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ABSTRACT

Geotechnical evaluation of the 163 m high concrete gravity Sardar Sarovar (Narmada) dam, under construction, and 1200 MW underground powerhouse (210m x 23m x 57.5m) and its ancillary structures has been done. The dam and powerhouse sites are occupied by basalt flows underlain by infra-trappean sedimentary rocks (Bagh beds) intruded by basic dykes. The area is structurally complex and seismically active. Intra-formational shears and sub-horizontal to low dipping weak layers like red bole, tuff, argillaceous sandstone having low values of shear parameters posed the problem of sliding stability of dam blocks. Concrete shear keys were provided as one of the remedial measures. Differential settlement was apprehended in the foundation of dam having varying physico-engineering properties and rock mass characteristics. Reinforced concrete mats were provided to treat the weathered and sheared rock mass and 34.5m deep reinforced concrete plug to prevent differential settlement of dam blocks located on river channel (dam base) fault. The horizontal seismic coefficient adopted for the dam is 0.125g.

The construction of 1200 MW underground powerhouse located in basalt is nearing completion. During progressive excavation of the machine hall (cavern) cracks were observed in the 57.5m high shotcreted walls. Additional longer rock bolts/ cables/ tendons were provided as remedial measures. Draft tube and exit tunnels are passing through dolerite rocks dissected by chlorite-coated joints and slaked rock zones. Rib supports were introduced after observing behaviour of the rock mass and collapses in part of these tunnels.

Key Words: Narmada dam, Bagh beds, intra-formational shears, settlement, sliding, shear keys, concrete plug.

INTRODUCTION

The Sardar Sarovar (Narmada) Project is the largest multipurpose water resources project located in the Narmada Valley in Gujarat state in India to irrigate 17.93 lakh hectare of land. The main dam is 1227m long and 163 m high from the deepest foundation level (129m from the river bed level). The construction of 1200 MW underground powerhouse is in progress. The size of the machine hall (cavern) is 23m (Wide) x 57.5m (High) x 212m (Long). The 240 MW surface powerhouse is already completed.

The area is geologically complex and structurally disturbed. Bieniawski's RMR (1976) and Barton et al's Q (1974) classification systems and United State Bureau of Reclamation (USBR 1984) criteria (USDI 1991) have been mainly used for the classification of the rock mass (Prakash 2001). Sub-horizontal weak geological layers having low shear parameters like red bole, tuff, argillaceous sandstone and intra-formational shears posed the sliding problems for the dam blocks. Weathered and sheared rock zones and faults having poor rock mass characteristics and different physico engineering properties posed the problem of differential settlement of dam foundations.

During progressive excavation of the underground powerhouse cracks were observed in the walls of the machine hall walls and collapses/ rock falls in the tunnels. Stability analysis and forensic geotechnical studies were done to know the causes of the development of cracks in the walls of the machine hall and collapses in part of the tunnels.

GEOLOGICAL SETTING

The Narmada project is located in the 'SONATA' (Sone-Narmada-Tapti-Lineament) rift zone (Graben) bounded by faults aligned in ENE-WSW direction. The 'SONATA' zone is also known as 'NSL' (Narmada-Son-Lineament) zone. This zone transacts the shield area of peninsular India into northern and southern blocks. It is characterized by the high gravity, positive isostatic, anomalous geothermal regime with relatively high temperature gradient and high heat flow, shallowing of magmatic crust, elevated 'Curie Point' and solidus of basalt geoisotherms and recurrent seismicity. It was reactivated several times in the geological past. Presence of a number of reverse faults in the area indicates compressive post Deccan trap activity. The area is still under compression as a consequence of northward movement of the Indian plate. It is evident from the continuity of seismic activities in this zone (Prakash and Srikarni 1998).

GEOLOGY

The project site is occupied by the Deccan basalt flows underlain by infra-trappean sedimentary rocks (Bagh beds) (Fig. 1). Basalt flows are of amygdaloidal, porphyritic and dense (aphanitic) varieties. Eight flows are exposed above bed level on the left bank and five flows on the right bank. Thickness of the individual lava flows varies from 7 to 56m. The sedimentary rocks (Bagh beds) comprising of quartzitic sandstone, argillaceous sandstone, shale, pebbly sandstone and limestone and basalt flows are sub-horizontally disposed. Contacts of some of the lithounits are sheared. A River Channel Fault has brought sedimentary rocks in juxtaposition with basalt flows at the dam base. Basic dykes exposed in the area are aligned in ENE-WSW direction. Tectonic imprints of both the Narmada and West Coast lineament trends are seen at the dam site.

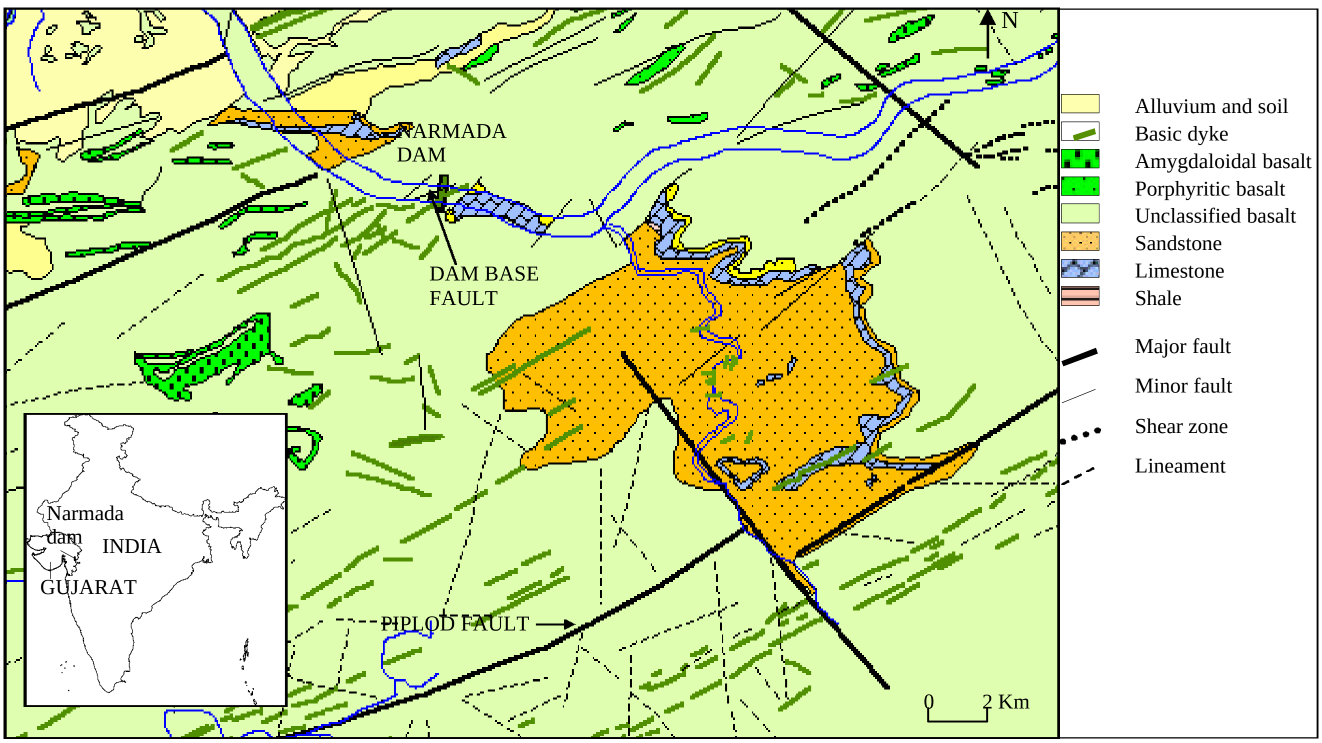


Fig. 1. Geology and location map of Sardar Sarovar (Narmada) Dam.

SEISMICITY

The project is located in the seismic zone III of the seismic zoning map of India (IS: 1893-1984 (1986)) in 'SONATA' zone. Seismic event in the area can have two aspects viz. vibrations due to shock and physical relative movements along the faults The data on earthquake occurrence in Peninsular India show that the Maximum Credible Earthquake (MCE) in this area can have maximum magnitude of 6.5. Based on the seismotectonic studies of the area the Piplod fault, which is a major closest fault (i.e. at 12 km shortest distance) to the dam site, has been assumed as causative fault for the aseismic design of the dam. About 15% of the epicenters of all earthquakes occurring in the area fall on the northern side of Narmada river and the rest 85% on the southern side of the river mainly along and adjacent to Piplod fault (Prakash 2002). No activity along river channel fault, located at the dam base has been observed prior and during the present stage of construction of the dam i.e. after partial filling of the reservoir (Elevation 100m). The horizontal seismic coefficient adopted for the Narmada Project is 0.125g (Prakash and Desai 2002). Seismic events occurring in the area are being recorded through a network of seismological observatories.

ROCK MASS EVALUATION AND GEOTECHNICAL PROBLEMS

Weak geological layers and features affecting the stability of the structures were identified and demarcated during construction stage engineering geological investigations. Major tests conducted at site included in-situ deformation modulus tests on fault zone material and surrounding rocks, in-situ shear tests for red bole, sheared contacts of sedimentary rocks and interfaces of rocks and concrete. Tracer studies were done for determining seepage losses through Limestone. Blast tests were conducted for estimation of design seismic coefficient. Photo-elastic studies and finite element analysis for deciding depth of concrete plug for the treatment of fault zone were done (Thatte et al. 1990). Hydro-fracture tests were done to know the stresses around the underground powerhouse. Stability of the underground powerhouse was assessed by Three Dimension (3-D) Finite Element Analysis (3-D FEM) and 3-D Distinct Element Code Analysis (3-D DEC). Based on these studies geotechnical problems of sliding, settlement and seepage were identified for the main dam. Problems of the underground powerhouse included development of cracks in the 57.5m high walls of the machine hall and collapses in part of the exit and draft tube tunnels (Prakash 2001).

MAIN DAM

Sliding

The dam may slide if the horizontal forces are excessive i.e. more than the forces resisting the slide at the boundary of the dam and the foundation, or along seams within the foundation.

A number of sub-horizontal to low dipping weak layers like red bole (ϕ=17º), tuff, agglomerate, shale, argillaceous sandstone layers (ϕ=17º) and intra-formational shears (ϕ=11º) were encountered in the foundation of spillway blocks having low values of shear parameters (Prakash 1990). Stability analysis indicated that the dam does not satisfy criteria for factor of safety against sliding as per Indian Standard specification (IS: 6512-1984 (1985)). Therefore, concrete shear keys were provided for the treatment of these weak layers (Mehta and Prakash 1990). Treatment was done on the similar line of treatment provided for the Itaipu dam (Brazil) (Moraes et al. 1982 and Parmar and Java 1990).

Treatment to red bole layer was provided in the foundations of spillway blocks 28 to 42 by excavating 3m wide drifts through approach shafts in grid pattern at right angle to each other leaving 4.5 x 8.5m rock pillar between them .The drifts were excavated in such a way that red bole layer was intercepted at mid height of the drifts and back filled with concrete. Open concrete shear keys were provided where rock cover was less than 5m (Fig.2).

Treatment to two layers of argillaceous sandstone and intra-formational shears occurring at about 10 to 18m below the general foundation levels in the foundations of Right Bank spillway blocks-44 to 51 was provided by excavating 3m wide and 3.6 to 6m (average height 4.5m; 2.5m+2m) high drifts through approach shafts in a grid pattern leaving rock pillar of size 8.5 x 8.5m.Tuff layer was removed from the crown of the

Fig. 2. Treatment of intra-formational shear associated with red bole layer.

Upper argillaceous sandstone treatment drifts. Crown of the upper drifts was excavated in the sound basalt and of lower drifts in the quartzitic sandstone. The bottom of the drift was kept just below the argillaceous sandstone in the quartzitic sandstone for proper keying (Fig. 3). The drifts replacing the lower and upper argillaceous sandstone are located directly one over the other separated by upper quartzitic sandstone (Fig. 4). Location of drift one above the other was planned to avoid concentration of stress on the weak layers and for easier drilling for contact grouting. In blocks 49 and 50, thickness of quartzitic sandstone in between argillaceous sandstone layers is less than a meter; hence, some of the lower and upper drifts were combined from safety consideration. The maximum height of the combined drifts is 12.5m. Consolidation-cum-contact grouting through holes spaced at 2m centre to centre

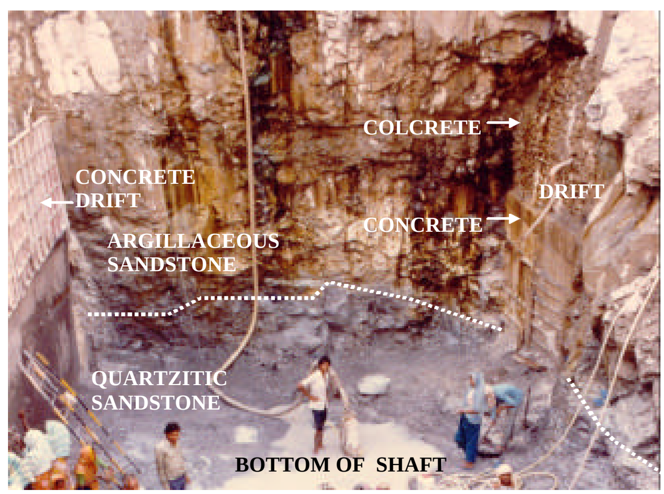
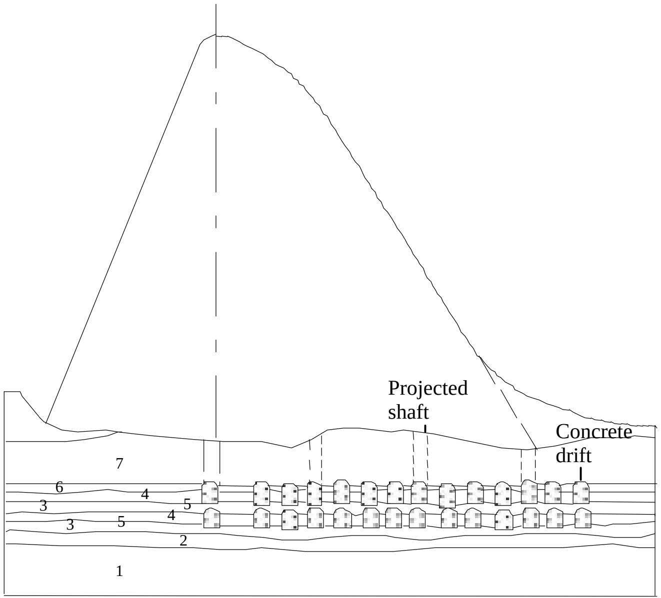


Fig. 3. Treatment of argillaceous sandstone through shaft and drifts.

(c/c) was done in grid pattern in the foundation of treated blocks to ensure good contact of the concrete, colcrete and rock.



1. Limestone 2. Pebbly sandstone (ϕ=45°) 3. Quartzitic sandstone (ϕ=44º) 4. Argillaceous sandstone (ϕ=17º) 5. Intra-formational shear-contact of argillaceous sandstone and quartzitic sandstone (ϕ=11º)
6. Tuff 7. Basalt

Fig. 4. Foundation treatment of two layers of argillaceous sandstone.

Alternative measures for the treatment included provision of mild curvature in the axis of the dam to mobilise shear resistance of all monoliths together and stilling basin type energy dissipater to provide passive resistance against sliding by protecting downstream rock from the scouring (Prakash 2001).

Settlement

The settlement problem is simple if the foundation rock is sound and strong and of one type. Differential settlement could be expected if the dam is placed on the foundations having varying lithounits of different physico-engineering properties. Faulting may also bring different types of rocks in juxtaposition to each other as in the case of Narmada (river channel) dam base fault (Fig. 5). The internal stresses thus imposed on the structure could be disastrous if not considered in design.

Narmada dam base (river channel) fault

This fault has brought sedimentary rocks in juxtaposition with the basalt at the dam base of four spillway blocks 41 to 44. It is aligned in N80ºE-S80ºW direction, dipping 60º towards N10ºW. This fault is reverse fault having displacement of the order of 210m with up throw side towards north i.e. towards Right Bank (Fig. 5). It is associated with 5 to 15cm thick gougey materials. Width of the fault zone is about 10 to 12m. Rock mass adjacent to fault zone is sheared and fractured.

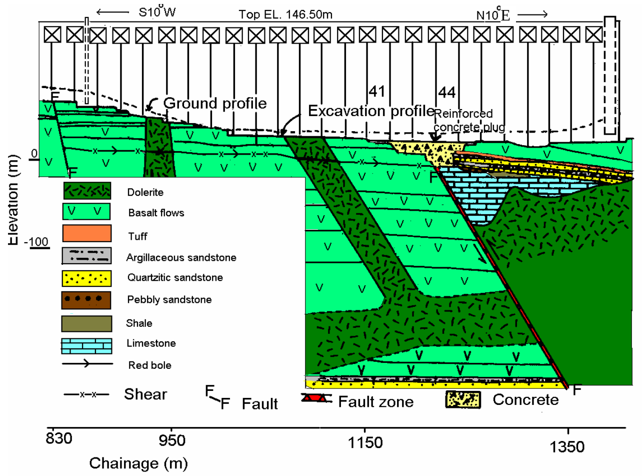
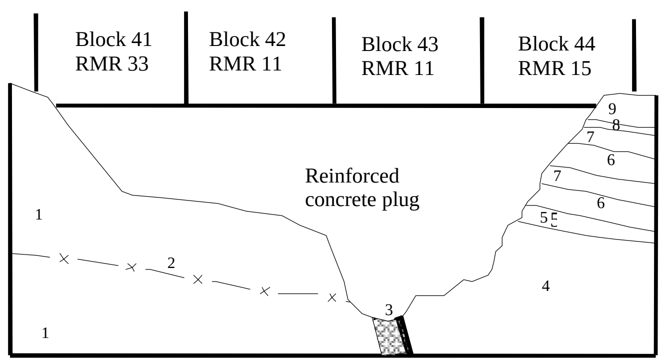


Fig. 5. Longitudinal geological section of dam showing sedimentary rocks in juxtaposition with basalt along dam base fault

Geotechnical assessment of the fault zone. Fault zone is almost unhealed (HL4-HL5) (USDI 1991). Rock mass rating of the fault zone material is very poor (RMR 11). In-situ test results indicated low values of modulus of deformation for the fault zone and relatively high values for the abutment rocks (Fig. 6). Ratio of modulus of elasticity and modulus of deformation of the basalt adjacent to fault zone vary from 1.87 to 2.4 and of sedimentary rocks from 2 to 4. In view of the low



1-Moderately jointed basalt (Me 5.3, Md 0.653) 2-Red bole
3-Shear/ Fault zone (Md o.40) 4-Limestone (Me 0.26, Md 0.45, RMR 63) 5-Pebbly sandstone (RMR 60, Md 0.055) 6-Quartzitic sandstone (RMR 63, Me 3.91) 7-Argillaceous sandstone (RMR 40, Md 0.55)
8-Tuff (RMR 40) 9-Basalt (Me 6.6)
(RMR-Rock Mass Rating Me-Modulus of elasticity Md-Modulus of deformation)

Fig. 6. Rock mass characteristics of dam base fault and abutment rocks.

modulus of deformation of fault zone and high modulus ratio of the abutment rocks of varying physico-engineering properties problem of differential settlement in the foundations of riverbed blocks 41 to 44 was apprehended (Prakash and Desai 2002).

Treatment. Based on two dimensional photo elastic studies depth of fault treatment plug was initially designed to be 1.5 times width of the fault zone but the actual treatment was carried out to a depth varying from 2.15 to 2.83m times the width depending on the site conditions and geotechnical judgment. The depth of the concrete (reinforced) plug provided was 34.5m in the upstream and 26m in the downstream (Mehta & Prakash 1990). In view of very poor quality of fault zone material (RMR 11) and poor quality of abutment rocks (RMR 11 to 33) this 34.5m deep concrete plug was reinforced to uniformly distribute the load and to safeguard against any local weak pockets, and to prevent differential settlement within the plug. For mobilising greater shear resistance high yield strength deformed anchor bars were also provided (Desai and Java 1983).

Treatment of weathered/ sheared rocks

Treatment to weathered / sheared rocks in the foundations of blocks- 3, 15 16 and 57 was provided in the form of reinforced concrete mat in single or two layers depending on the nature of weathering and foundation topography to uniformly distribute the load and to prevent differential settlement (Prakash and Srikarni 1998).

Seepage

In general appreciable seepage was not observed in the foundation through basalt flows except at few locations where flow contacts were sheared and weathered. Limestone occurring in the foundation of dam and in the reservoir is of siliceous nature (average SiO_2 20%) and non-cavernous. Local permeable pockets/zones in the limestone were treated by increasing depth of curtain grouting below the dam base. There is no possibility of piping of the fault zone material as the total length of seepage path under the dam and stilling basin is more than 2.5 H (Height of the water coloumn) (Prakash 2001).

UNDERGROUND STRUCTURES

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 utilize rock itself as structural material.

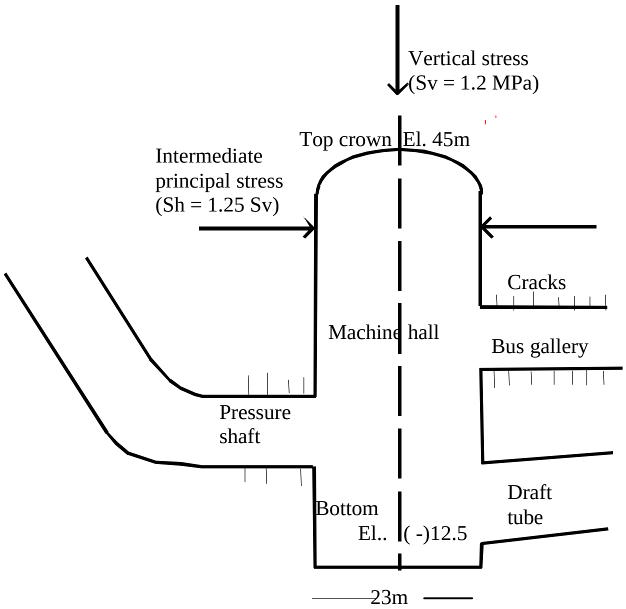
Machine hall

The rock mass of the machine hall belongs to fairly good category (RMR>60) except along and adjacent to shear zones (RMR 35-45). Six numbers of cross drifts from the central exploratory drifts were excavated from El. 45m to 39m. Powerhouse cavern was excavated to 5m, 9m and to its full width of 23m. The bench height varied from 2.5 to 4.0m. The initial support provided for the machine hall (cavern) consisted of 6 m long pattern rock bolts (25mm diameter) and two layers of 38mm thick shotcrete with wire mesh in between. Spacing of the rock bolts in the roof and walls was at 1.75m center-to-center (c/c) and at 2.5m c/c, respectively. In the middle third height of the wall (El. 13 to 33m), additional rock bolts of 7.5 m lengths were added to make the overall spacing of 1.52m c/c (Divatia and Trivedi 1990). 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 (up to 10m) and bus galleries (up to 17m). Minor rock falls in the crown were also observed at few places (Prakash and Sanganeria 1993).

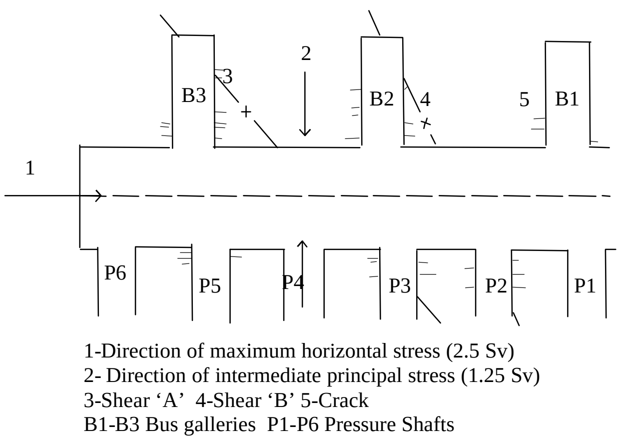
Forensic geotechnical studies of the machine hall

Despite the progressive visual observations of the development of cracks in the shotcrete/ concrete, instruments installed (Single and multi-point bore hole extensometers and stress meters) in the machine hall walls were not showing any movement. Therefore, new multi-point borehole extensometers were installed at critical places besides Demac joint gauges and Crack monitors to know further movement of cracks as well as walls. Fissures (cracks) in the distress area/ zones were continued to be monitored visually with the help of glass plates (Prakash 2003).

Nature of cracks in the walls of the machine hall. Cracks (fissures) developed in the pressure shafts and bus galleries are aligned parallel to the longer axis of the machine hall (Fig. 7 and 8). Sub-horizontal to low dipping cracks developed in the upstream and downstream wall, in en echelon pattern parallel to then excavated profile of the ramp. A few vertical cracks were also observed in the shotcreted walls of the machine hall near the major shear zones. To ascertain the extension of cracks inside the walls windows were opened at few places in the shotcreted walls.



Section of Machine hall



Plan of Machine hall

Fig. 7. Plan and section of machine hall showing disposition of cracks in the bus galleries and pressure shaft.

Stability analysis. Geological and 3-D FEM and 3-D DEC back analysis were done to investigate the cause of the development of cracks and also to know the present and future behaviour of the underground powerhouse cavern.

The geological stability analyses revealed that geological features were not responsible for the development of cracks in the upstream and downstream walls as the major shear zones in the machine hall are forming stable wedge and other major discontinuities are also not posing the problem of plain failure of the rock mass (Prakash and Srikarni 1998).

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.

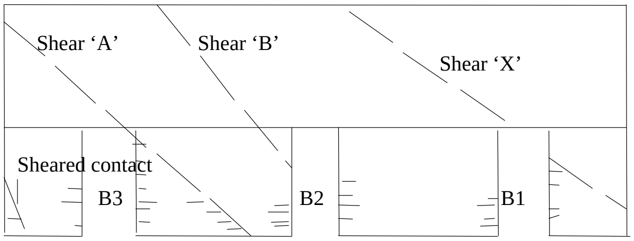
Three dimension (3-D) FEM analysis indicated that the maximum depth of the 1.0 Factor of safety (FOS) contour was at 10m distances in the upstream wall and 6.5m in the downstream wall. The factor of safety of the pillars in between Draft Tube Tunnels was sufficient but between the pressure shafts it was less than the 1.5. In the area affected by cracks safety factor contour of 1.5 extended up to 25m in depth.

Three dimension (3-D) DEC back analysis indicated 1.5 Factor of safety (FOS) contour in general was about 16 to 17m away from the face of the cavern wall i.e. inside the rock mass. However, around bus gallery-3 it was at a distance of 20m where maximum displacement was also observed (Prakash 2003).

Review of the design supports.

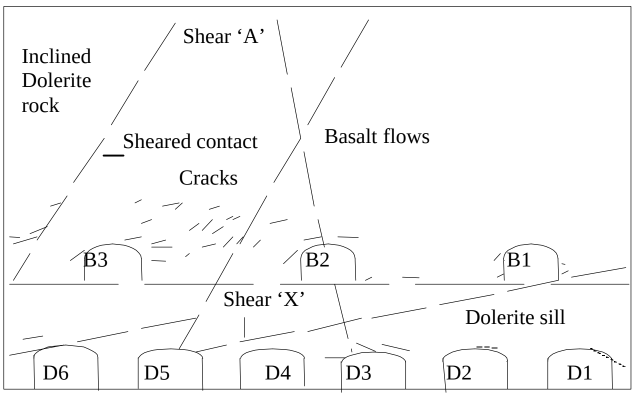
Review of the design supports from the various approaches (empirical approaches of Cording et. al. (1971), United States Corps of Engineers (1980), Hoek and Brown (1980), Barton et al. (1980)) and plot of rock bolt and cable lengths for arch support in various hydroelectric projects also indicated that the 6m long rock bolts used in the arch of the Narmada are adequate for permanent arch support. Performance monitoring for sixteen years also established that the roof of the cavern has remained stable. 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. Thus 6 to 7.5m long rock bolts installed earlier in the sidewalls 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, Hoek 1995, Prakash and Srikarni 1998)

Rock mass behaviour and development of cracks. The powerhouse cavern is having shallow rock cover (30 to 60 m). Cracks in the 57.5m high walls of the machine hall appear to be developed due to *stress relief* under low confining stress (Hoek 1995). This is analogous to the situation of excavating near vertical high slopes in jointed rock mass in the absence of adequate supports. Symmetry of pattern of cracks parallel to the longer axis of the cavern in the pressure shafts and bus galleries suggest that that these cracks are manifestation of gradual adjustment of loosened rock mass due to the deficiency of sidewall support. Visual observations of the pattern of cracks were identical as observed in the 3-DEC back analysis (Fig. 8).



Plan showing cracks as observed in the model studies at El. 20m.

(B1-B3 Bus galleries D1-D6 Draft tubes)



Section of the downstream wall showing cracks as observed in the model studies at 5m distance from the outer face.

Fig. 8. Cracks as observed in 3-D DEC discontinuum analysis (Model studies) of the downstream wall of the machine hall.

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, 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 seal the gaps of already loosened area (Prakash and Srikarni 1998).

Tunnels

The downstream water conductor system of the underground powerhouse (draft tube tunnels and Exit Tunnels) is located mainly in dolerite rocks (RMR = 30 to 45 and Q = 0.6 to 1.5) dissected by chlorite-coated joints and slaked rock zones (Fig. 9). Problems of rock falls and roof falls were experienced during excavation of tunnels through dolerite rocks (Fig. 10). 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 chlorite coated joints having low shear parameters. About 50% rock bolts were observed slipped in the slaked/ chloritized zone during tensioning. Pattern rock bolt (25mm diameter, 4 to 6m long spaced at 1.75m c/c) support with two layers of 38mm thick shotcrete with wire mesh in between could not prevent the roof falls/ collapses in the tunnel sections occupied by the chloritised, slaked and jointed dolerite rock. Therefore, rib supports were installed in the major part of tunnels (Prakash 1994). Rib supports besides providing positive supports removed the fear psychosis among the site staff for working inside the tunnels.

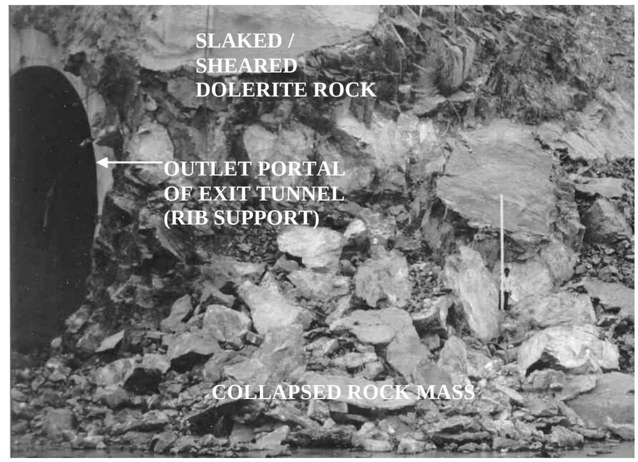


Fig. 9. Exposure of slaked dolerite rock at the outlet portal of the exit tunnel

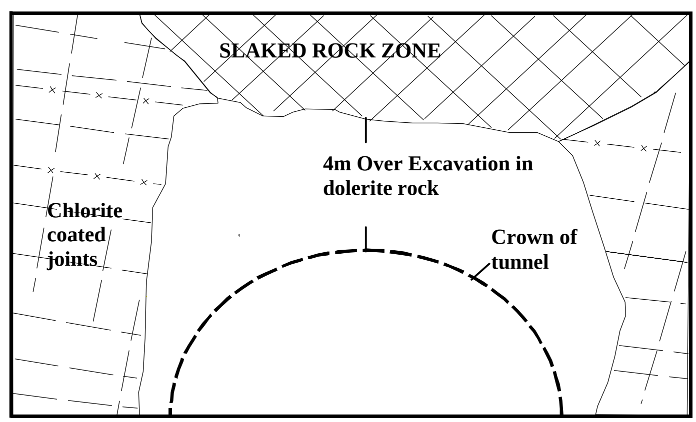


Fig. 10. Geological section of the draft tube tunnel passing through slaked dolerite rock.

CONCLUSIONS

Geotechnical study of the Narmada dam has shown that large dams can be successfully constructed on weak foundations based on the proper evaluation of rock mass. Foundation treatment for sub-horizontal geological weak features posing the problem of sliding included provision of concrete shear keys. Reinforced concrete mats were provided to treat the weathered and sheared rock mass and 34.5m deep reinforced concrete plug to prevent differential settlement of dam blocks located on river channel (dam base) fault.

Types and causes of the distresses during progressive excavation and construction of underground powerhouse structures were identified by observing behaviour of the rock mass visually as well as with the help of instruments. Based on the forensic geotechnical studies cables/ tendons and additional longer rock bolts were timely installed in the walls of the machine hall to stabilize the rock mass. Rib supports were introduced after observing collapses in parts of these tunnels. Pattern rock bolt support could not prevent the roof falls/ collapses in the tunnel sections occupied by the chloritised, slaked and jointed dolerite rock.

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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.

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

Desai, N.B. and Java, G.L. [1983] “Treatment of Geological fault in foundation of Sardar Sarovar [Narmada] Dam”, *52nd. Annual R&D session CBIP*, Technical session V, Paper No.2, pp. 13-31.

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

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

IS: 6512-1984 [1985] “Criteria for design of solid gravity dams”, Bureau of Indian Standards, New Delhi, pp. 1-22.

IS: 1893-1984 [1986] “Criteria For Earthquake Resistance Design Structures”, *Bureau of Indian Standards [IS]*, New Delhi., pp.1-77.

Mehta, P.N. and Prakash, I. [1990] “Geotechnical Problems and Treatment of Foundations of Major Dams on Deccan Traps in The Narmada valley, Gujarat, W. India”, *Sixth International Congress International Association for Engineering Geology*, Netherlands, pp.1921-1927.

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. [2001] “Engineering Geological Evaluation of the Major Dams in Lower Narmada Valley, Gujarat”, *Unpublished Ph.D Thesis*, MS University of Baroda, Vadodara, pp.1-148.

Prakash, I. [1990] “Geomechanical Properties of The Foundation Rocks at Sardar Sarovar [Narmada] and Karjan Dams, Gujarat”, *Journal of Engineering Geology*, India, Vol. XIX Nos. 3 & 4, pp.37-52.

Prakash, I. [1994] “ Experience of Excavation for Underground Structures Through Dolerite at Sardar Sarovar [Narmada] Project, Gujarat”, *Journal of Engineering geology*, India, Vol. XXIII, Nos. 1 to 4, pp. 25-37.

Prakash I. and Desai, N. [2002] “ Engineering Significance of fault-A Geotechnical review of Narmada Dam Base Fault” *IGC Conference on Geotechnical Engineering: Environmental Challenges*, Vol.1, Allahabad, , pp. 494-497.

Prakash, I. [2003] “Application of Forensic Geo-technical Engineering in the Stability Evaluation of Narmada Underground Powerhouse Cavern [Machine hall]”, Indian Geotechnical Society Workshop on Forensic Geotechnical Engineering, Chennai, pp. 44-44.

Thatte, C.D., Desai, N. B. and Datir, U.D. [1990] “GERI’s Role in Geotechnical Investigations for Sardar Sarovar”, *India Journal of Power and River Valley Development*, Special Number [September], pp. 237-248.

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

USDI [1991] “Engineering Geology Field Manual”, *United States Department of Interior (USDI)*, Scientific Publisher, Jodhpur, pp.1-598.