of the Underground Excavation in the Deccan Basalts of Sardar Sarovar (Narmada) Project, Gujarat, India

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SYNOPSIS The Sardar Sarovar (Narmada) Project, Gujarat State envisages the construction of an underground power house (6x200 MW) and its ancillary structures in the Deccan basalt. The basalt lava flows in the area are intruded by dolerite dykes and sills and dissected by fractures, shears and faults. These features have posed varied geotechnical problems like block falls, wedge failures, roof collapses and water seepage during the excavation of machine hall, access tunnel, draft tube tunnels and exit tunnels. The adequacy of support system designed on the basis of Barton's and Bieniawski's rock mass classification is constantly monitored and reviewed from time to time. The main power house cavern (210x23x58m) is being entirely supported by rock bolts and shotcretes with wiremesh. In the shotcreted upstream and downstream faces of the power house cavern cracks for maximum height of 22 m has been observed and are under evluation. The rib supports have been introduced in tunnels passing through slacked zones in dolerite dykes and sills traversed by faults and shear zones.

INTRODUCTION

The Sardar Sarovar (Narmada) Project is a multipurpose river valley project located in Gujarat (Fig.1). The project envisages construction of 1270 m long and 162 m high concrete gravity dam, 1200 MW underground power house and 240 MW Canal Head Surface power house. The main power house cavern and its ancillary structures including six pressure shafts, six draft tube tunnels, an access tunnel and three exit tunnels are located in the right abutment (Fig.2).

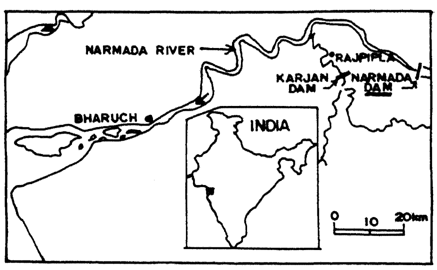

Fig. 1-Location plan of Sardar Sarover(Narmada) Dam Project.

Geotechnical investigations were carried out for assesing the rock mass condition included geological mapping core drilling (1530 m length), laboratory and field testing of rock cores and rockmass. Detailed geological investigations by excavating an exploratory drift at the roof level of the cavern extending beyond the full length of power house were started and completed in the year 1979-80. This exploratory dirft was widened later (1983-84) to full width (23 m) in a length of 165 m and to a depth of 18 m and instruments were installed to monitor the behaviour of roof arch. Hydraulic fracturing tests around the cavern have been conducted in the year 1991 for evaluation of in-situ stresses.

The main civil works for power house started in the year 1987 and are still in progress. The geotechnical problems encountered during the underground excavation are discussed in this paper.

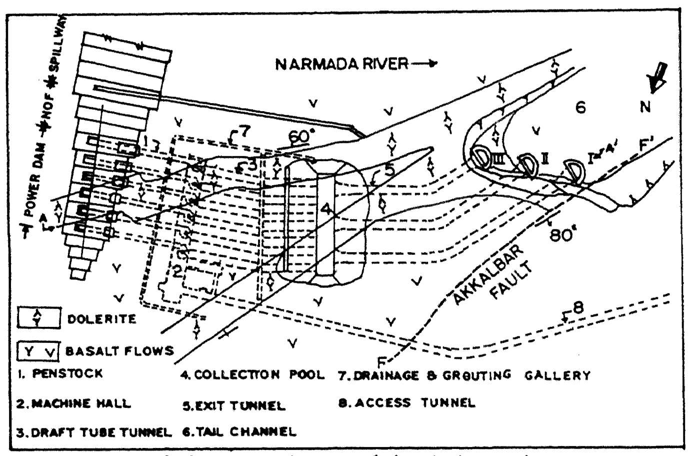

FIG. 2. Geology and layout of river bed powerhouse

GEOLOGY

The project area is occupied by Deccan basalt flows of Cretaceous-Eocene age which unconformably overlie the infra-trappean sedimentary rocks. Underground structures of power house are located in the sub-horizontally disposed basalt flows intruded by dolerite dykes and sills. The individual lava flows exhibit compositional as well as textural variation, both laterally and vertically and are mainly composed of dense, porphyritic and amygdular varieties of basalt with intervening discontinuous layers of agglomerate (Mehta and Prakash, 1990). Rock mass at depth around machine hall (cavern) and tunnels is fresh but jointed and dissected by shear zones at places. The presence of chlorite coated joints and calcified veins along the fractures and shear zones have adversely affected the strength of otherwise competent rock mass.

PHYSICO-ENGINEERING PROPERTIES OF ROCKS

-The physico-engineering properties of rocks have

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been evaluated on the basis of laboratory and field tests (Prakash, 1990).The properties of the individual lithounits and rock mass adopted in the designing of the different structures are summarised in Table-1

Table-1: Physico-engineering properties of rocks

Properties	Basalt	Dolerite
(A) Intack rock		
Specific gravity	2.85	2.95
Water absorption	0.7%	0.9%
Uniaxial compressive strength	11.5 MPa	77.6 MPa
Tensil strength	11.7 MPa	7,7 MPa
(B) Rock mass		
1. Modulus of deformation	0.12×10^4 to 0.14×10^4 MPa	0.22×10^4 MPa
2. State of secondary stress in rock mass		
i) Vertical	1.4×10^4 MPa	1.3×10^4 MPa
ii) Horizontal parallel to the cavern	1.195x104MPa	0.96×10^4 MPa
iii) Horizontal perpendicular to the cavern	0.90×10^4 MPa	

Hydro-fracture tests have been conducted for determining precise data of in-situ stresses after the part excavation of the cavern. The results show that the horizontal stresses are 3 times the vertical stresses along the power house cavern and about 1.2 times the vertical stresses in the direction perpendicular to the axis of the cavern. The direction of the maximum principal horizontal stress is North + 5°(Gowd et al. 1992). It has been noticed that secondary stresses measured earlier in the exploratory drift are different than evaluated by the Hydrofracture test.

GEOTECHNICAL PROBLEMS

I. Power House Cavern (Machine hall):

The problems encountered during the excavation and construction of the cavern includes rock falls from the roof arch and development of cracks in the upstream and downstream rock faces. The machine hall is located 30 to 65 m below the average ground level between two ENE - WSW trending dolerite dykes (Fig.2 & 5). Jointed basalt and agglomerate are exposed at the crown and on the sides above El 20m. A major part of Trubo-generator Units are located in the dolerite sill having chlorite coated joints. Rock mass is dissected by shear zones (Fig.3) and practically devoid of ground water. However, drainage galleries have been provided all around the cavern to drain out seepage water after filling of the reservoir.

Rock mass classification:

The rock quality has been evaluated by adopting Barton's 'Q' - system and Bieniawsi's RMR method. Four units have been identified for design consideration (Table-2).

Table-2: Rock mass Rating of Different Units

No.	Units	Bieniawski's RMR	Barton's 'Q'
I.	Dolerite	72	14.2- 18.5
II.	Basalt	67	9.3- 15
III.	Shear zone at dolerite basalt contact	23	0.33-0.5
IV.	Basalt between shear zones	30	0.25

Design support system:

The supports are being provided based on the New Austrian Tunnelling Method (NATM) depending on the rock mass classification of power house cavern. The rock mass is supported by tensioned grouted rock bolts of 6 to 7.5 m length at a spacing varying from 1 m to 1.75 m staggered and two layers of 38 mm thick shotcrete with wiremesh (Fig.3).

Instrumentation:

The presence of weak features in the power house cavern and designing the support system on NATM method have necessitated monitoring of the cavity by instruments. Single point and multipoint bore hole extensometers load cells, pore pressure cells and stress meters have been installed to estimate the deformations likely to take place on the roof and side walls and review the support system accordingly. Demac points and crack meter have also been installed after the development of cracks in the walls. Deformation of agglomerate layer in the roof arch has been recorded by the instruments.

Rock falls in the crown:

Rock fall in the crown and arch occurred in Feb.,1988 between RD 1540 and 1556 m involving about 125 cubic meter of rock mass. Three point bore hole extensometer installed in the year 1984 at RD 1508 m to study the behaviour of agglomerate layer indicated that one of the contact of the agglomerate with basalt is getting opened at a very small but constant rate of 0.024 mm/month resulting in the rock fall. Total opening noticed before the rock fall from August, 1984 to Feb.1988 was 3.03 mm (Geol & Jethwa, 1991). Overbreaks of the order of 1.5 to 2 m have also occurred in the upstream of the roof arch between shear zones 'A' and 'B' (Fig.3). As a remedial measures to contain the fall in these areas, additional rock bolts in between the pattern rock bolts have been provided besides two additional shotcrete layers with wiremesh. No further opening of the contact and roof fall has so far been observed in the treated area.

Development of cracks in the walls:

i) Upstream wall:

The crack started developing in the wall when excavation progressed to E1.14 m. The first 18 m high crack at RD 1569 m was noticed in March, 1991. Length of the crack increased from 18 to 22 m in Sept.1991, and new cracks developed during benching operation from E1.14 m to (-)2 m till April 1992. Most of the cracks developed between RD 1547 and 1580 m are vertical in nature with maximum opening of the order of 15 mm. A few horizontal cracks have also been noticed. These vertical & horizontal

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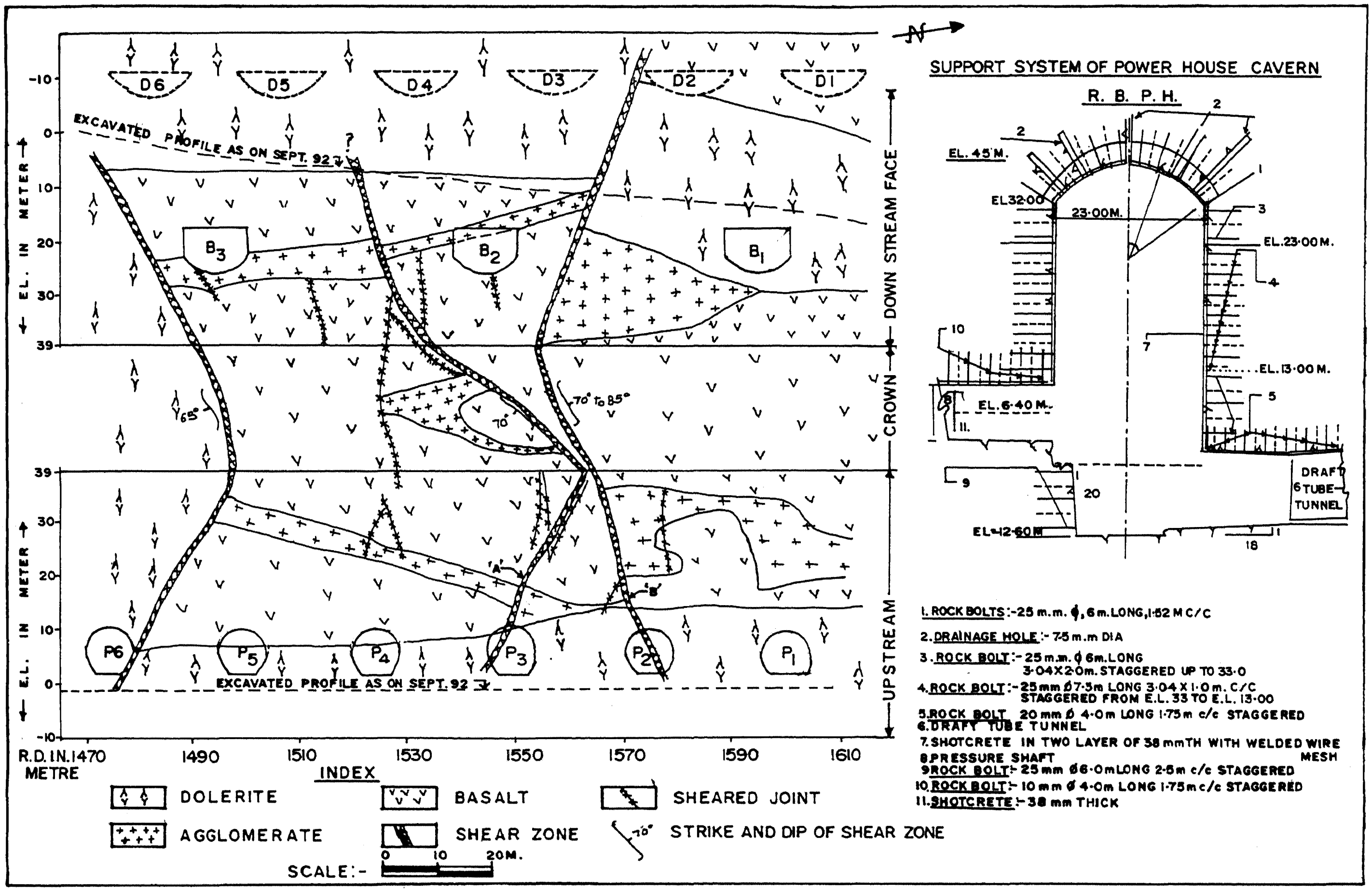

FIG. 3. 3 - D GEOLOGICAL LOG OF MACHINE HALL

cracks are developed along and adjacent to shear zones 'A' and 'B' above pressure shafts - 2 & 3 between E1.11 and 37 m. Extension of the cracks inside the rock mass has been observed by opening windows in the shotcrete. Snapping of the wiremesh has also been noticed. Detachment of the shotcrete has been observed just below the spring line of the machine hall in about 30 m length. Maximum dislocation of the shotcrete from rock face of the order of 200 mm has been noticed between E1.36.5 and 37.5 m, that is just below the spring line E1.39 m (Fig.4).

Deformation of the rock mass has continued since March, 1991. Reappearance and develoopment of new cracks in the shotcrete and widening of the existing cracks during benching operation from E1.14 to (-) 2 m are some of the evidences of continuous deformation. Glass plates installed across the cracks are also found brocken. Monitoring of these cracks by Demacpoints and crack meter is in progress.

Probable cause of the development of cracks:

The probable reasons for the development of cracks can be one or the combination of (a) Differential movement of rocks in the vicinity of shear zones; (b) Adjustment of rock mass between inadequately supported pressure shaft openings; (c) High in-situ

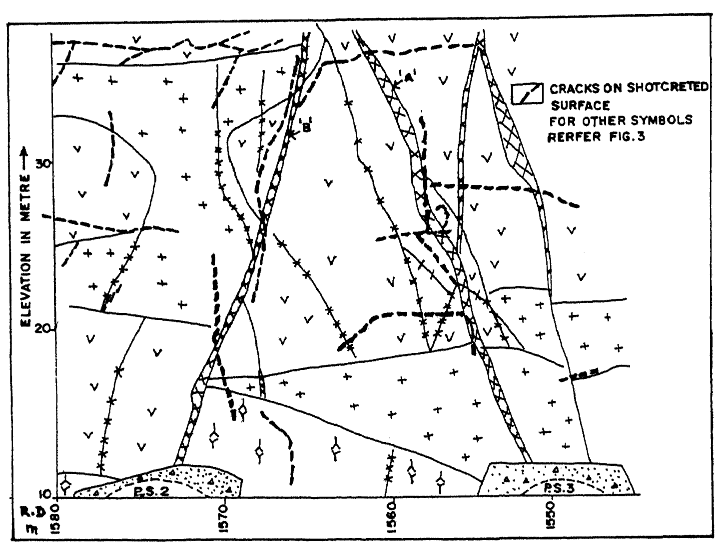

FIG.4 :- PART OF UPSTREAM FACE OF THE MACHINE HALL SHOWING DISPOSITION OF CRACKS.

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stresses acting on the walls; (d) Sliding or rotational movement of wedge formed between two shear zones 'A' and 'B'. The wedge formed **converges** inside the rock mass in the upstream of the cavern (Fig.3). The size of the wedge is smaller at top and gradually widens towards bottom.

Three dimensional finite element analysis is in progress to evaluate stress pattern and displacement of the rock mass for proper understanding the forces responsible for the development of cracks in the upstream wall.

ii) Downstream wall:

The cracks were also observed in April 1992 between RD 1490 and 1525 m. These cracks are along and parallel to shear zone 'A' between E1.7 and 26 m. Wedge formed by shear zone 'A' and 'B' is diverging inside the rock mass and thus it is a stable wedge. Another wedge formed between shear zone 'A' and vertical joints in the basalt with the combination of low dipoping shears towards free face in the underlying dolerite sill and high stress can be responsible for the development of cracks in the downstream wall.

Remedial measures:

The remedial measures includes provision of additional longer rock bolts at shorter spacing and or long tendons besides additional layer of shotcrete and improvement of the disturbed rock mass by grouting and drainage. The exact dimension and spacing of the tendons will be governed by the results of finite element analysis under progress.

II. Acceess Tunnel:

The D-shaped 8.5 m wide and 9 m high, access tunnel passes through basalt and agglomerate for a length of 230 m and dolerite dyke/sill in the remaining 630 m length. The Akkalbar fault aligned in N60°E - S60° W direction, dipping 70° towards NW is about 8 to 10 m wide and cuts the tunnel at a distance of 500 m from inlet portal (Fig.2). The support system of access tunnel for most of its reach comprises 25 mm dia 6 m long pattern rock bolts at 1.75 m c/c with 2 layers of 38 mm thick shotcrete layer with wiremesh in between. Steel sets were also provided in few critical reaches. Problems due to shear zone, fault and water seepage has been encountered during the tunelling operation.

i) Shear zone:

A sub horizontal shear running near the crown of the tunnel at the interface of agglomerate and basalt has resulted in the overbreaks causing flat roof in the initial 230 m length. Problem of flat roof has also been encountered in dolerite sill where sub horizontal sheared joints are present near the crown of the tunnel. As a remedial measure spacings of the rock bolts in the crown has been reduced from 1.75 m c/c to 0.75 m c/c in such reaches.

ii) Akkalbar Fault:

For the tunnel section affected by Akkalbar fault, rock load of 26 t/m was estimated considering it a crushed rock as per Terzaghi's classification. Steel ribs of ISMB 300x140mm (44.2 kg/m) and ISMB 450x200m (79.4 kg/m) were provided at 500 mm centre to centre and back filled with concrete (Shah et.al.1992). This fault is exposed in the reservoir of rock fill dam located about 160 m north east of the tunnel. Seepage of the order of 50 - 60 liters per minute was noticed in the tunnel from the fault zone when the reservoir water level was around E1.66 m. The seepage was anticipated to increase manifold when the reservoir reaches its FRL at E1.95.10 m. Grouting from the roof reduced the leakage about 50%. The seepage water is proposed to be diverted through drainage holes channelised to the sump well.

iii) Shallow rock cover:

A stretch of about 50 m length of tunnel passes below the already constructed 57 m high rock fill dam with a water storage of about 30 m depth. The toe of the dam is about 12 m from the alignment of tunnel on one side and open cut 40 m deep in the collection pool on other side. The rock cover over the tunnel in this reaches was low varying from 10 m to 17 m. The excavation in this part was done very carefully by smooth blasting techniques and monitoring the peak particle velocity at the toe which was limited to 6.25 mm/sec. Steel ribs were provided in the low cover reach.

III. Draft Tube Tunnels:

The draft tube (D.T) tunnel of 10 m diameter are passing through mostly dolerite sill dissected by chlorite coated joints and low dipping shears. Excavation of the heading portion of the DT-1 and 2 is completed and of DT-3, 5 and 6 is in progress. Unfavourable orientation of the discontinuities and presence of chlorite in the dolerite sill (RMR=45, Q = 0.63) have resulted in the roof fall in the DT-2 and DT-3 near interconnecting galleries. Major rock fall occurred on 6.11.90 between RD 74 and 86 m in the DT-2 along the sub horizontal shears involving overbreak of the order of 4.5 m (Prakash and Chidambranathan, 1991). Design rock bolts 20 mm dia, 4 m long, 1.75 m c/c could not prevent the dilation of joints and shears in this area. Steel ribs have introduced in these tunnels after the collapses. However, problems of flat roof and overbreaks are continuing in all the tunnels in the reaches occupied by dolerite sill.

IV. Exit Tunnels:

Horse shoe shaped exit tunnels (E.T) of12.5 m diameter are passing through basalt, agglomerate, dolerite dyke and sill. The Akkalbar fault runs parallel and close to the alignment of E.T-1 from the outlet portal (RD 0 m) to kink point (RD 222 m). Joints sympathetic to the fault are traversing all the three tunnels, but they are more prominent in the E.T -1. A low dipping shear zone dipping 30° towards south (outlet end) is crossing the alignment of E.T-1 at RD 100 m, E.T-2 at RD 62 m and E.T-3 at RD 50 m. Minor water seepage has been noticed along this shear (Prakash & Chidambranathan, 1991). About 50% of the tunnel length is passing through slacked dolerite/ chlorite coated joints (Fig.5). The physico-engineering properties of the dolerite are given in the table-3.

Laboratory shear tests of the chlorite coated joints have given the value of C as zero and ϕ = 18°. Low value of the shear parameters of the chlorite coated joints are indicative of poor shear strength of rock mass.

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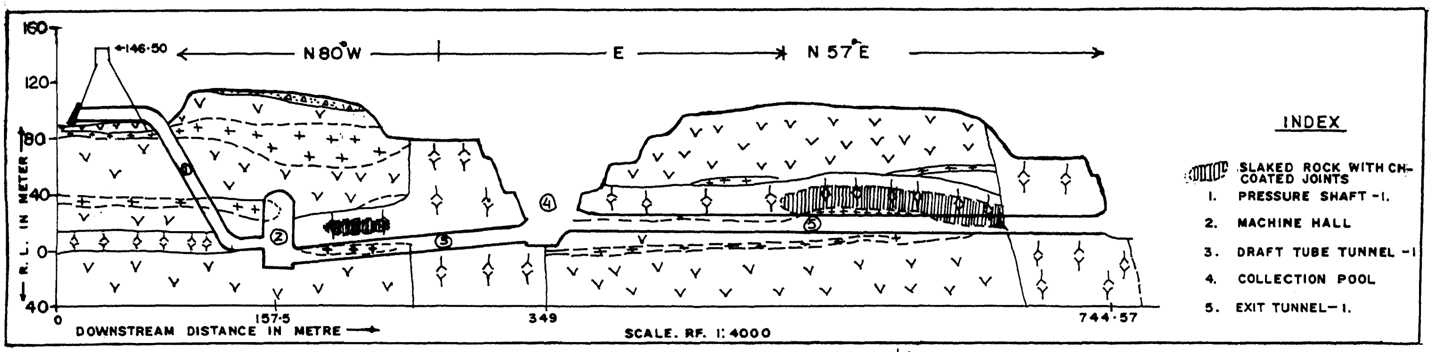

FIG.5.GEOLOGICAL SECTION ALONG WATER CONDUCTOR SYSTEM (A - A') (FOR INDEX REFER FIG-3)

Table:3 Physico-engineering properties and rock mass rating of dolerite (E.T - 1).

Properties/rating	Chloeirized Dolerite	Chloritized and slacked Dolerite
I. Properties		
% of water absorption	0.5 - 1.4	0.8 - 2.7
% of porosity	1.4 - 3.9	2.3 - 7.5
True specific gravity	2.85- 2.95	2.8 - 2.9
Uniaxial compressive strength (saturated)	60 MPa	34 MPa
II. Rock mass rating		
R.M.R. value	52	49
'Q' value	1.06	1.0

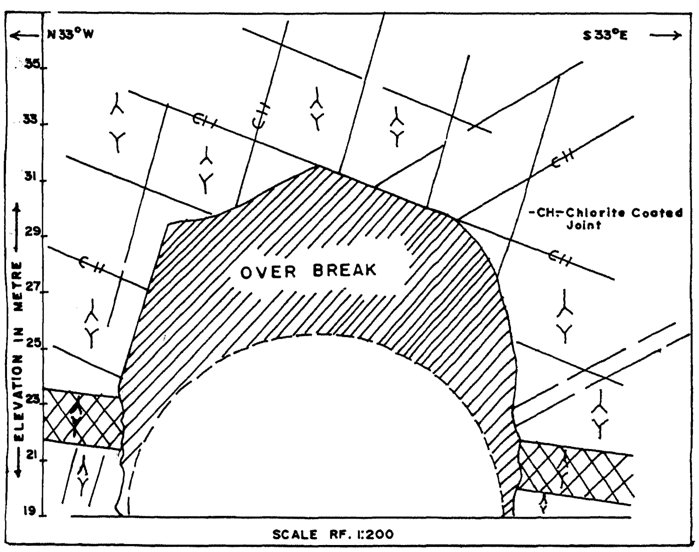

FIG.6. GEOLOGICAL CROSS SECTION OF EXIT TUNNEL- I. RD.79 m. (FOR INDEX REFFER FIG.3)

Major roof falls/block falls occurred in the exit tunnel-1 between RD 0 and 113 m after the installation of the rock bolts (Fig.6). About 50% rock bolts have been reported to be slipped in the slacked/chloritized zone during tensioning. Second layer of the shotcrete with wiremesh was not provided in the month of May 1990 and subsequently resulted in the roof falls in the month of Sept.1990 after the entry of flood water. Intersection of three sets of chlorite coated joints are forming removable/detachable blocks of size varying from 0.5 x 1 x 2 m to 1 x 2 x 3m resulting in block falls in the exit tunnels at places. Pattern rock bolts could not prevent the collapses in the slacked rock zones. Design support system based on rock mass classification included pattern tensioned grouted rock bolts and shotcreted with intervening wiremesh. Goodman & Hatzor (1990) opined that in highly discontinuous rock formations general rock classification is questionable. After the collapses support system was reviewed and steel rib supports were introduced in all the tunnels at critical locations.

CONCLUSIONS

Geotechnical problems encountered during the construction of the underground power house and its ancillary structures in the Deccan basalt were not anticipated during pre-construction stage investigation. Critical examination and evaluation of the rock mass during construction stage geotechnical investigations have helped in reviewing the support system from stability and safety considerations. Support system based on the rock mass classification consisting of pattern rock bolts and shotcrete was designed for all the underground openings. After the rock falls from crown in the tunnels and development of cracks in the machine hall, the support system has been re-evaluated and reviewed. In all the tunnels steel ribs have been introduced at the locations of adverse rock mass conditions where unstable rock blocks are formed due to intersection of joints and shears. The state of stresses in the excavated rock mass around power house cavern is under evaluation for deciding the additional treatment required for stabilization. Tendons and longer rock bolts are being designed to stabilize the individual rock wedges formed in between the critical joints and shears in the main power house cavern.

The data collected and implications of the geotechnical problems encountered indicated that a synthesis of the information gathered during construction stage

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studies is of prime importance in such projects.

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