01 May 2013, 2:00 pm - 4:00 pm

Application of Observational Method in the Successful Construction of Underground Structures, Sardar Sarovar (Narmada) Project, Gujarat, India

Indra Prakash
Geological Survey of India, India

Commemorate the Legacy of Ralph B. Peck
April 29 - May 4, 2013 CHICAGO
Seventh International Conference on
Case Histories in Geotechnical Engineering
and Symposium in Honor of Clyde Baker

APPLICATION OF OBSERVATIONAL METHOD IN THE SUCCESSFUL CONSTRUCTION OF UNDERGROUND STRUCTURES, SARDAR SAROVAR (NARMADA) PROJECT, GUJARAT, INDIA

Indra Prakash
Ex. Dy. Director General
Geological Survey of India
Gandhinagar, Gujarat, India

ABSTRACT

The Sardar Sarovar (Narmada) underground powerhouse is located in Deccan Basalt flows in the lower Narmada valley in Gujarat state. These flows are intruded by dolerite dykes and sill. Basalt flows and dolerite rocks are considered good tunneling media for locating underground structures. Therefore, initial designing of supports was done considering good rock mass conditions. However, during construction numerous geotechnical problems were encountered necessitating review of support system. Rock falls and collapses were observed in tunnel sections passing through dolerite dykes and sills. Cracks were observed in the walls of the powerhouse cavern. During progressive excavations back analysis was done to know the causes of distress in rock mass and structures The 'Observational Method' adopted during construction resulted in the safe execution of structures by timely modification and installation of adequate support system.

INTRODUCTION

The Sardar Sarovar (Narmada) Dam has been constructed across Narmada River in the Lower Narmada valley in Gujarat State (Fig.1). The Underground Powerhouse of installed capacity 1200 MW (6 x 200 MW) is located at 160m downstream of the dam on the Right Bank in basalt flows intruded by dolerite dykes and sill. The size of the machine hall (Cavern) is 23 m (wide) x 57.5m (high) x 212m (long). The tailrace system of power house comprises of six Draft Tube Tunnels of 10m finished diameter (10.5m excavated).

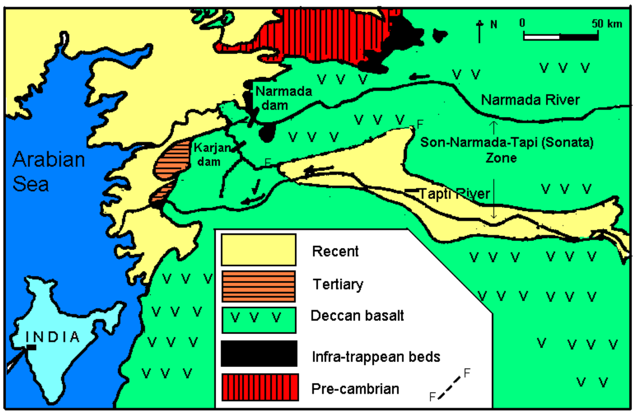

Fig.1. Location map of Narmada dam

Three horse shoe shaped Tail Race Tunnels (Exit Tunnels) of 12m finished diameter (excavated 13m), off take from the collection pool to discharge the tail water in main river through tail race channel (Fig.2.).

Ideally designing of the structures should be based on proper geotechnical investigations. However, despite extensive pre-construction stage geotechnical investigations it is not possible to identify and delineate exactly all subsurface weak features which may adversely affect the structures. Surprises are always there which are to be tackled during construction of underground structures.

There are generally two design approaches, in the first approach design is to be finalized prior to the commencement of the construction process and in the second approach it can be modified during construction as per actual observed site conditions (Peck 1969). The objective of Observational Method is to achieve greater overall economy without compromising safety. The Observational Method in ground engineering is a continuous, managed, integrated process of design, construction control, monitoring and review that enables previously defined modifications to be incorporated during or after construction as appropriate (CIRIA 1999).

At Sardar Sarovar Project Machine hall and other underground structures were excavated by heading and benching as per New Austrian Tunneling Method (NATM). This method relies on the mobilization of inherent strength of the rock mass. Underground structures at Sardar Sarovar site were initially provided primary support such as shotcrete with wire mesh and pattern rock bolts to enable rock mass to support itself. However, after observation of the cracks in the walls of the cavern and collapses in tunnels during progressive excavations additional remedial supports were provided to stabilize rock mass and structures.

GEOLOGY AND ROCK MASS CHARACTERISTICS

The rocks in which underground power house and its ancillary structures have been excavated consists of amygdaloidal and porphyritic basalt flows separated by pockets of agglomerate and intruded by ENE-WSW trending, 25 to 30m thick, vertical and inclined (60^o-65^o towards SSE) dolerite dykes and low dipping (20^o -25^o SE) dolerite sill aligned in NE-SW direction (Fig.2). These rocks are strong (>60 MPa compressive strength) but jointed having block size of approximately 1 to 2 cubic meters.

Rock mass inside the underground structures in general is fresh except in isolated places where it is weathered and altered. Barton's 'Q' and Bieniawski's RMR systems have been applied to evaluate rock mass (Bieniawski 1976). Basalt rock is jointed and belongs to fair quality (Q=9.16, RMR=60). Inclined dolerite dyke is of good quality (Q=10, RMR=63), whereas Vertical dolerite dyke (Q=1.5, RMR=45) and dolerite sill (Q=0.6 to 1.25, RMR=40) belongs to poor category mainly due to presence of chlorite-coated joints and slaked zones.

INITIAL DESIGN SUPPORTS

The original support designed for the power house cavern consisted of pattern rock bolts and two layers of shotcrete with a sandwiched layer of welded wire mesh. Roof supports provided included tensioned rock bolts of 25mm dia., 6m long and 1.75m center to center (c/c) pre tension to 14 tonnes load and two layers of 38mm thick shotcrete with wire mesh in between. Wall supports included 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 were provided to make the overall spacing of 1.52m c/c. Similarly, original support system of all the tunnels comprised of 25mm diameter, 4 to 6m long pattern rock bolts spaced at 1.75m c/c with two layers of 38mm thick shotcrete with wire mesh in between (Divatia and Trivedi 1990). However, in highly discontinuous rock formations, the validity of empirical design methods based upon general rock classification is questionable (Goodman and Hatzor 1990).

EXCAVATION SEQUENCE

Machine hall and other underground structures were excavated by heading and benching method by adopting New Austrian

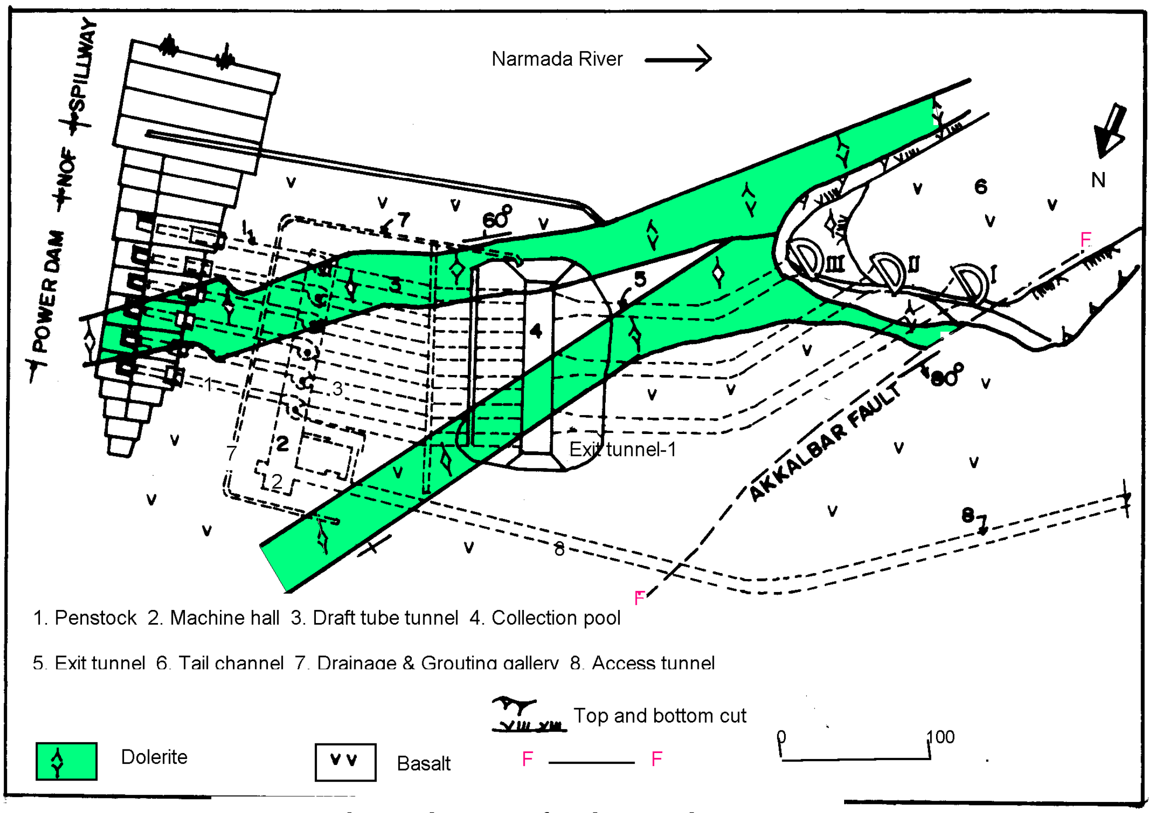

Fig.2. Geology and Lay out of Underground Structures

tunneling method (NATM). Six numbers of cross drifts from the central exploratory drifts were excavated from El. 45m to 39m. After observing the behaviour of rock mass of the crown, powerhouse cavern was progressively enlarged from 5m to 9m and then to its full width of 23m. The bench height varied from 2.5 to 4.0m. Similarly walls of the cavern were excavated from top to bottom in steps.

GEOTECHNICAL PROBLEMS OF POWER HOUSE CAVERN

The machine hall is having shallow basalt rock cover (varying from 35m to 60m) that is about 1D. Crown of the powerhouse cavern is at El. 45m and bottom at El. -12.5m. Longer axis of the powerhouse cavern is nearly aligned parallel to the direction of maximum horizontal stress ±N5^{o}E. During benching operation cracks (fissures) were noticed in the shotcrete of the machine hall walls. Identification of the distress zones/ areas in the underground structures was done by the visual examination. Systematic studies were done to know whether these cracks were superficial cracks in the shotcrete/ concrete or they were manifestation of the deformation of the inadequately supported rock mass.

Following problems were observed in the machine hall during progressive excavation:

Rock fall in the Crown

Rock fall in part of the crown occurred between R.D. 1540 and 1556m along the contact of agglomerate and basalt bounded by two shear zones (‘A’ and ‘B’) in the month February 1988 (Fig.3). At that time pattern rock bolts were already installed. Height of the over break was of the order of 1.5 to 2m. About 125 cubic meters of the rock mass was detached along contact. This contact was already being monitored with the help of three point bore hole extensometer. Analysis of Instrument’s record revealed that total opening of the contact was 3.03 mm from August 1984 to February 1988 and it opened at a very small but constant rate of 0.024 mm/ month prior to the rock fall. Pattern rock bolt supports could not prevent the opening of the contact and thus rock fall from the crown. Therefore, this area was back filled with concrete and tied with longer rock bolts in stable rock mass. To prevent further opening of the contact in adjacent area additional longer rock bolts were provided besides two additional shotcrete layers with sandwiched wire mesh.

History of development of Cracks (fissures) in the machine hall walls

Excavation of the roof with pattern rock bolt support system was completed from El. 45m (crown level) to 39m in the month of December 1989. Then excavation of wall was started and completed by providing pattern rock bolt supports and shotcrete up to El. 20m by the month January 1992. Further excavation in the machine hall was done by excavating a ramp

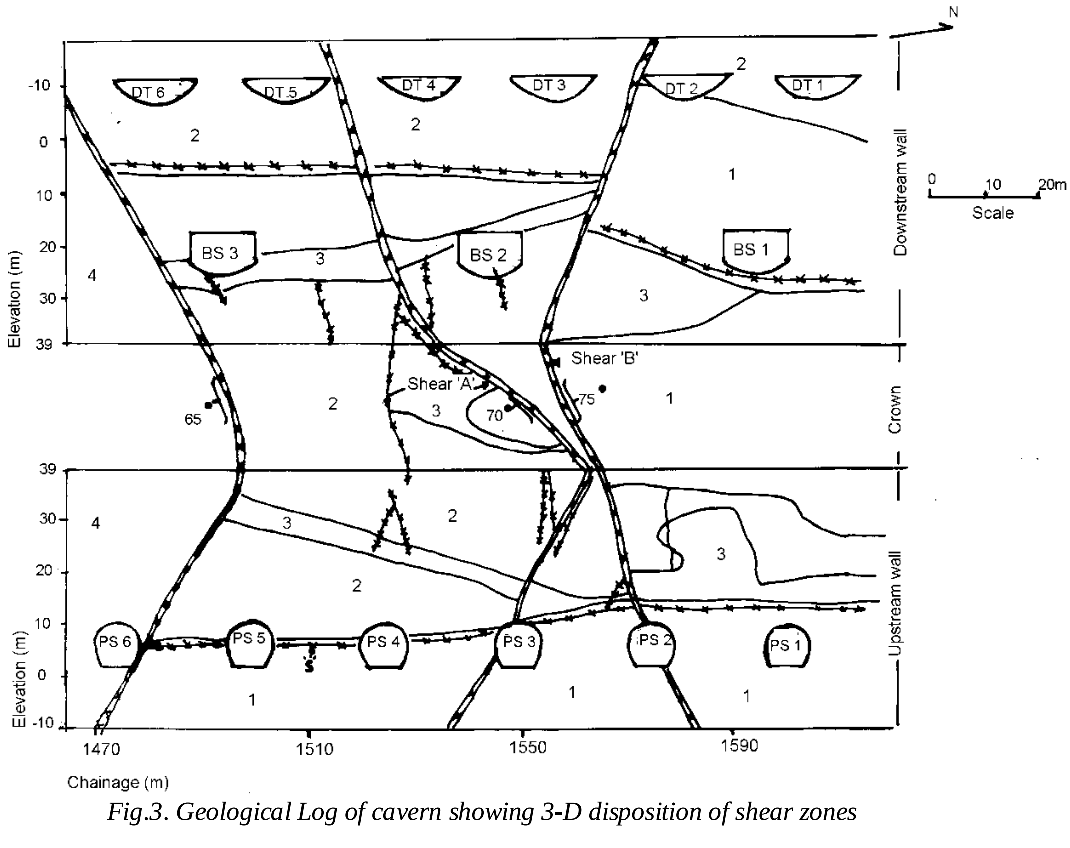

Fig.3. Geological Log of cavern showing 3-D disposition of shear zones

adjacent to downstream wall from El. 20m (service bay level) from service bay side to El. 4m river end side. From this ramp excavation of upstream wall was done up to El. -1.9m by the month June 1992. These excavations have been carried out by providing shotcrete and pattern rock bolt supports.

Observations of Cracks in the Upstream Wall. In the month of March 1991 an 18m long vertical crack was observed at ch.1569m. At that time bottom of the excavation was at El. 14m. It was initially considered that this crack in the shotcrete of the wall is of superficial nature and thus was stitched by criss-cross 4 to 6m, long inclined rock bolts. Additional layers of shotcrete with wire mesh were also provided. More cracks were observed between El. 13.50m and El. 36m between Ch. 1545 and 1585m in the month of October 1991 when the excavation was up to El.10m. The crack (at ch.1569m) stitched earlier reappeared when the excavation reached up to El. 1.9m. During further excavation up to El. -2m New cracks were observed between El. 11 and 37m, in the month of April 1992 along and above pressure shafts No. 2 and 3 adjacent to shear zones ‘A’ and ‘B’. By this time peripheral fissures were developed around three pressure shafts No. 2, 3 and 5. Most of these cracks were vertical in nature having maximum opening of the order of 15mm. A few sub-horizontal cracks were also noticed in the wall besides dislocation of shotcrete (up to 200 mm) in about 30 m length from rock face that is just below the spring line of the machine hall (El. 39m).

Observations of Cracks in the Downstream Wall. During the same period when cracks (fissures) on the upstream wall were developing, the cracks and popping in shotcrete of downstream wall between ch.1505m and ch.1520m were observed between El. 9.0m and El 27.0m along and adjacent shear zone ‘A’. More cracks were observed developed during the period April 1992 to May 1994 in the downstream wall below spring level.

Cracks (fissures) were also observed in shotcreted/ concreted part of the pressure shafts up to 10m distance and bus galleries up to 17m distance from portal face.

Visual observations of cracks and instruments record

No movement inside the rock mass was recorded by then installed Single and Multi-Point Bore Hole Extensometers and Stress Meters despite cracks were observed developing and enlarging in the shotcrete of machine hall walls, bus galleries and pressure shafts. This has created doubt in mind whether these cracks were superficial cracks or locations of the instruments were such that they could not record any movement or instruments were not working. For immediate confirmation a few windows were opened through shotcrete going inside the rock mass. Thus, it was concluded that cracks observed in the shotcrete/ concrete in the machine hall were not superficial cracks but they were manifestations of the deformation/ movement of the rock mass. Further, visual observation of the machine hall was continued by installing

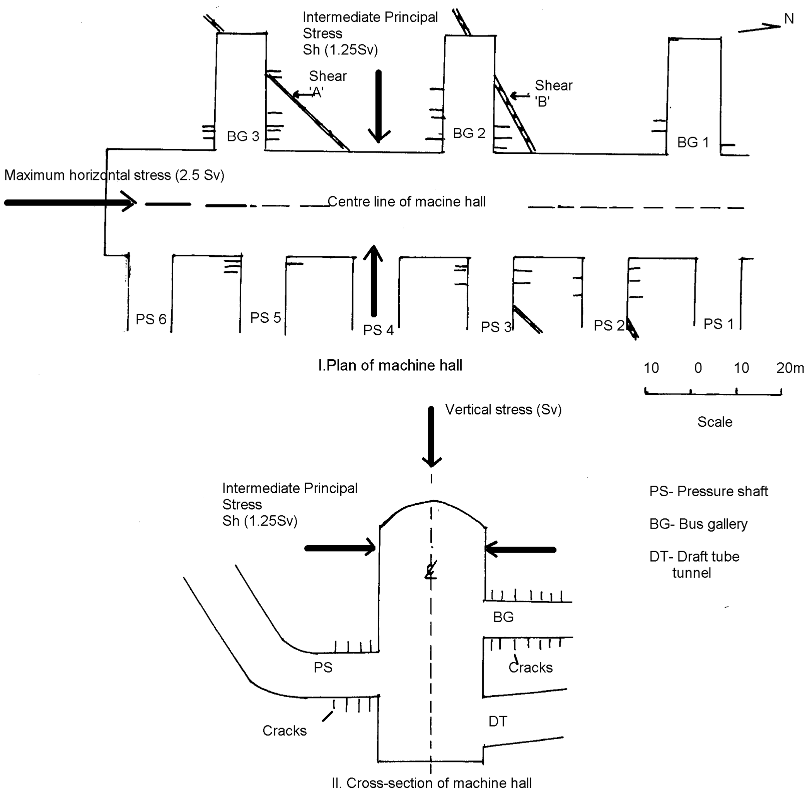

Fig.4. Vertical cracks in the Bus galleries and Pressure shafts

glass plates across the cracks. A few additional new multi-point borehole extensometers were installed in the wall besides Demac Joint Gauges and Crack Monitors in the bus galleries to monitor cracks and further deformation of rock mass (Prakash 2003).

Disposition of cracks

Cracks observed in the pressure shafts and bus galleries were vertical in nature and aligned parallel to the longer axis of the machine hall (Fig.4). Other vertical cracks in the walls were parallel to the shorter axis of the cavern. A few sub-horizontal and low dipping cracks observed in the downstream wall were developed in en echelon pattern, parallel to slope of then excavated profile of the ramp. Some of the cracks developed along and adjacent major shear zones “A” and “B”.

Probable causes of the development of cracks

Probable causes of the development of cracks cracks could be due to one or the combination of following reasons:

(a) Differential movement of rocks in the vicinity of shear zones;
(b) Adjustment of rock mass between inadequately supported pressure shaft and bus galleries openings;
(c) High in-situ stresses acting on the walls;
(d) Sliding or rotational movement of wedge formed between two shear zones ‘A’ and ‘B’;
(f) Inadequate design of supports.

To investigate the causes of the development of cracks and to know the present and future behaviour of the underground powerhouse cavern stability analysis considering geological features and stresses has been done.

Rock wedge analysis. The power house cavern is having shallow rock cover about 1D. 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 sidewalls of the openings. Two major shear zones (“A” and “B”) are traversing the machine hall. These shear zones are forming stable wedge in the upstream wall as the plunge of the intersection of shear zones is inside upstream wall (Fig.5). In the downstream wall these shears are diverging by virtue of their orientation and thus they are not creating problem of wedging.

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 ± 20°) to the slope face. In the cavern prominent shear zones and major joints are striking at an angle more than 30^0 to the alignment of upstream and downstream walls thus they are not posing problems of plane failure.

The Rock wedge and plane failure analyses of major discontinuities revealed that these features were not responsible for the development of cracks in the walls of machine hall.

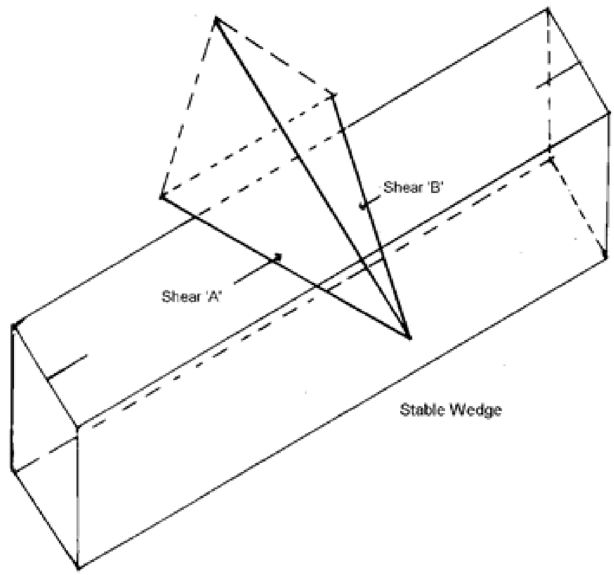

Fig.5. Disposition of stable wedge in the crown

Evaluation of in-situ stresses

The hydro-fracture tests indicated that minimum in-situ stress is vertical due to shallow rock cover and is equal to depth below surface times the unit weight of the rock (0.026 MN/m3). The major in-situ stress is approximately 2.5 times the vertical stress and is parallel to the longer axis of the cavern axis. The intermediate principal stress perpendicular to the cavern axis is approximately 1.25 times the vertical stress. As the average cover over the cavern roof is only about 45m (minimum 35m and maximum 60m), the vertical stress is approximately 1.25 MPa and the horizontal stress acting perpendicular to the cavern axis is approximately 1.5 MPa. The direction of the maximum principal horizontal stress is North ±5° (Prakash and Sanganeria 1992).

In-situ horizontal stresses measured in the machine hall perpendicular to the longer axis of the cavern by hydro-fracture test is low (1.5 MPa) and compressive strength of the rocks surrounding powerhouse cavern is much higher (> 60 Mpa). Therefore, there is no possibility of development of cracks due to in-situ stresses (Prakash 2002).

Three Dimension Numerical (FEM and DEC) Analysis

Three Dimension Finite element (3-D FEM) and Three Dimensional Distinct Element Code (3-DEC) back analyses

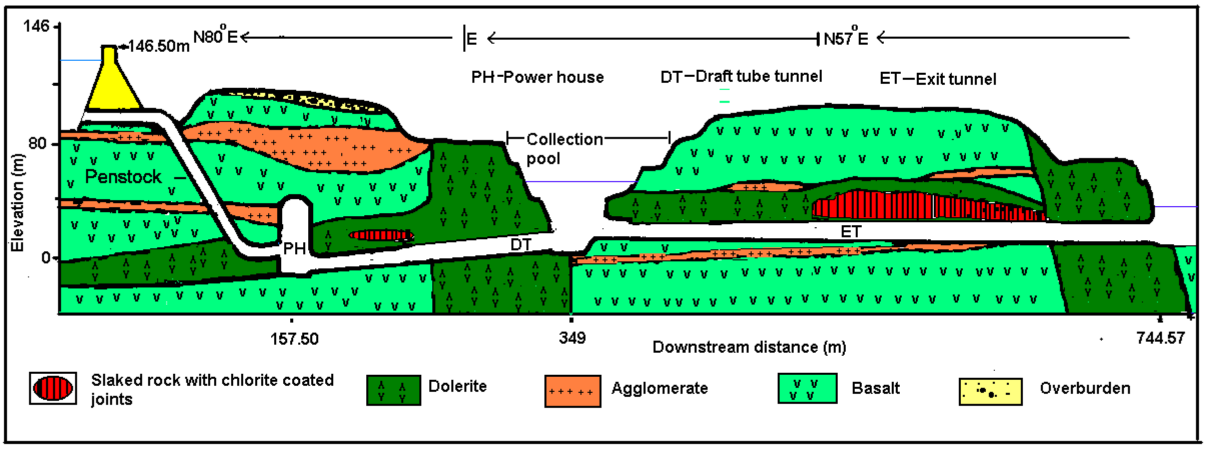

Fig.6. Geological section of underground water conductor system

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. Both the analyses gave almost identical results. Contour of Factor of safety of 1.5 in general is about 16 to 17m away from the face of the cavern wall inside the rock mass and around bus gallery-3 it is at a distance of 20m. These analyses corroborate the actual field observations where maximum displacement has been observed nearly at same locations.

Review of adequacy of initial design supports

After the development of cracks review of the design supports of the power house cavern was done 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)). It has been observed in the plot of rock bolt and cable lengths of various projects that 6m long rock bolts installed in the arch of the Narmada cavern falls within the precedent range. Actually no distress has been observed in the crown of the cavern except minor rock fall along open contact. Thus installed roof supports have been considered adequate for permanent arch support.

Similar approaches and plot for sidewall support for a 57-58m high cavern gave the average length for rock bolts and cable 10-11m and 20m, respectively. Study indicated that 6 to 7.5m long rock bolts provided for the side walls of the Narmada powerhouse cavern were too short and thus they could not provide adequate restraint and thus could not prevent development of cracks in both upstream and downstream walls (Goel and Jethwa 1992, Prakash and Srikarni 1998).

REMEDIAL MEASURES

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, and were installed. In addition to it a number of 25m long 50ton capacity cables 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 was done in the upstream and downstream walls to stabilize the loosened rock mass.

GEOTECHNICAL PROBLEMS AND TREATMENT OF TUNNELS

Problems of roof falls and collapses were observed in all the tunnels during progressive excavation as detailed below (Prakash & Desai 2004) (Fig.6):

Access Tunnel

Problems of flat roof and block falls were observed in the tunnel sections passing through vertical dolerite rock and dolerite sill dissected by chlorite coated joints. In the reaches occupied by widely spaced joints spacing of the pattern rock bolts was reduced from 1.5 to 0.75m. Individual rock blocks were tied by longer rock bolts in stable rock mass. In the area occupied by closely spaced chlorite coated joints (RMR=35, Q=0.6) steel ribs were installed (in 10m length) instead of rock bolt supports to prevent collapses.

Draft Tube Tunnels

Over breaks 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.7). Problem of flat roof was observed at many places. Initially longer rock bolt supports were installed to tie the rock blocks but they failed due to presence of chlorite coated joints and slaked rock zones. Slabs of rocks started falling along with rock bolts. Therefore, in these reaches rib supports were installed to prevent roof falls and collapses.

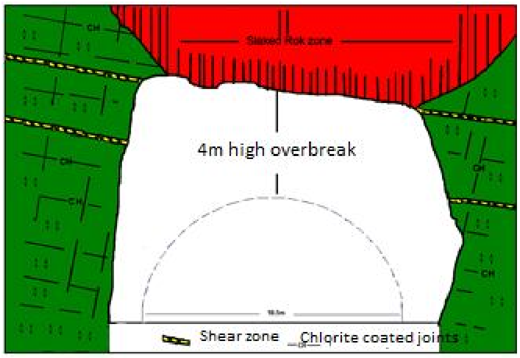

Fig.7. Rock fall from the crown of draft tube tunnel

Exit tunnels (Tail Race Tunnels)

A major fault ("Akkalbar" fault) is running parallel and close to the alignment of this tunnel (E.T.-1) up to the kink in about 200m length. Adverse effect of this fault was noticed during the excavation of tunnel. Joints sympathetic to the fault in the E.T.-1 were forming removable/ detachable blocks of size varying from $1m^3$ to $6m^3$ causing major block falls and roof collapses. Crumbling and slaking nature of rock mass was observed on the exposed surfaces (Fig.8).

Re-assessment of the rock mass was done by further subsurface exploration and it was noticed that about 50% length of the all the exit tunnels was passing through dolerite rock dissected by chlorite coated joints. In these zones about 50% rock bolts were noticed slipped during tensioning. Size of tunnel section was reduced from 11 to 7m and length of rock bolts was increased up to 10m but these measures could not prevent the collapses. Therefore, rib supports were introduced in major part of all the exit tunnels (Prakash and Sanganeria 1993).

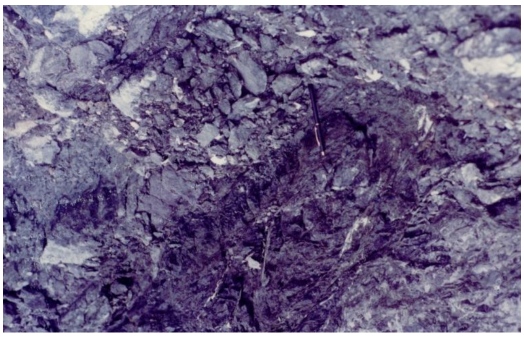

Fig.8.Crumbling and slaking nature of dolerite rock

DISCUSSIONS

The cavern of the powerhouse is having shallow rock cover (1D). Initially it was thought that basalt flows would not pose any problem for the excavation with the installation of pattern rock bolt supports and by providing immediate shotcrete with wire mesh as envisaged in the design. It was true for the excavation of the entire crown which could be constructed without any problem except minor block fall. But the problem was observed during the excavation of walls below the half height of the cavern. Cracks in the walls were started appearing up to spring level from the service bay level. Thus rock mass has not acted as competent structural material after the installation of initial design supports.

Review of the design supports indicated that 6 to 7.5m long rock bolts initially installed in the walls of machine hall walls were too short. Thus, they could not restrain the deformation of the rock mass and failed to prevent the development of cracks in the walls. As the powerhouse cavern was having shallow rock cover the vertical walls behaved like steep rock slopes and thus cracks developed due to stress relief under low confining stress. This is analogous to the situation, which arises when excavating very steep slopes in hard but jointed rock masses. Thus in the absence of adequate supports, vertical tension cracks which are common in steep rock slopes were formed parallel to the walls as observed in the pressure shafts and bus galleries. Longer rock bolts (12m) and cables (10.5 to 32m) were provided as remedial supports in the walls of machine hall based on the experience of similar other projects (Hoek 1995) (Fig.9).

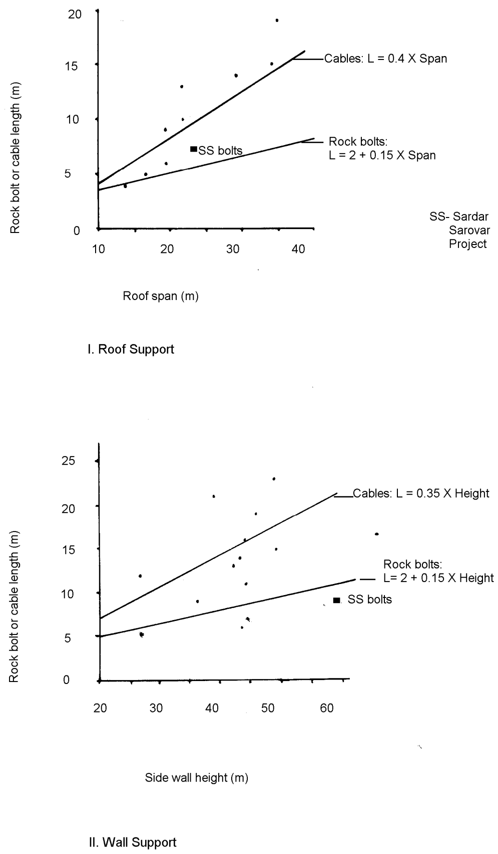

Fig. 9.Plot of Roof and Wall Support of various projects

In other underground structures namely access tunnel, draft tube tunnels and exit tunnels (tail race tunnels) rock falls and collapses were observed during the excavation mainly in the area occupied by dolerite rocks dissected by chlorite coated joints and also in the reaches passing through slaked rock zones.

Prior to the construction dolerite rock was considered good tunneling media. However, after the observation of slippage of rock bolts and collapses in the tunnel sections re-evaluation of rock mass and support system was done. It was observed that pattern rock bolt supports could not prevent rock fall of large removable rock blocks and also collapses in the tunnels passing through chloritized and slaked dolerite rock. Therefore, rib supports were installed in the major part of tunnels. Introduction of rib supports besides providing positive supports removed the fear psychosis among the site staff for working inside the tunnels.

CONCLUSIONS

It is desirable to continuously observe behavior of the rock mass and structures during construction visually as well as with the help of instruments. Anticipated rock mass conditions may differ at depth and surprises may be observed during actual construction. Under these circumstances modifications in the design of supports and structures may have to be done as per actual site conditions. At Sardar Sarovar (Narmada) Project additional longer rock bolts and cables were provided in the walls of machine hall (Cavern) and rib supports were introduced in tunnels after observing cracks and rock falls during construction. The ‘Observational Technique’ adopted resulted in timely modification of supports and thus safe execution of underground structures.

REFERENCES

Bieniawski, Z.T. [1976]. “Rock mass classification in rock engineering”, *Exploration for rock engineering, Proc. of the symp*., 1, 97-106. Cape Town: Balkema.

CIRIA. [1969]. Construction Industry Research and Information Association’s Report 185.

Cording, E.J., Hendron, Jr. A.J. and Deere, D.U. [1971]. “Rock engineering for underground caverns”, *Symposium on Underground Rock Chamber*s, January 13-14, Phoenix, Arizona.

Divatia, E. and Trivedi, V.P. [1990]. “Special Design Features of Right Bank Powerhouse Complex”, Indian Journal of Power and River Valley Development, Special Number (September), pp.261-269.

Goel, R.K. and Jethwa, J.L. [1992]. “Engineering Geological Problems and Assessment of Support Requirements For A Powerhouse cavern”, *Journal of Engineering Geology, India*, Vol. XXI Nos. 1&2, pp. 91-98.

Hoek, E.[1995]. “Review of River Bed Powerhouse Design, Sardar Sarovar Hydroelectric Project, Gujarat, Narmada Project Report, India, pp.1-22.

Hoek, E. and Brown, E.T.[1980]. “Underground excavations in rock”, *The Institution of Mining and Metallurgy, London*, pp.1-523.

Peck, R.B. [1969]. “Advantages and Limitations of the observational method in applied soil Mechanics”, *Geotechnique* 19, No. 2.

Prakash, I. [2003]. “Application of Forensic Geo-technical Engineering in the Stability Evaluation of Narmada Underground Powerhouse Cavern [Machine hall]”, *Workshop on Forensic Geotechnical Engineering, Indian Geotechnical Society,* Chennai, pp.1-8.

Prakash, I. and Desai, N. [2004]. “Rock Mass Evaluation of The Sardar Sarovar (Narmada) Dam and Underground Powerhouse, India”, *Fifth International Congress on Case Histories in Geotechnical Engineering, New York, NY*, Paper No. 2.26, pp.1-8.

Prakash, I. and Sanganeria, J.S. [1993]. “Geotechnical Problems of The Underground Excavations in the Deccan Basalt of The Sardar Sarovar [Narmada] Project, Gujarat, India”, *Third International Congress on Case Histories in Geotechnical Engineering, St. Louis, Missouri*, pp.889-894.

Prakash, I. and Srikarni, C. [1998]. Geotechnical Investigations of Sardar Sarovar (Narmada) Project, India, Eighth International Congress International Association for Engineering Geology and the Environment, Vancouver, Canada, pp. 377-384.

Prakash, I. [2002]. Engineering Geological Evaluation of the Major Dams in Lower Narmada Valley, Gujarat, *Unpublished Ph.D Thesis*, MS University of Baroda, Vadodara, pp.1-148.

United States Army Corps of Engineers [1980]. “Engineering and Design: Rock Reinforcement”’, *Engineer Manual EM 1110-1-2907 1980*, Washington D.C.