APPENDIX

top of the flows are marked by agglomerate/tuff with or without red bole, besides clinkery fragmented zone.

BRIEF GEOLOGY OF THE NARMADA DAM SITE

The dam site is occupied by basalt flows ('Aa' type) underlain by sedimentary sequence of Bagh Beds consisting of quartzitic, argillaceous and pebbly sandstones, shale and limestone Contacts of the sedimentaries are sheared. These are overlain by horizontally disposed 7 to 56 m thick basaltic flows containing dense, amygdaloidal and porphyritic varieties that are separated by agglomerate, tuff and red bole at interfaces. The Bagh Beds and volcanics are profusely intruded by dykes of basic composition that are aligned in ENE-WSW direction. One of the major dolerite dykes forms the right abutment of the proposed dam. Contacts of the dykes with the host rocks are highly sheared.

Sedimentaries have been brought in juxtaposition with the basalt at the base of the dam by a river bed fault aligned in N 80°E-S 80°W direction, cutting across the foundations of five spillway blocks. The fault zone is about 12 m wide. The basalt on the footwall side of the fault is highly sheared and fractured. About 15 cm thick clay gauge is confined to the hanging wall. Sedimentaries adjacent to the fault zone on the right bank are also fractured.

The fracture, shear and fault planes trend in NNE-SSW direction. Lineaments trending in NE-SW and NNW-SSE direction that are sympathetic to Narmada fault are also observed.

BRIEF GEOLOGY OF THE KARJAN DAM SITE

The rock types exposed in the area are a sequence of Deccan Basalt flows ('Aa' and 'Pahoehoe' type). These are mainly porphyritic and amygdaloidal in composition. Each identifiable flow is characterised by a fine grained or porphyritic dense besalt towards the base, turning vesicular/ amygdular towards the top. The interfaces of the consecutive flows are generally occupied by thin, highly weathered, ferruginious seams. The whole sequence of flows is traversed by ENE-WSW trending dykes. These dykes are absent in the foundation area of the dam. Persistent sets of joints, shears and faults trend in N-S to NNE-SSW direction.

Table-1 : Physico — Engineering properties of the foundation rocks

Rock type	Specific gravity	Percentag water absorption	Unconfined compressive strength kg/cm² (wet)	Poisson's ratio	Ultrasonic velocity M'sec.	Modulus of elasticity x 105kg'cm² (dynamic)	In-situ modulus of deformation of rock-mass x 105 kg cm²
1	2	3	4	5	6	7	8
Porphyritic basalt	2.77	0.52	794.00	0.315	5,670	6.6*	—
Massive bassait	2.90	1.12	617.01	—	—	7.26	0.140
Amygdaloidal basalt	2.82	1.47	469.09	0.31	4,460	5.30	0.653
Dolerite	2.94	1 50	643 30	0.33	5,670	7.33	0 130
Agglomorate	2.79	1.74	454.74	0 33	4,800	4.47	—
Sandstone	2.64	1.46	348.23	0.29	4,395	3.91	0.55
Pebby sandstane	2·74	1.20	502.69	—	—	—	0.055
Fault breccia	2.94	4.94	33.75	—	—	—	0.040**
Limestone	2.73	1.38	564.57	0.27	5,230	6.26	0.45

*Static modulus of elasticity. **Modulus of elasticity of fault zone.

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out on the selected location of foundations are summarised below (Table 2) :

Table-2 : Test results of in-situ deformability of foundation rocks, Narmada dam

Location	Remarks	Modulus of deformation x10^5 kg/cm^2	Molulus of eleasticity x10^5 kg/cm^2	Ratio of the two modulu
Ch. 637 m El. 90.75 m	Jointed basalt with pockets of agglomerate	0.04	0 075	1.87*
Ch. 859 m El. 48.0 m	Highly jointed calcified basalt	0.10	0.24	2.4
Construction sluice blocks	Calcified trap	0.243	—	—
	Calcified dole -rite	0.42	—	—
	Shear zone	0.31	—	—
Ch. 1190 to 1225 m	Fault zone	0.04	—	—
	Off-shoot of fault zone	0.162		
Ch. 1221 to 1228 m				
El. 0.72 to 1.08 m	Argillaceous sandstone	0.03	0.06	2
El. 2.07 to 2.28 m	Quartzitic sandstone	0.05	0.2	4

**Directon of loading horizontal.*

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**Directon of leading horizontal.*

The average values of the ratio of the modulus of elasticity and modulus of deformation of the basalt vary from 1.87 to 2.5 and indicate weathered jointed nature of the rock mass. Sedimentary rocks adjacent to the fault zone are also highly jointed as indicated by the high ratios of the two modulii (2 to 4). Low value of the modulus of deformation of the river bed fault zone (0.040 x 10^5 kg/cm^2) and high modulus ratios of the abutment rocks have posed the problem of differential settlement for the river bed spillway blocks. Treatment to the river bed fault has been provided in form of 34 m deep reinforced concrete plug.

Shear parameters of the foundation rocks and weak material have been obtained by laboratory and in-situ shear tests. Laboratory test results of the geomechanical properties of the shear zones are tabulated below (Table-3) :

Table-3 : Laboratory values of geomechanical properties of shear zones and red bole.

Location	Mechanical analysis %				Attenberg limit in %			Density F.D.D.	Test FMC	Box Shear Test		Remarks
	Gravel	Sand	Silt	Clay	LL	PL	PI	M.D.D. in gm/cc	OMC in %	kg/cm²	ϕ Degree	
Ch. 720 to 705 m	42	43	13	02	—	—	—	1.731	13.64	0.0	32.0 to 31.0	Sheared material of weathered joint in bassalt
Ch. 1094 to 1109 m	37	50	12	01	—	—	—	1.71	11.00	0.0	35 to 36 50	Material of shear zone across the dolerite dyke in the continuation of red bole layer
Ch 1117 to 1120 m	04	50	34	12	31	18	13	1.715	21.47	0.03	26 to 40	Shear zone in the basalt block 47
Ch. 1245 m	08	67	14	21	21	15	06	—	—	—	—	Contact material of amygdaloidal basalt and tuff layer.
Ch. 1245 m	09	58	19	14	21	14	07	—	—	—	—	Contact material of Plagonite tuff/Argillaceous sandstone.
Ch. 1397 to 1380 m	06	64	27	03	42	35	07	1.792	12.36	0.02 0 05	26.50 to 32.50	Weathered and sheared material along cotact of basalt flows block-50 & 51.
Right bank	80 to 90	6 to 19	1	2	—	—	—	1.68	—	0	36 to 39	Shear zone material of the sheared contact of the dolerite dyke. Permeability varies from 1.3×10^{-3} to 9.2×10^{-4} cm/sec.

Note:—1. Representative values of mechanical analysis have been considered.

2. Geomechanieal tests of red bole show 6 to 12% clay and angle of internal friction-29° to 30°.

The average values of the shear parameters obtaied by in-situ shear tests of various lithounits, sheared contacts, joints and red bole layer are as below (Table-4) :

TABLE-4 : AVERAGE VALUES OF SHEAR PARAMETERS OF THE FOUNDATION ROCKS AND WEAK ZONES (based on the in-situ shear test).

Shear Tests	Shear Parameters Cohesion 'C'	Angle of internal friction ϕ
Through Quartzitic sandstone	0	44°
Argillaceous sandstone	0	17°
Through upper contact of Argillaceous sandstone and Lower contact of Quartzitic sandstone	0	11°
Through Lower contact of argillaceous sandstone and upper contact of quartzitic sandstone	0	26°
Through contact of massive trap and agglomerate	0	18°
Through pebbly sandstone	0	45°
Through red bole	0	17°
Through contact of massive basalt and amygdaloidal basalt	0	47°
Through weathered joints in basalt	–	27° to 39°

Note : A few tests have also indicated very low values of cohesion. These values are recommended to be naglected and hence stated as zero.

Laboratory box shear tests of the red bole layer indicate value of 'ϕ' 29 to 30° (Table-3) while the in-situ shear tests gave the value 17° (Table-4). Therefore, in the design of the foundation treatment more realistic shear parameters based on insitu shear tests have been recommended to be adopted. Low value of shear parameters have been obtained for the argillaceous sandstone, sheared contacts of the sedimentaries, tuff red bole layer and some of the weathered and sheared joints in the basalt.

Red bole layer having low value of angle of internal friction (17°) posed the problem of sliding for the spillway blocks 28 to 40 located on left side of the river bed fault. Sheared contacts of sedimentaries, tuff and sheared joints in the basalt also posed the sliding problems for the spillway blocks 45 to 51, located on right side of the river bed fault. Open and underground concrete shear keys across the weak layers have been provided by excavating weak material and back filling it with concrete to increase the shear friction factor of these blocks.

KARJAN DAM

The foundation of the Karjan dam is occupied by the dense porphyritic and any amygdular varieties of basalt. Numerous weathered seams pinching and swelling in nature, measuring 1 cm to 1 m in thickness are encountered at intervals of 3 to 5 m dissecting the foundation rocks.

The average values of the physico-engineering properties of the rock core samples of the foundation rocks are tabulated below (Table-5).

Table-5 : Engineering properties of the rock cores of Karjan Dam

Rock type	Water absorption %	Porosity %	Specific gravity Apparent	Specific gravity True	Permeability 'k' in cm/sec.	Unconfined compressive strength in Mpa.	Tensile strength (Brazilian) in Mpa
Porphyritic basalt	2.20	4.92	2.73	2.69	2.18×10^{-9}	62	12.00
Amygdaloidal basalt	1.65	4.57	2.76	2.86	0.0×10^{-9}	73	12.50
Amygdaloidal porphyritic basalt	1.84	5.15	2.80	2.58	2.73×10^{-9}	74	10.00

The average values of the water absorption (%) varies from 1.48 to 2.20, porosity (%)—4.57 to 5.15, specific gravity—2 58 to 2 86, unconfined compressive strength from 62 to 74 Mpa. These values are comparable with the foundation rocks of the Narmada dam and are within the normal limit of fresh, moderate to good rock values of basalts.

CHARACTERS OF THE WEATHERED SEAMS

The weathered seams consist of sheared and weathered rock pieces associated with thin clayey material at places. These are of wavy, branching and of pinching/swelling nature. Thickness of the seam veries from a few millimetres to a metre. Zeolite/calcite infilling have been noticed along some of the weathered seams.

In-situ shear tests carried out on the weathered seams in the foundations of overflow and non-overflow blocks indicated value of cohesion 'C'= 0 kg/cm² and the value of angle of internal friction 'ϕ'=22° to 26°. Low values of shear parameters have necessitated provision of concrete shear keys along the weak layers at the toe of the dam blocks to resist the sliding forces.

CONCLUSION

The geomechanical properties of the Deccan Basalt and Infra-Trappean Sedimentaries in the Narmada valley of Gujarat including specific gravity, water absorption, porosity, unconfined compressive strength, modulus of deformation-elasticity and shear strength have been obtained for the classification of the foundation rocks and for the consideration in the design of the dam and providing remedial measures.

Deep reinforced concrete plug has been provided in the river bed fault at Narmada dam site in view of the low modulus of deformation value of the fault zone material and high modulos ratio of the adjoining rocks. Concrete shear keys have been provided in the foundation of Narmada and Karjan spillway blocks in order to prevent them against sliding along sub-horizontal weak layers having low values of shear strength.

The different units of basalts and also that of the sedimentaries in the Narmada valley have varied characteristics. The variations in the physico-

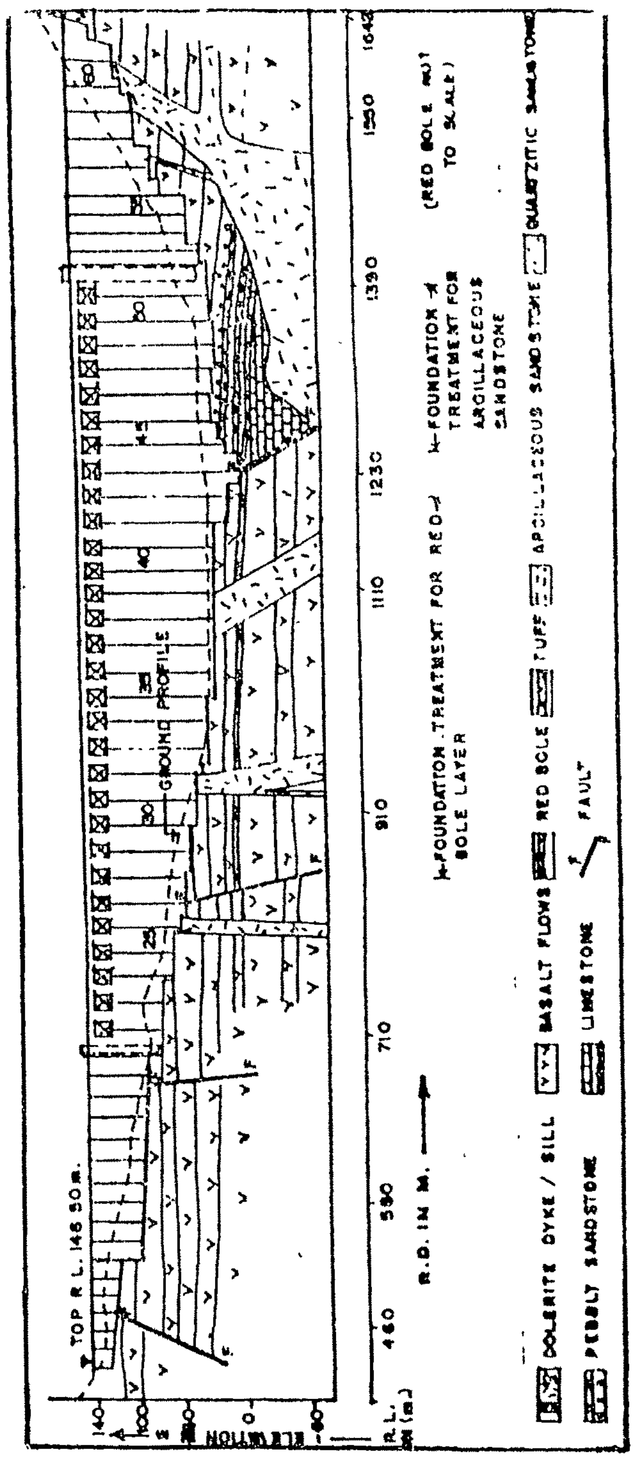
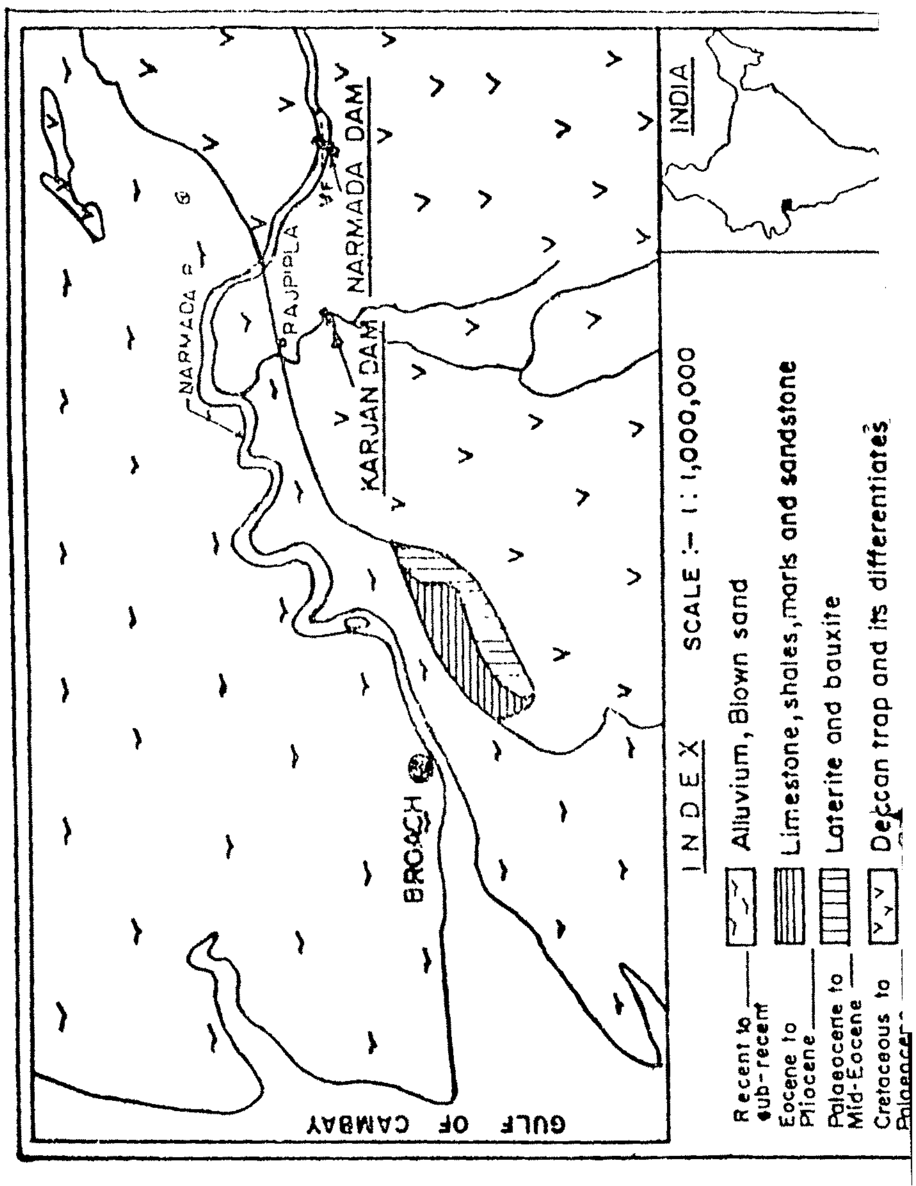


FIG. 2: GEOLOGICAL SECTION OF NARMADA DAM


GULF OF CAMBAY
BROACH
NARMADA R
RAJPIPLA
KARJAN DAM
NARMADA DAM
INDIA
INDEX
SCALE :- 1:1,000,000
Recent to sub-recent
Alluvium, Blown sand
Eocene to Pliocene
Limestone, shales, marls and sandstone
Palaeocene to Mid-Eocene
Laterite and bauxite
Cretaceous to Palaeocene
Deccan trap and its differentiates

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Geotechnical problems and treatment of foundation of major dams on Deccan traps in the Narmada Valley/Gujarat/Western India

Les problèmes géotechniques et traitement de la fondation des barrages majeurs sur les cascades de Deccan dans la vallée de Narmada/Gujarat/de l'Ouest de l'Inde

P.N.Mehta & Indra Prakash
Engineering Geology Division, Geological Survey of India, Jaipur, India

ABSTRACT: Foundation conditions of two major concrete gravity dams located across Narmada and Karjan rivers on Decan traps in the Narmada Valley have been evaluated. Weak foundation media in the Deccan basalt flows are tuff, agglomerate and red bole. Besides, the rocks are dissected by joints, shears and faults. Older sedimentaries have been brought in juxtaposition with the basalt by faults in the vicinity of the Narmada site.

A red bole layer and argillaceous sandstone occur below the foundation of the Narmada dam and weathered seams lie in the foundation of Karjan dam. In-situ shear tests carried out on these sub-horizontally disposed weak layers have indicated low values of cohesion (C) and angle of internal friction (Ø) posing sliding problems of the spillway blocks.

Foundation treatment at Narmada dam site, inter-alia, includes excavation of the drifts in the grid pattern aggregating more than 10 km length along the sub-horizontal weak layers and back filling with concrete and also by providing open concrete shear keys to have adequate resistance against sliding. Similar treatment has been done in the Karjan dam foundation.

Treatment of the river bed fault at Narmada dam site has been provided in the form of a 34 m deep reinforced concrete plug after detailed geotechnical and photo elastic studies.

RESUME: Les conditions des foundations des deux mayeurs barrages de concrets gravite situe sur less rivieres Narmada et Karjan sur less trappes de Deccan dans la vallie de Narmada, avaient etc. evalues. Le millieu feuble de la foundation dans l 'e' coulement basalte,de, Deccan sont tufs, agglomere et bol rougl. Ces e' coulements sont dissiques par les joints, Usailles et failles. Les sediments plus vieux e'laient juxtaposes anee la basalte par les failles en proximite a l'emplacement de Narmada.

Une couche de bol rouge et les bandes agrileux du gres se produisent desous la foundation de barrage de Narmada et les couches atterees se trounent dans la foundation du barrage de Karjan. Des e'preune des u'sailles conduites un-site sur ces faibles counches dispose' sub-horizontallement ont indique's les valeurs basses de cohesion et un angle du friction interne posant les problemes de glissement des blocs passe - deversoir.

Le traitement de la foundation a' l'emplacement du barrange de Narmada inter-alia a inclue IIexcavation des monceaux en modele de grille, agreglant plus que 10 Km en longeur, belong des couches faibles sub-horizontal et remplissant de concret et aussi fournissant les cles' ouvertes des cisailles de concret pour avoir une resistance adequate contre le glissement un traitement pareil a' etc' failt dans la foundation du barrange de Karjan'.

Le traitement de la faille due lit de riviere a 'et' fourni a 'l' emplacement du barrage de Narmada en forme d'un tampon de concrete renforce' apres les e' tudes ge' otechniques et photo- elastiques de'taille' es.

INTRODUCTION

The Narmada is the largest of all the west flowing rivers debouching in the Arabian sea It has a total length of about 1311 km from its source to sea with an average maximum flood discharge of 69,375 cumecs and minimum being 8.5 cumecs. In order to harness the vast irrigation and hydro-electric potential of the river, it is proposed to construct a 129 m high (above the bed level), and 1270 m long concrete gravity dam to generate 1450 MW power and irrigate 1,792 M. Ha.

The Karjan river is one of the Major tributaries of the Narmada and joins about 25 Km downstream of the Narmada dam. The 100 m high and 903 m long masonry-cum-concrete dam has been completed at about 22 km from the Narmada dam (Fig.1) in 1986 for irrigation.

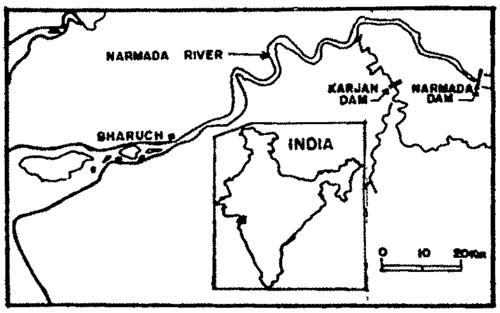

Fig.1-Location plan of Narmada and Karjan Dams

Detailed rock mechanics laboratory and field studies have been done at these sites for obtaining the geomechanical properties of the individual lithounits. The main geotechnical problems at the project sites were related to the presence of a number of weak layers in the foundation of Narmada and Karjan dams which have necessiated special measures in the design and treatment. These have been studied in detail and have been discussed in the paper.

GEOLOGICAL SETTING

The Deccan basalt flows of Cretaceous-Eocene age cover a vast area of Gujarat. The traps comprise numerous flows of 'Pahoehoe' 'Aa' and intermediate types of basalts inttruded by basalt and dolerite dykes. In the Narmada river section near the dam site, thin Cretaceous sedimentary rocks, normally underlying basalt flows occur in the form of inliors.

The Narmada and Karjan dams are located in a graben bounded by faults parallel to the Narmada-Son-lineament aligned in ENE-WSW direction. The Narmada-Son lineament is considered to be a major tectonic boundary-a geofracture dividing the Indian shield into a southern penisular block and a northern foreland block (Biswas 1983). Recent studies have indicated that Central Indian shear Zone is following Son-Narmada and Tapi lineaments. It represents an intercontinental collision suture and joins with Singhbhum shear on the east and Delhi-Aravalli fault on the west in a Sickle shaped trend (Dutta, 1988).

Geology of the Narmada and Karjan dam sites

The Narmada dam site is occupied by basalt flows ('Aa' type) underlain by sedimentary sequence of Bagh beds (Cretaceous). The lava flows have 5° to 15° dips. The flows are characterised by fine grained or porphyritic, hard and dense basalt. Agglomerate and red bole layers are also present at the top of some of the flows. Thickness of individual lava flows varies from 7 to 56 m. The sedimentary rocks known as Bagh beds comprise quartizitic sandstone argillaceous sandstone, shale and pebbly sandstone and limestone. Contacts of the sedimentaries and some of the flows are sheared. The sedimentaries and basalts are profusely intruded by dolerite and basalt dykes aligned in ENE-WSW direction. One of the major, 40 m wide, dolerite dyke is exposed on the right abutment. Contacts of the dykes with basalts are highly sheared.

The fracture shear and fault pattern in the area trend in ENE-WSW direction. The discontinuities aligned in NE-SW direction are also observed. A river bed fault trending almost in E-W direction has brought the sedimentaries in juxtaposition with basalts.

At the Karjan dam site, Deccan basalt flows of 'Aa' and 'Pahoehoe' type are exposed in the river bed and the abutments. A characteristic feature of the rocks in this area is the presence of weathered rock seams at the interfaces of many flows. Thickness of the individual flows varies from 3 to 30 m. The 'Aa' flows are characterised by fine grained or porphyritic hard or dense basalt towards the base and become amygdular or tuffaceous twoards the top.

Persistent sets of joints, shears, faults trend almost in N-S to NNE-SSW direction. At the dam base, a 12 m wide river bed fault trending in N-S direction traverses the foundations of three blocks. Joints and fracture trending in ENE-WSW direction are also observed indicating tectonic imprint of both West Coast and Narmada-Son lineament in this area.

GEOTECHNICAL PROBLEMS AT NARMADA DAM SITE

Problem of sedimentaries and tuff:

On the right bank infra-trappean sedimentaries under-lying basalt flows are encountered in the foundation of spillway blocks-44 to 51. These comprise argillaceous sandstone quartzitic sandstone, pebbly sandstone shale and limestone of varying thicknesses.

The sedimentaries are gently dipping towards N5^{o}- 30^{o}W (right bank). Tuff layer is of pinching and swelling in nature and highly sheared. Argillaceous sandstone is hard but highly sheared and fractured in the proximity of the river bed fault. The quartzitic sandstone is hard and competent and has widely spaced joints. The pebbly sandstone disintegrates under saturation. The shale occuring as lenses is hard and competent. The limestone is of non-cavernous nature, but has a few open joints.

The contacts of the litho-units are sheared. The sheared contact material along the bedding planes of lower and upper argillaceous sandstone is gougy and contain clayey material with sheared rock pieces. Its thickness varies from a centimeter to 20 cms. The average values of the shear parameters obtained by in-situ shear tests of various litho-units and their sheared contacts are given in table :

Table: Average values of shear parameters of the sedimentary rocks and sheared contacts.

Shear tests	Shear Parameters Coheision 'C'	Angle of internal friction ø
Through quartzitic sandstone	0	44^{o}
Through argillaceous sandstone	0	17^{o}
Through upper contact of argillaceous sandstone and lower contact of quartzitic sandstone.	0	11^{o}
Through lower contact of argillaceous sandstone and upper contact of quartzitic sandstone.	0	26^{o}
Through pebbly sandstone	0	45^{o}

In view of the low in-situ shear test results obtained at the contacts argillaceous sanastones and quartzitic sandstone, present in the foundations of spillway blocks-44 to 51, treatment for safety against sliding was provided. The treatment envisages removal of the argillaceous sandstone layers and its sheared contacts by excavating 3 m wide and 3.6 - 6 m high drifts in a grid pattern, leaving rock pillars of size 8.5 x 8.5 m in between adjacent drifts and back filling the drifts with concrete/colcrete.The drifts replacing the Lower and Upper argillaceous sandstone are lying directly one over the other separated by upper quartzitic sandstone (Fig.2). For proper

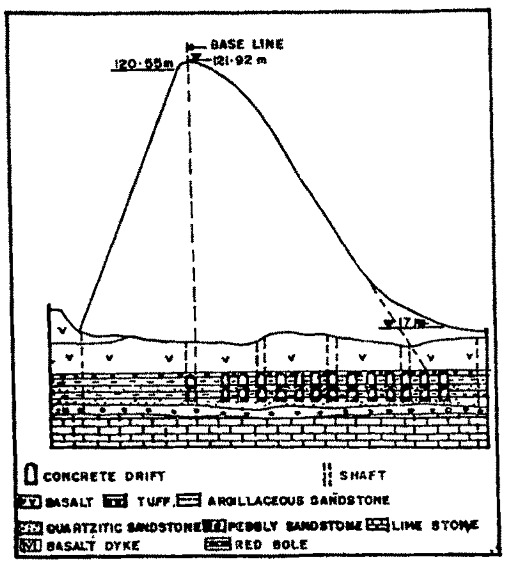

Fig.2 Typical Section of foundation treatment of argillaceous sandstone layers

keying, crown of the upper drifts have been excavated in the sound basalt and of lower drifts in the quartzitic sandstone. Tuff layer was removed from the crown of the upper argillaceous sandstone treatment drifts.Some of the lower and upper drifts in block 49 and 50 are combined due to thining of quartzitic sandstone in between argillaceous sandstone layers. Maximum height of the drifts is 12.50 m.

The drifts were excavated aggregating 5186 running metres through 26 shafts of 3 to 4 m diameter. Maximum depth of the shaft is 40 m in the block-50 between EL(+) 24 m and (-) 16 m. Total rock excavation involved through shafts and drifts for the treatment of argillaceous sandstone was 88870 cubic metres.

In order to ensure a good contact of the roof rock with concrete/colcrete, consolidation-cum-contact grouting through holes spaced at 2 m c/c was done in grid pattern from the foundation rock. It would also help in filling the shrinkage gap between concrete and rock, open natural and blasted joints and other discontinuities in the foundation.

Red bole: A red bole layer having rolling dip of about 5^{o} to 15^{o} towards upstream was delineated in the foundations of left bank spillway blocks 28 to 42 between El 22 and (-) 10 m. Its continuity towards right bank is cut by the river channel fault. The red bole layer separates two amygdaloidal basalt flows which are intruded by two roughly parallel NE-SW trending dolerite dykes. shearing was observed along the red bole layer and also across the dolerite dykes in the lateralcontinuity of the bole layer. Shearing has resulted in the formation of gougy material along the red bole. The thickness of the red bole layer varies from 5 cm. to 40 cm. When soaked in water, it crumbles to powder immediately.

Mechanical analysis of red bole material indicate6 to 12% clay. Laboratory tests of remoulded samples of red bole and sheared material across the dolerite dyke gave values of angle of internal friction as 29^{o} to 30^{o} and 35^{o} to 36.50^{o}, respectively. In-situ shear tests on red bole indicated the value of cohesion 'C' as 0 Kg/Cm^2 and 'Ø' as 17^{o}.

The stability analysis indicated possibility of sliding along the low dipping red-bole layer occuring about 10 to 30 m below the foundations and warranted suitable remedial measures. Treatment to the red bole layers was provided in the foundations of spillway blocks-28 to 42 by excavating 3 m wide drifts in a grid pattern, leaving 4.5 x 8.5 m rock pillars in between them. These drifts were back-filled with concrete to increase the shear friction factor of these blocks against sliding. Earlier the treatment was designed only for the foundation area occupied by the red bole layer and the higher value of angle of internal friction 'Ø' (48^{o}) was considered for the area occupied by the dolerite dyke. During the excavation of drifts, shearing across the dolerite dyke in the continuity of the red bole layer was observed in the foundation of spillway blocks 28 to 33 and 34 to 40. Accordingly, treatment to this shear-zone was also provided considering value of 'Ø' as 36^{o}. Total area of the treatment of red bole in each block is approximately seventy percent of the foundation area. Total rock excavation through shafts and drifts aggregating to 4858 m length was 47590 Cu.m.

River bed fault: A river bed fault is exposed at the dam base cutting across the foundations of five spillway blocks 41 to 44. It is aligned in N80°E - S80°W direction dipping 60^{o} towards N10°W. The sedimentaries on the right bank have been brought near the surface in juxtaposition with the basalt by this fault. The length of the fault is about 1.6 km and width at the dam base is 12 m. The basalt on the footwall side is highly sheared and fractured.

About 15 cm thick clay gouge is confined to the hanging wall. In view of this fault, foundation of blocks 41 to 44 are having different lithological units of varying physico-engineering properties on both sides of the fault.

In-situ test results have indicated low values of the modulus of deformation for the fault zone ($0.05 \times 10^{5} Kg/cm^{2}$). The high values of the modulus of deformation has been obtained for the basalt (0.52×10^{5} Kg/Cm^{2}) and sandstone (0.38×10^{5} Kg/cm^{2}). The average values of the ratio of the modulus of elasticity and modulus of deformation of the basalt vary from 1.87 to 2.5 indicative of weathered and jointed nature of the rock mass. Sedimentary rocks adjacent to the fault zone are also highly jointed as indicated by the high ratio of two modulus (2 to 4). In view of the low value of modulus of deformation of fault zone and high modulus ratio of the abutment rocks of varying pysico-engineering properties, problem of differential settlement in the foundations of river bed blocks 41 to 44 was apprehended.

Treatment of the fault envisaged part removal of fault zone material by excavating a trench and back filling with concrete. For the determination of the depth of the fault plug two dimensional photo-elastic studies were done These studies have shown that plug depth of about 1.5 times the width of the fault zone would be adequate for the treatment of fault zone.For plug depth equal to 1.5 to 2 times the width of the fault zone stress distributions are similar to those obtained in roof of the inspection galleries of the dam (Desai, 1983).

The normal foundation level of the dam blocks adjacent to river channel fault is about EL (+) 18 m. As a part of the treatment, fault zone material was removed by excavating a trench in the foundation of river bed blocks down to 34 m depth (EL (-) 16 m) in the upstream and 26 m depth (EL (- 8.0 m) in the down stream of the dam. Thus the actual depth of the fault zone treatment was about 2.15 to 2.83 times the width of the fault zone.

Hammock reinforcement consisting of four layers of 36 mm diameter high yield strength deformed steel bars, spaced 50 cm para-

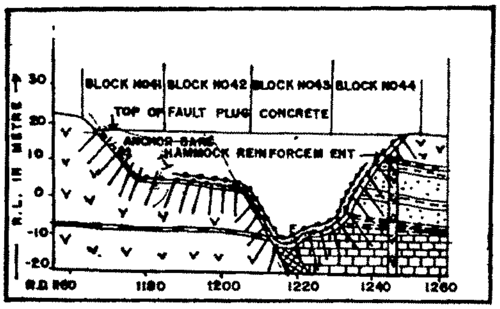

Fig.3 Geological Section across fault zone (for Index see Fig.2)

llel to dam axis in one direction and parallel to strike in other direction was provided in the fault plug to uniformly distribute the load and to safeguard against any local weak pockets, and to prevent differential settlement within the plug(Fig.3). For mobilising greater shear resistance high yield strength deformed anchor bars 36 mm diameter, 8 m longg in a grid of 3x3 m were also provided.

Treatment of weathered zone below block-16

Foundation of the block-16 is occupied by a slightly weathered to fresh, highly jointed amygdaloidal basalt layer. A weathered zone is present 5 to 6 m below the general foundation level. Thickness of this weathered zone varies from 0.1 to 1.5 m. In-situ deformability tests were carried out at this location. The average value of deformation modulus is 0.04×10^5 Kg/cm^2 and of modulus of elasticity is 0.075×10^5 Kg/cm^2. The ratio of the two modulii (1.87) also indicates weathered and jointed nature of the rock mass. From the central part of the foundation area this weathered rock zone was removed during the excavation of the trench for the in-situ tests and later as a part of the foundation preparation. Therefore, part foundation of the block is free from the weathered rock but major part is lying over weathered rock zone having 5 to 6 m rock cover. In the finite element analysis, maximum settlement observed at the toe of this 50 m high block was 0.5 cm with the applied stresses, between 18 and 22 kg/cm^2. Corresponding tension observed near the upstream heel was 1.8 kg/cm^2. As a remedial measure for uniformly distributing the load and to prevent differential settlement, if any, two tier reinforcement of 33 mm diameter tor steel bars spaced at 0.6 m c/c was provided in entire foundation area of the block.

GEOTECHNICAL PROBLEMS AT KARJAN DAM SITE:

The foundaton rocks at the Karjan dam site are similar to the Narmada dam and comprise the sub-horizontal to low dipping Deccan basalts which are jointed and faulted with a number of weathered seams. These weak features had posed serious geotechnical problems necessitating careful evaluation and suitable treatment.

Weathered seams

In the foundation area of overflow and non-overflow sections, numerous weathered seams are encountered at intervals of 3 to 5 m within the baslatt flows. A number of exploratory drill holes conclusively established presence of four seams in the foundation of spillway blocks within a depth of 20 m from the general foundation level. The weathered seams have been delineated in the entire foundation area below the dam base during extensive drilling programme and foundation excavation after the construction of part of the dam

Nature of the weathered seams

The weathered seams consist of a zone of highly to completely weathered and shattered basalt varying in thickness from a cm to 2 m. A thin persistent layer of clay is observed along the weathered seam. Thickness of clayey material (7 to 9% clay) varies from a few mm to 2 cm. Zeolite and calcite infilling have been noticed at places besides slicken-sided surfaces within the seam. The seams developed at the interfaces of the lava flows and along some of the open joints possibly as a result of selected weathering due to percolation of water along these planes. The presence of slicken-sided and gougy material at places and also lateral displacement of the vertical shear zones are clear evidences of movement and shearing along some of the seams. Conspicuous seepage was noticed through these seams in the foundation of river bed blocks.

Sliding problem

These weathered seams may act as potential planes for sliding of dam blocks. In situ shear tests carried out on the weathered seams in the foundations of overflow and non-overflow blocks indicated value of Cohesion 'C' = 0.Kg/cm^2 and the value of angle of internal friction '$\emptyset$' = 22° to 26°. In view of the low values of shear parameters and continuity of these seams in the foundation, sliding stability analysis of the spillway blocks and non-overflow block was done. Based on these analysis concrete shear keys were provided in the foundation of spillway blocks against sliding. The shear keys are located at downstream one third foundation area of the blocks where stresses are likely to be maximum under impounding condition (Fig.4).

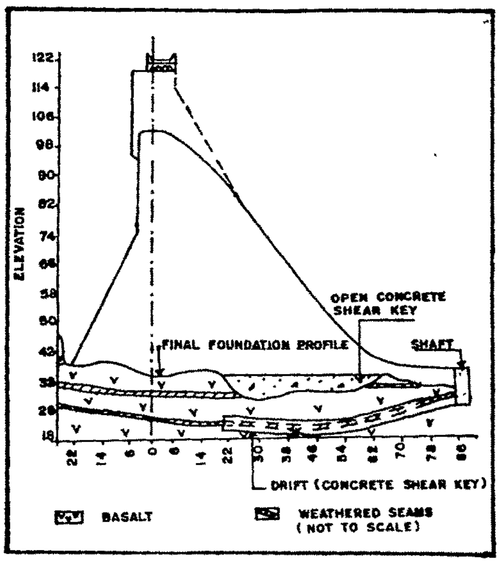

Fig.4 Typical section of foundation treatment of weathered seams

In part of the foundation area, where excavation was carried out below the weathered seams, shear parameters of the contacts of masonry to rock (C = 7 Kg/Cm^2 and $\emptyset$=45°) and concrete to rock (C = 14 Kg/Cm^2 and $\emptyset$ 45°) were considered based on the insitu tests.

Treatment to the shallow seams underlain by 3 to 5 m good basalt rock cover was provided by excavating trenches and back filling them with concrete. For the deeper seams and where the blocks were already partly constructed, drifts were excavated from the approaches through shafts located in the apron area and back filled with concrete without affecting the normal concreting work in the dam blocks (Fig. 4)

Settlement and Seepage Problems :

Weathered seams in general are of pinching and swelling in nature and of wavy disposition, no serious problem of settlement of dam blocks. But the sub-horizontal weathered seam met in the foundation of left non-overflow block-L3 was a thick seam, just 4 to 5 m below general foundation level. Thickness of the seam near the heel and toe of the block was 2 m. Minimum thickness of the seam was observed at the axis of the block as 0.5 m. Part foundation of the block, near block-L3/L4 joint, was free from seam. In view of the highly decomposed nature of the thick seam underlying shallow rock cover in part foundation of the block-3 problem of differential settlement was apprehended. However, on engineering analysis and judgement, no treatment was considered necessary and this 60 m high block has not shown any distress after three years of completion of the dam.

In order to check seepage along the weathered seams additional curtain grounting has been done in the entire foundation of the Karjan dam. Despite that, seepage observed in the drainage galleries have not reduced appreciably. Further, grouting is being carried out to check the seepage.

posing

River channel fault:

Karjan river flows from south to north in an 13 km long straight channel governed by a fault. At the dam site the N-S trending vertical river channel fault cuts through the foundation of three river bed blocks. Width of the fault zone is about 12m. Fault zone material consists of hard sheared basalt associated with thin gougy material at places. After 2 to 3 m stripping of bed rock from the bed level of the 12-14 m deep gorge portion of the river channel, rock mass forming the fault zone was found competent. Contact grout hole pipes were embeded in the concrete to seal the shrinkage gap. A mat containing nominal reinforcement extending 3 m on either side of the block joint was provided on top of the plug to prevent propagation of cracks upward in case of the cracking of the concrete plug at the block joint due to shrinkage or widenining of the rock-concrete gap. Additional drainage was provided through lognitudinal and cross - galleries in the body of the dam in the fault zone area besides intensive consolidation and curtain grouting.

Shear zone :

A 2 to 6 m wide shear zone traveres the foundation of 35 m high block - L_1 from upstream to downstream, trending in $N50^{O}W$ - $S50^{O}E$ direction. The shear zone is dipping at an angle of 60^{O} towards SW,i.e. towards the abutment. Shear zone material consists of highly to completely weathered basalt with calcite/ zeolite veins.As a foundation strengthening measure, soft sheared material was scooped out from the foundation area down to 4 to 5 m in depth. Hammock reinforcement was provided in the excavated trench and back filled with concrete. The width of the shear zone is maximum at the toe, i.e., about 6 m, where stresses are also likely to be high. Tor steel of 25 mm diameter at 300 mm c/c both ways were provided in the downstream one third area in the high stress zone and at 500 mm c/c both ways in the remaining part of the foundation area. To mobilise greater shearing resistance 32 m diameter anchor rods going down 5 to 9 m deep inside the good rock at spacing varying from 1 to 3 m were also provided and intensive grounting was carried out.

Conclusion :

The Deccan basalt is generally considered to be competent rocks for the dam foundation but the geomechanical properties and engineering behaviour of individual litho-units of basalts vary laterally as well as in depth. These basalt flows posed varied foundation problems like sliding, settlement and seepage necessitating adoption of special measures in the design and treatment of the Narmada and Karjan dams foundation.

The foundation problems emanating due to the presence of infra-trappean sedimentaries, having varying physico-enineering properties, in juxtaposition with the basalt at the dam base of Narmada dam are unique in this area. Therefore, extensive geological studies and geotechnical investigations are called for at each site for the quantitative assessment and evaluation of the foundations of the dams located on the Deccan traps.

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ƎOTECHNICAL PROBLEMS AND TREATMENT OF WEATHERED ROCK SEAMS OCCURRING IN THE FOUNDATION OF KARJAN DAM, WESTERN INDIA

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'RACT

Karjan dam is located on the Deccan Basalt flows of Cretaceous - Eocene age in the Narmada valley in Gujarat state. ıracteristic feature of the basalt flows in this area is conspicuous presence of a number of sub-horizontal weathered rock ; posing the foundation problems of settlement, sliding and seepage. Concrete shear keys have been provided to increase liding stability of dam blocks besides other remedial measures. Additional curtain grouting has been done after letion of the dam to reduce seepage.

,WORDS

n dam, Weathered rock seams, Settlement, Sliding, Shear friction factor, Concrete shear keys, Seepage.

:ODUCTION

100m high and 903m long masonry-cum-concrete gravity ın dam has been constructed across the narrow gorge of Karjan River, a tributary of Narmada River in the year 1986 nainly irrigation (Fig. 1). Detail surface and sub-surface ogical investigations have been carried out in the area les rock mechanics studies. A number of sub-horizontally osed weathered rock seams were detected and precisely ıeated during construction stage investigations by 69 bore s aggregating to 1100m length. Geotechnical problems nating from the presence of these seams in the foundation ntire dam needed careful assessment and evaluation of the ıdation rocks for providing adequate remedial measures.

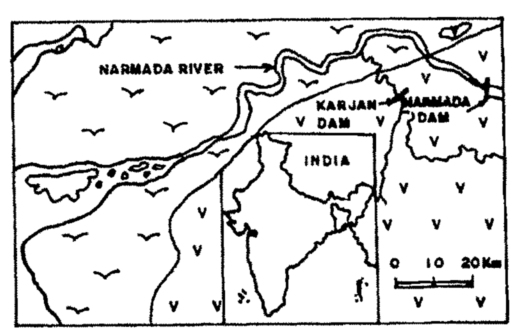
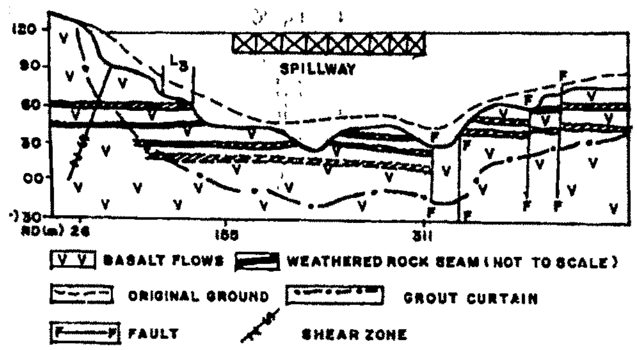


Fig.1 Location plan of Karjan Dam

ƆLOGY OF THE DAM SITE

; area is occupied by "Aa" and "Pahoehoe" type of the :can basalt flows of Cretaceous-Eocene age. The "Aa" flows characterised by fine grained or porphyritic dense basalt towards the base becoming amygdular or tuffaceous at the top. These flows are exposed on the abutments. In the river section, "Pahoehoe" type basalt characterised by wrinkled (ropy) and vesicular top, and pipe amygdules at the base is exposed. Persistent sets of joints, shears and faults in the area trend in N-S to NNE-SSW and also in NW-SE directions (Prakash, 1990).

characteristic feature in this area is the presence of ;athered rock seams at 4 to 10m intervals, developed along :erfaces of flows and sub-horizontal joints (Fig. 2).



'ig. 2 Longitudinal geological section of part of the dam howing disposition of weathered rock seams.

IATURE OF WEATHERED ROCK SEAMS

'he weathered rock seams consist of a zone of highly to ompletely weathered basalt varying in thickness from 1cm o 2m, developed along flow contacts and also along ub-horizontal open joints due to percolation of seepage water. 'he presence of slickensided surface and gougey material, nd displacement of the vertical shear zones along seams learly indicated shearing and lateral movement resulting in he formation of potential weak planes for sliding.

In-situ shear tests conducted on the seams indicated the value of angle of internal friction "ϕ"= 22° to 26° and the value of cohesion "C" = 0.

GEOTECHNICAL PROBLEMS DUE TO WEATHERED ROCK SEAMS AND THEIR REMEDIAL MEASURES

The weathered rock seams posed the problem of settlement, sliding and seepage. Treatment of these weak features depended on the thickness, properties of seam material, strike and dip of the seams and their precise disposition below the foundations.

Settlement

Weathered seams in general are of pinching and swelling nature, and wavy disposition posing no serious problem of settlement/differential settlement of dam blocks except in the foundation of left non-overflow block-L3 where minimum thickness of the seam was 0.5m and maximum 2m and rock cover was also less than 5m (Fig. 2). Part foundation of the block near block-L3/L4 joint was free from the seam (Mehta and Prakash, 1990). On engineering analyses and judgement except consolidation grouting no other treatment was considered necessary and this 60m high block has not shown any distress even after ten years of completion of the dam.

Sliding

Weathered rock seams occurring in the foundations of dam blocks have low shear parameters ("C" = 0, "ϕ" = 22° to 26°). These seams were considered as potential planes for sliding (Prakash, 1990). Based on the stability analyses concrete shear keys were provided in the foundation of all spillway blocks and in the foundations of right non-overflow blocks R1-A, R1-B and R2 besides other following remedial measures considered/provided to achieve required shear friction factor :

Curvature in the alignment of the dam. A mild upstream curvature in the alignment of the dam was initially considered for mobilising greater resistance against sliding. On the basis of geological analyses abutment rocks were not found suitable for arch action as they are dissected by steeply dipping shears, joints and faults aligned almost parallel to the probable direction of thrust in case of curved axis. These abutment rocks are also traversed by sub-horizontal weathered rock seams (Fig. 3).

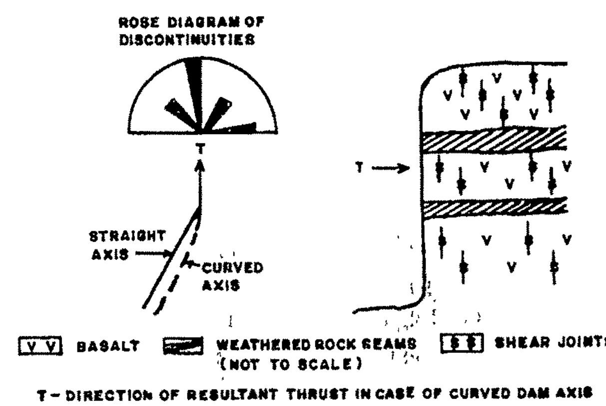


Fig. 3 Critical geological abutment conditions.

Change in the design of dam blocks. The spillway section was provided with a flatter upstream batter of 1:2 below El.75m instead of the original 1:15 below El.85.70m to increase the stability against sliding of blocks by taking advantage of additional weight of concrete and water on the upstream face.

Provision of concrete shear keys. Remedial measures like flattening of the upstream batter, roughening of the foundation base for greater friction, combining two or more blocks ogether in the stability analysis did improve the factors of afety against sliding but they were not adequate to yield the equired minimum values of the sliding factor or the shear riction factor (Parmar and Vyas, 1983). Therefore, open or mderground concrete shear keys (plugs) were provided in all he spillway blocks and also in three right non-overflow blocks lepending on the various stages of construction to achieve equired sliding factor (F1 = 1.5) and shear friction factor F2 = 3.0).

Treatment to shallow seams overlain by 3 to 5m good basalt ock cover was provided by excavating trenches and back illing them with concrete. For the deeper seams and where the blocks were already partly constructed, drifts were excavated from the approaches through shafts located in the apron area and back filled with concrete without affecting normal concreting work (Fig. 4).

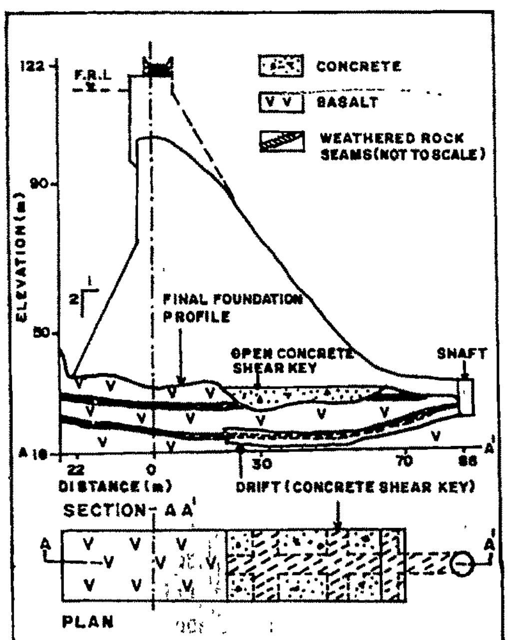


Fig. 4 Typical section and plan of spillway block showing foundation treatment of weathered rock seams.

Selection of stilling basin type energy dissipator. Basalt rocks in the downstream of spillway blocks are underlain by weathered rock seams and are blocky in nature, dissected by joints, shears and faults. Based on the geological evaluation of the rock mass and model studies of various alternative choices, a sloping apron-cum-stilling basin type of the energy dissipator was provided to check retrogression along weak geological features and to increase the passive resistance in the downstream by protecting downstream rock from the scouring. Deepest anticipated scour level in case of roller bucket, ski-jump and stilling basin was at El.12m, 14m and 33m, respectively. So far, no problem of scouring has been observed in the downstream area.

SEEPAGE

Conspicuous seepage was observed through weathered rock seams during the excavation of shafts and drifts in the foundations of spillway blocks (Mehta and Prakash, 1990). Nearly all the drill holes during pre-construction stage investigations recorded high permeability (upto 75 Lugeons). To reduce the permeability of rock foundation initial curtain grouting was done in four stages with 5, 10, 15 and 20 kg/cm^2 pressures, gradually increasing with depth when the reservoir level was at minimum. Depth of curtain grouting in spillway section varies from 42 to 60m. It was observed in five spillway blocks that post grouting seepage was more than 100 litres/minute. It clearly indicated ineffectiveness of initial curtain grouting. Therefore, to reduce the seepage additional curtain grouting was done with uniform high pressure of 20 kg/cm^2, after filling of the reservoir upto El.78m, to seal remaining gaps/permeable windows in the grout curtain. Seepage has reduced in general by about 70 to 90% after providing additional curtain grouting (Fig. 5).

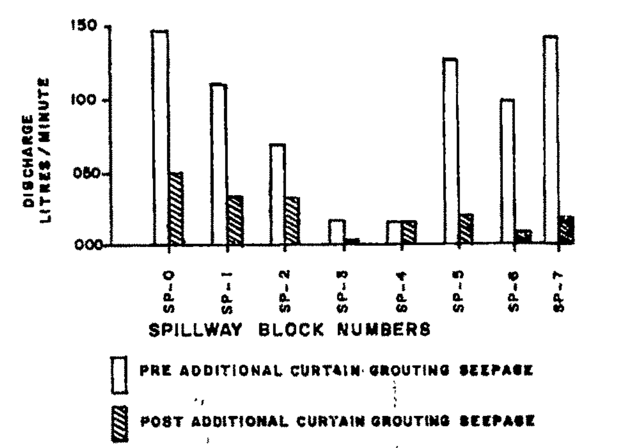


Fig. 5 Pre- and post-additional curtain grouting seepage observed through foundation drainage holes in spillway blocks.

CONCLUSION

Weathered rock seams occurring in the foundation of Karjan dam have posed mainly problems of sliding and seepage. Experience has shown that even untreated thick (2m), moderate to highly weathered rock seam overlain by shallow jointed basalt rock cover (3 to 5m) has not caused any foundation settlement problem probably due to its confined condition and slow loading/construction of the dam.

Analyses of weathered rock seams have shown that they are potential planes for sliding. Concrete shear keys were provided to act against sliding wherever the required shear friction factor was not available, especially in the spillway blocks. Flattening of the upstream batter of spillway blocks has also been done to increase the sliding stability. Stilling basin type of energy dissipator arrangement has been provided to increase the passive resistance against sliding and also to prevent retrogression along weak features.

Conspicuous seepage was observed through weathered rock seams during excavation of the foundation and even after initial curtain grouting as such type of features are pathways for ground water movement. Therefore, additional curtain grouting with uniform high pressure has been done to effectively grout the seams and to seal remaining permeable windows in the grout curtain. Now, seepage has reduced appreciably.

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EXPERIENCE IN SEISMOTECTONIC INVESTIGATIONS FOR THE EVALUATION OF DESIGN EARTHQUAKE FOR MAJOR ENGINEERING STRUCTURES, W. INDIA

EXPERIENCE EN INVESTIGATIONS SEISMOTECTONIQUES AFIN D'EVALUER LA PROBABILITE DE SEISMES POUR LES CONSTRUCTIONS DES STRUCTURES MAJEURES, INDE

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ABSTRACT

Experience in seismotectonic investigations for the evaluation of design earthquake for major engineering structures has indicated that due to lack of detailed and sufficient geologic, tectonic and seismological data, certain conservative assumptions are made for determining the seismic coefficient for the structure. In view of the safety and economy of the project, realistic value of the seismic coefficient has to be ascertained well in advance by a multidesciplinary approach.

A few examples from the projects located in north-western Himalaya and Peninsular India (Narmada-Tapi Rift Zone) have been cited to stress upon the need for detailed seismotectonic investigations.

ABSTRAIT

L'expérience en investigations séismotectoniques afin d'évaluer les probabilités de séismes quand il s'agit de faire le plan d'une structure majeure, a révélé que l'insuffisance en donnees géologiques, tectoniques et séismologiques conduit à un manque de précision dans la détermination du coéfficient séismique de la structure en question. Pour que le projet soit économique et réponde aux normes de sécurité, la valeur réelle du coéfficient séismique doit être calculée bien avant de commencer la construction, et cela par une démarche multidisciplinaire.

Nous citons quelques exemple de projets situés dons l' Inde péninsulaire (Zone de faille de la Narmada-Tapi) et dans l' Himalaya afin de souligner le besoin d'investigations séismotectoniques détaillées.

Introduction

During the last three decades there has been many fold increase in the construction of medium and major projects both in the Himalaya and in the Peninsular India. One of the most important inputs during the planning and designing of the project is the knowledge about the geology and seismotectonic history of the area for adopting a suitable design earthquake.

On the basis of the past earthequake history the country has been divided into five seismic zones and this forms a useful guide in adopting seismic coefficients for civil engineering structures (IS 1893 ; 1975). With the necessity of safe and economic planning of the large dams and nuclear Power Plants, precise geoseismological studies of the area are essential in order to incorporate a suitable seismic factor in the design of the structures.

In the last decade efforts have been made to systematise geoseismological studies in the country as indicated by the case histories of a few selected projects located in the Himalaya and Peninsular India (Fig. 1).

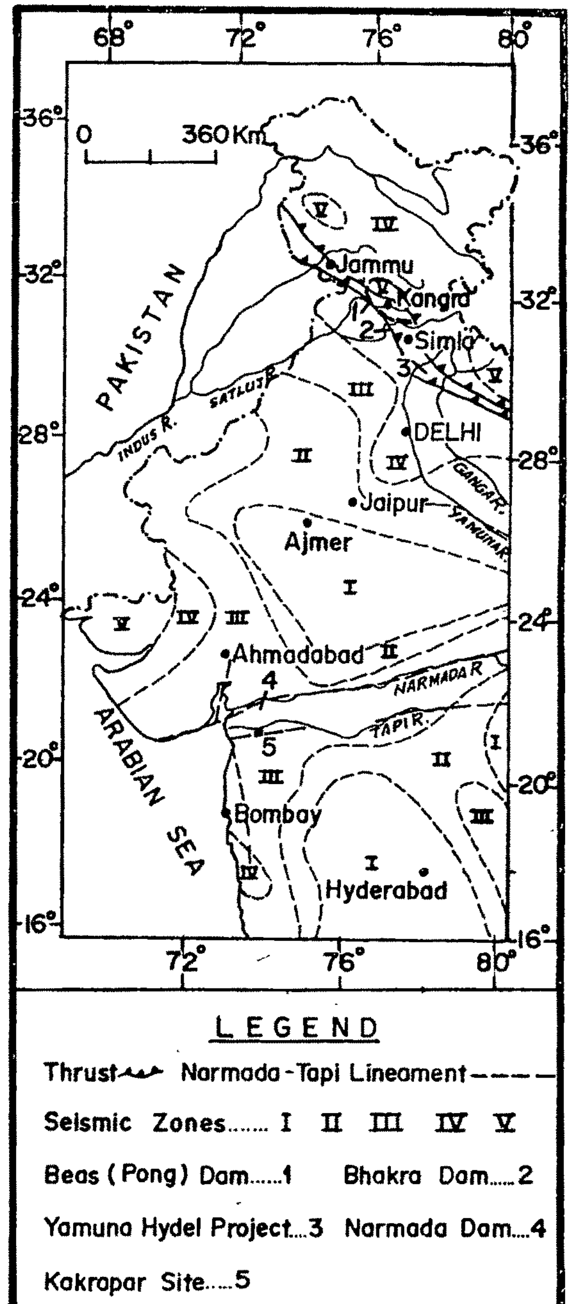

Fig.1: Plan showing location of projects and seismic zones.

Projects in the Himalaya

A number of dams have been constructed in the terminal gorge of the outer Himalaya. This region is folded, faulted and thrusted during the Tertiary era and there are evidences that the earth movements are continuing till the Recent. The continental boundaries of the Indian plate are defined by the Kirthar and Sulaiman ranges in the west and north-west, the Himalayan ranges in the north and north-east and Burmese arc in the east (Pal 1972). Himalayan belt is a compression zone with a record of high seismic activity (Fig.2) A number of earthquakes have been recorded and the expected maximum magnitude of the earthquake in the region is as high as 8 on Richter Scale, the maximum intensity is X or more on M.M. Scale and the peak ground acceleration around 50 percent of gravity (Kaila and Rao 1979).

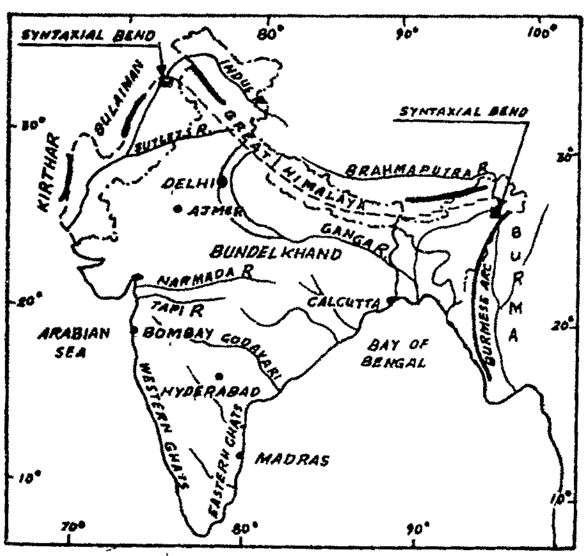

Fig.2: Index map of Indian sub-continent.

Yamuna Hydel Project

A 55 m high concrete dam across the river Tons and 6.2 km long 7 m diameter tunnel with an underground power house has been constructed under phase I of the Project. Phase II of the Project comprises 5.9 km long tunnel with a surface power house. Slates, quartzites and limestones of Jaunsar Group (Silurian - Devonian age) are folded into syncline and thrust over the Dagshai-Subathu rocks of Eocene to Lower Miocene age (Krol thrust) which in turn are thrust over the Nahan rocks of middle Miocene age (Nahan thrust). Investigations indicated that there have been subsequent movements along the thrusts when the Tertiary rocks were thrust over the sub-Recent deposits. Carbon dating of the material associated with the sub-Recent faults have indicated that these movements might have occurred some 36 000 - 38 000 years ago (Jalote and Jalote 1981).

The second tunnel for the Yamuna Hydel Project would cross the Krol and the Nahan thrusts, active faults and the highly crushed and brecciated intrathrust zone with high mountain pressures. The project area lies in Zone IV of the seismic zoning map of India (IS 1893 ; 1975) and within issoseismal VII to VIII on Rossi isoel scale of the Kangra earthquake of 1905 located about 200 km north-west of the project area.

Based on this data, a seismic coefficient of 0.1g has been provided in the design of the structures (Jalote et al 1975). The second tunnel passing through the squeezing ground and the active faults has been trifurcated with reduction in the diameter of the tunnels, provision of flexible lining and steel supports.

Bhakra dam project

The 225.6 m high straight gravity concrete dam is located in narrow terminal gorge of the river Sutlej to store 1973.56 million cubic metres of water. The Project was commissioned in the year 1963. The foundation rocks comprise steeply dipping Tertiary sandstones and shales which have been intersected by transverse and bedding faults and shear zones. Two regional thrusts lie in the reservoir area. During the Kangra earthquake of 1905, which originated about 80 km north-west of the Bhakra dam site, the area fell within isoseismal VIII on Rossi Forrel scale. The site lies within zone IV of the seismic zoning map of the country and a seismic factor of 0.15g has been taken in the design of the structure (Palta, 1979).

Beas (Pong) dam project

A 132.6 m high earth dam across the river Beas is located in the Himalayan foothills. The bedrock units at the dam site, folded into minor anticlnes, and synclines consist of sandrock and clay shale of the Pinjor formation of the Upper Siwaliks (Pliocene). At places there are evidences that the rocks of the Pinjor formation have been thrust over the Boulder conglomerate of Lower Pleistocene age along the Satlitta thrust. The thrust is located 2.7 km downstream of the dam axis and with an upstream dip of 30° lies 1.5 km below the dam foundation. Three periods of subsequent movements have been noted along this thrust during the Middle Pleistocene, Late Paleistocene and Recent times (Jalote and Tikku 1975) when the Upper Siwaliks were thrust over the Recent to sub-Recent deposits at places.

The area has experienced several earthquakes of magnitude greater than 5 on Richter scale. The Kangra earthquake of 1905 located about 60 km north east of the dam site was the severest earthquake of magnitude 8 on the Richter scale. The project area fell within isoseismal VII and VIII of the Kangra earthquake. The ground displacements were not recorded along the trace of the thrust. Periodic geodetic survey and high precision levelling is being carried out to know the possibility of present activity along the thrust. Based on the blast test results, 0.12g was adopted as a seismic co-efficient in the design of the earth dam and 0.15g to 0.20g for concrete structures (Jalote and Tikku op. cit).

Projects in Peninsular India

The Peninsular shield is comparatively a stable region and no mountain building activity has been noticed since the Pre-cambrian time. Nevertheless the movement of the Indian Plate towards north has resulted in the formation of deep seated lineaments. The Narmada-Son lineament is the most significant fault which has divided the Indian Plate into two main tectonic blocks. These have been further sub-divided into rifts and grabens with central massif (Iyengar, 1977). Investigations indicate that the margins of the Peninsular India have shown sub-Recent movements (Kailasam 1979). The studies by Kaila and Rao (1979) show that the West Coast is a zone of moderately high seismicity, the East Coast of slightly high seismicity and that Bilaspur-Hyderabad is a zone of low seismic intensity The Bundelkhand-Ajmer zone has not shown any seismic activity. Koyna earthquake of 6.5 (1979) and a number of other earthquakes have been recorded in the margin of shield area though these are generally of moderate magnitude and frequency. With this background it has now been realised that detailed geoseismological investigations should also be carried out for the projects located in Peninsular India.

Narmada Project

A 138 m high concrete dam is under construction across the Narmada river near Navagam. This is one of the well investigated projects in the country due to its magnitude and close proximity to the Narmada-Son lineament which is considered to be active. Deccan basalts of Cretaceous-Eocene age overlying the infratrappean sedimentaries form the foundations of the dam. A fault along the river channel intersects the dam axis near the right bank. A number of other faults have also been mapped in the area.

The dam site lies in the Narmada-Tapi rift zone trending in E-W to ENE-WSW direction. Neotectonic movements in the area have been recorded recently (Srinivasan et al 1981). Narmada dam has been considered to lie in a "mobile" belt of about 20 km width bounded by ENE-WSW trending fault towards north and Piplod fault towards south (Project Report 1981). The maximum magnitude of the earthquake felt in this zone is 6.5 (Narmada earthquake 1846). The Project area falls in the zone III of the seismic zoning map (IS 1893 - 1975). Micro-earthquake studies in the area show evidences of sub-zero and very low magnitude micro-earthquake activity with very shallow depth of focii (upto 5 km) and epicentres randomly distributed around the dam site.

The design horizontal seismic

coefficient as worked out by applying various techniques varied between 0.08g and 0.11g and an average value of 0.1g was recommended. Statistical studies considering the earthquake data of the last 300 years have indicated that during the life time of dam, taken as 100 years, the earthquake which may occur would have maximum magnitude of 5.8 (CWPRS 1979). Recent investigations by the Roorkee School of Research and Training in Earthquake Engineering, Roorkee have recommended an earthquake of magnitude 6.5 assumed to be associated with the Piplod fault located at a distance of about 12 km from the site and depth of focus 18 km (Project Report 1982). However, it has not been possible to establish active status of the Piplod fault? Based on the Koyna earthquake (1967) record, the deterministic approach gives a peak ground accelaration of 0.16g (Project Report 1981). However, it may be mentioned that the Koyna earthquake lies in a different seismotectonic province and it is more than 300 km away from the dam site.

Kakrapar Nuclear Power Plant

An attempt was made to systematically work out the design basis earthquake for the proposed nuclear power plant near Kakrapar in Gujarat An area within a radius of 300 km around the site was scanned on the landsat imageries on 1:1000,000 scale and available large scale aerial photographs to study the regional geological and tectonic features of the area (Mehta 1981). The available data on the past earthquakes was collected to build up a seismotectonic frame-work of the area. On the basis of the studies the afea can be divided into six distinct seismotectonic provinces (Fig.3). The Kakrapar site lies at the triple junction of the Narmada-Tapi, West Coast and Godavari seismotectonic provinces. But the more dominant lineaments in the area are in ENE-WSW direction parallel to Narmada-Tapi lineament. The severest historical earthquake of magnitude 6.5 on Richter scale is located in this province at a distance of about 160 km from the site.

In order to determine peak ground acceleration at the site, with the seismotectonic approach the following factors were borne in mind (IAEA 1979).

1. Where the earthquake of greatest magnitude or intensity has been correlated with a fault or thrust in the same seismo-tectonic province in which the site is located, it is assumed that the epicentre lies on the lineament closest to the site.
2. Where the greatest magnitude earthquake lies in the same seismotectonic province as the site but cannot be correlated with a tectonic structure, it is assumed to lie at the site for the purpose of seismic computations.

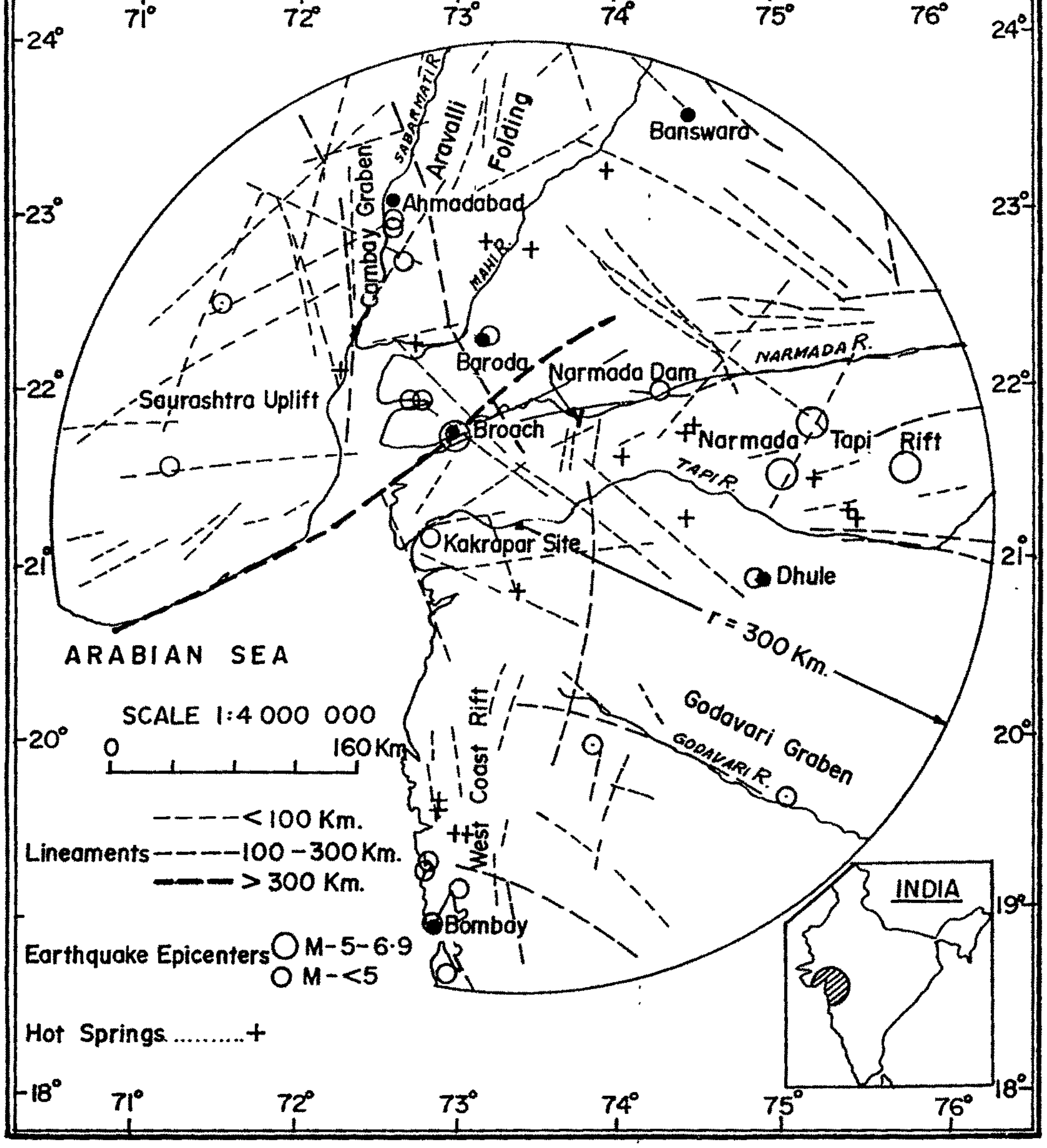

Fig.3: Seismo-tectonic provinces in radius of 300 km around Kakrapar site (lineaments interpreted from landsat imageries on 1:1000,000 scale - only major lineaments have been shown).

3. Where the epicentre of the greatest magnitude earthquake cannot be associated with any of the tectonic structures and they do not lie in the same seismotectonic province, the accelaration at the site is determined assuming that the epicentre of the earthquake is at closest point to the site on the boundary of tectonic province.

There are a number of faults sympathetic to the Narmada Tapi and West Coast lineaments but in the absence of sufficient geological and microseismic data, it was not possible to precisely indicate the fault which has been responsible for the highest magnitude earthquake in the area. There are evidences of neotectonic activity in the area to show that the Narmada and Tapi lineaments, bounding the Narmada-Tapi seismotectonic province are active and a few earthquakes might have been associated with crustal adjustment along the Tapi lineament. Thus for computation of the peak ground acceleration, an earthquake of magnitude 6.5 with depth of focus assumed as 30 km and associated with the Tapi fault located at a distance of 30 km from the site has been suggested.

Conclusions

In this paper an attempt has been made to briefly review the seismicity of north-western Himalaya and the Peninsular shield areas with special reference to the design seismic coefficient adopted for the civil engineering structures, based on the geological and seismological studies of these regions.

Himalayan region is a zone of highest seismic activity in India. several earthquakes have been recorded all along the Himalayan belt extending from Kashmir in the north-west to Assam in the north-estern part of India. The maximum earthquake magnitude recorded is as 8 on Richter scale. Recent to sub-Recent movements along the major thrusts and faults have also been recorded. Thus the projects located in the Himalaya have greater risk requiring comprehensive geoseimological studies. However, the selection of design seismic coefficient for projects located in the Himalaya are mostly guided by the past seismic history of the area as revealed by a few case histories of the projects cited in the paper.

For a long time the Indian Peninsular shield area was considered to be tectonically and seismically stable. But after the Broach (Gujarat) earthquake of 1970, Koyna (Maharashtra) earthquake of 1967 and the Kothagudem (Andhra Pradesh) earthquake of 1969, it has been realised that the marginal areas of the Indian shield are tectonically active like other shields of the world. There are also evidences of Recent to sub-Recent activity in the Narmada-Tapi, Godavari and Gondwana basins of negative gravity anomaly with central aseismic zone. The magnitude and frequency of the earthquakes in these areas are lower than the Himalaya. At the Narmada and the Kakrapar project sites located in the Narmada-Tapi rift zone, detailed geoseismological studies have been carried out to determine the design earthquake. But still there is scope for more field studies to understand the geological and tectonic history of the area. Further, geological data is required to be collected to delineate precisely active faults, their disposition, length and order of displacement. It is also important to determine the relationship of the earthquake with faults with greater certainty. In the absence of this information the design earthquakes may have to be based on assumptions.

There are certain constraints in carrying out systematic geoseismological studies particulary in the inaccessible and difficult Himalayan terrain. According to the modern concepts an area in a radius of 300 km around the site should be studied in detail for seismotectonic evaluation. This is a large area and unless studies are initiated well in advance by a multidesciplinary team of geologists, seismologists and geophysysicists, the exact picture may not emerge during the designing stage of the project. In recent years efforts have been made in this direction but still more studies are required to be carried out for each major construction site in the light of the state-of-art existing today.

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22 km from each other on the Deccan Basalt in the Narmada valley (Fig.1). The Narmada dam which is in construction stage is a 162 m high and 1270 m long concrete dam. The Karjan dam which has already been completed, is a 100 m high and 903 m long mesonary concrete dam on Narmada's tributary Karjan. In tnis paper an attempt has been made to collate and evaluate tne gaomechanical data of the foundations of these two major dams.

GEOLOGY OF THE AREA

The area at and around the Narmada and Karjan dams is occupied by the Deccan Basalt flows of Cretaceous-Eocene age which unconformably overlie the sedimentaiy rocks of Bagh Beds. The stiatigraphic sequence in this area is as below.

Age	Formation	Description
Upper Cretaceous to Eocene.	Deccan (Basalt) Trap.	Basalt flows with dykes and sills of delerite & basalt.
Upper Cretaceous	Bagh Beds	Sandstone, Shale and limestone.

'Pahoehoe' and 'Aa' type of flows are exposed in the Narmada valley. The 'Pahoehoe' flows are having relatively thin units of about 3 to 5 m thickness. Three sub-units in the following order can be recognised from bottom to top.

(a) The basal part of the unit contains vertical pipe vesicles with secondary minerals.

(b) Middle part consists of dense hard rock.

(c) Top section has spherical vesicles filled with secondary minerals. Ropy structures are well developed at places. Top surface is generally weathered.

A typical 'Aa' flow of 'Hawaii' type is tripartite with a basal clinkery zone, thick and massive middle part exhibiting columnar joints, and upper clinkery fragmented zone. In this area the basal zone is almost absent and

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GEOMECHANICAL PROPERTIES OF THE FOUNDATION ROCKS AT SARDAR SAROVAR (NARMADA) AND KARJAN DAMS, GUJARAT.

By

Indra Prakash*

ABSTRACT

The Narmada and Karjan dam sites are occupied by Deccan Traps. The older sedimentaries have been brought in juxtaposition with the basalt by a series of faults in the vicinity of the Narmada dam site. The basalt, its variants and litho-units of the sedimentaries have different geomechanical response characteristics. The geological and geomechanical properties of the individual lithounits and rock mass essential for designing the structure and providing the remedial measures at the Narmada and Karjan Dams have been evaluated

INTRODUCTION

The Deccan Basalt comprising multiple sub-horizontal flows of a few to 70 m thickness occupy a vast area of western part of India. Proper understanding of the properties of the rock material and of the rock mass are considered necessary for evaluation of an engineering project. The possible effects of loads on foundations depend on the physical properties of the rocks and the associated weak zones and structural features. In designing a dam, it is required to quantitatively evaluate the geological conditions and the physical and mechanical properties of rocks forming the foundation which affect the stability of the dam. These properties of the foundation rocks can be obtained by various laboratory and field tests.

Extensive geological and rock mechanics studies have been carried out at the Sardar Sarovar (Narmada) dam and Karjan dam sites located about

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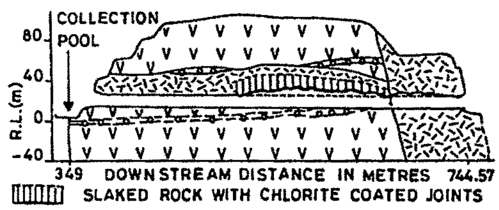

Figure 9. Geological section of exit tunnel-1

Cracks in the bell mouth sections of the oulet portals of exit tunnels have been observed in the structural concrete while the excavation of the tail race channel was in progress (June, 1997). These are of en echelon pattern and not following any geological features. Designing of additional supports to strengthen bell mouth sections of the tunnels is in progress.

9 CONCLUSIONS

Geotechnical problems of sliding, settlement and seepage related to geological features in the dam foundations were anticipated during construction stage investigations. Experience has shown that problem like scouring of foundations during successive floods could have been avoided if proper construction sequence had been followed at site.

Critical examination and evaluation of the rock mass during construction stage geotechnical investigations of the underground structures have helped in timely reviewing supports system from stability consideration. Therefore, construction stage geotechnical investigations are essential for taking remedial measures in time for the safety and stability of structures.

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8.6 *Reasons for the development of cracks in the machine hall and remedial measures*

Cracks developed in the pressure shafts and bus galleries are aligned parallel to the longer axis of the machine hall. These cracks are not following geological discontinuities (Fig.8). Sub-horizontal to low dipping cracks developed in the downstream wall in en echelon pattern and sub-horizontal cracks in the upstream wall are parallel to the excavated profile of the existing ramp. A few cracks observed near the major shear zones 'A' and 'B' are due to modification of stresses around these weak features as also observed in the 3-D Finite Element Analysis (FEM) carried out by National Institute of Rock Mechanics. Symmetry of pattern of cracks parallel to the longer axis of the cavern in the pressure shafts and bus galleries suggest that these cracks are developed due to tensile stresses acting on inadequately supported rock mass.

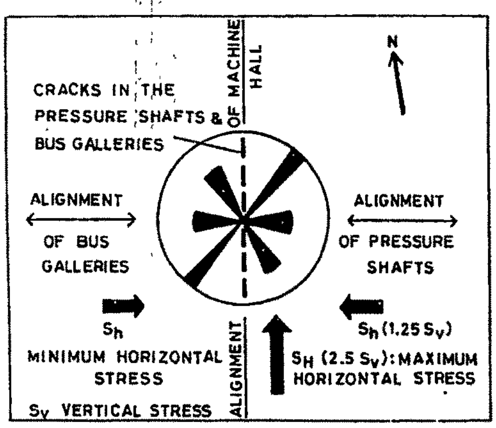

Figure 8. Rose diagram of discontinuities.

Additional supports have been installed in the machine hall in the area affected by cracks. The remedial support in the upstream wall consisted of 10.5 to 32m long 80 tonnes capacity cables, tensioned to 50 tonnes and then fully grouted. In addition 12m long, 32mm dia, rock bolts, tensioned to 20 tonnes, were installed at various locations. In the downstream wall, a large number of 12m long 32mm dia, rock bolts, tensioned to 20 tonnes before grouting, were installed in addition, a number of 25m long 50 tonne capacity cables were also installed. These cables were tensioned to 5 tonnes before grouting.

Continuity of glass plates breaking in the machine hall and slight deformation observed by the instruments suggests that the rock mass has not yet stabilised, despite installation of longer anchors/ cables/ tendons in the upstream and downstream walls in the area affected by cracks. Designing of additional supports to strengthen both the walls is in progress

8.7 *Access Tunnel*

The D-shaped 8.5m wide and 9m high access tunnel passes through basalt and agglomerate for a length of 230 m and dolerite dyke / sill in the remaining length. The initial support system comprised 25mm dia, 6m long pattern rock bolts at 1.75m c/c with two layers of 38 mm thick shotcrete with wire mesh in between Problems due to shear zones, fault and water seepage were encountered during the tunneling operations Flat roof caused by overbreaks along sub-horizontal shears in the initial 230m length necessitated installation of additional rock bolts.Therefore, spacing of the rock bolts in the crown has been reduced from 1 75m c/c to 0 75m c/c in this reach. Additional drainage has been provided in the water seepage area to guide and drain out water into the drain well. In the tunnel section traversed by Akkalbar fault (8 to 10m wide) steel ribs were installed

Part of the tunnel section had to be excavated below 57m high Rock fill dam with a water storage of 30m depth. Rock cover over the tunnel in the 50m length varied from 10m to 17m. Excavation in this part was done by smooth blasting techniques (Peak particle velocity was limited to 6.25mm/sec). Steel ribs were also installed in this low rock cover reach.

8.8 *Draft Tube Tunnels*

Major part of the tunnels (10m dia.) are passing through dolerite sill. Roof falls have occurred in the draft tube tunnel- 2 & 3 in the reaches occupied by sub-horizontal shears and slaked rock zones. Overbreaks of the order of 4.5m in height have been observed in these reaches even after the installation of rock bolt supports. Therefore, rib supports have been installed in all the tunnels after the collapses instead of initial designed rock bolt supports.

8 9 *Exit tunnels*

Horse shoe shaped exit tunnels (E T.) of 12.5m dia, are passing through basalt, agglomerate, dolerite dyke and sill. The Akkalbar fault runs parallel and close to the alignment of the E T.-1 for about 222m length Joints sympathetic to the fault are observed in this tunnel. Intersection of three sets of chlorite coated joints in dolerite rock, form removable/detachable blocks of size varying from 1m³ to 6m³ resulting in block falls at places. Collapses in part of the tunnel sections traversed by chlorite coated joints and slaked rock zones have occured despite installation of design rock bolt supports (Fig.9) Therefore, rib supports have been installed in all the tunnels at critical locations (Prakash & Sanganeria 1993)

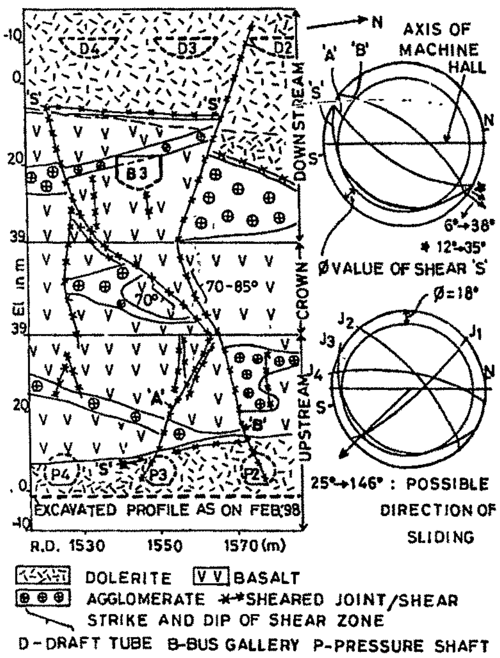

Figure 5. Geological map of part of machine hall.

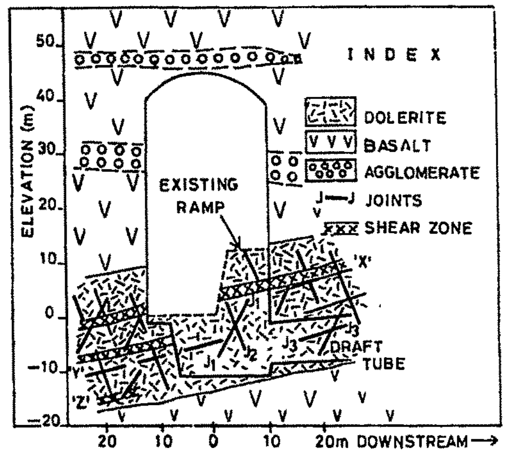

Figure 6. Geological cross section of machine hall.

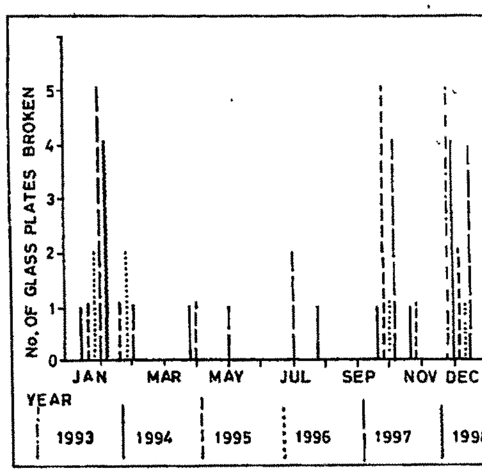

Figure 7. Glass plates breaking data of the downstream wall and bus galleries.

of their orientation and thus they are not creating any problem of wedging (Fig. 5).

Major part of the remaining excavation in the downstream is required to be carried out in the dolerite sill dissected by chlorite coated joints. Sliding wedge is formed with the intersection of joints J_1 and J_3 having 25° plunge towards S34°E (i.e towards free face). Similar minor rock wedges are formed with the intersection of joints J_1 and low dipping shears 'X', 'Y' and 'Z' (Fig.6). Therefore, it has been suggested to adequately stitch these rock wedges during progressive excavation.

Prominent shear zones and joints are striking at an angle more than 30° to the longer axis of the cavern and thus ruling out the possibility of plane failure.

8.4 *Analysis of Glass Plate Data*

Glass plates have been installed in the downstream wall and bus galleries to monitor the existing cracks. Analysis of the data for the period October, 1993 to February, 1998 show that the activity of glass plate breaking is intermittantly continuing and is of periodic nature. The activity is prominent during the period October - February of each consecutive year (Fig. 7). Reasons for enhancement of the activity can be attributed to either reduction in the strength of the rock mass after saturation or to the development of hydrostatic pressure in the walls or to both.

8.5 *Instrumentation in the machine hall*

Multi-position bore hole rod extensometers, Demac gauges and 3-D crack monitors have been installed in the pressure shafts, in the downstream wall and bus galleries by Central Soil and Material Research Station after the development of the cracks. Majority of the instruments earlier installed are not working Analysis of available instruments data upto December, 1996 revealed that at most of the locations deformation is unpredictable. Very slight (< 2 mm) deformation has been observed at few locations in the pressure shafts- 2 & 3, in the downstream wall (R.D. 1502 to 1525m) and in the bus galleries- 2 & 3.

against the designed intensity of 135 cumecs/m directly impinging on the concrete floor due to differential height of the dam blocks (at that time), development of cross flow conditions due to incomplete divide wall and development of uplift pressures due to macro-turbulance along different layers within the concrete and along concrete and rock contact This scoured pit has been backfilled with concrete. Designing of the anchors to hold different layers of concrete and concrete with foundation rocks is under progress Now, dam blocks have been raised to uniform height

8 GEOTECHNICAL PROBLEMS OF UNDER GROUND POWER HOUSE

8.1 *Machine Hall*

The machine hall (cavern), located in the sub-horizontally disposed basalt flows intruded by vertical and inclined dolerite dykes and sill, is 210m long, 23m wide and 57m high. Rock cover above the crown of the machine hall varies from 35 - 60m. Part excavation of the machine hall is completed from El (+) 45m to El. (-) 1.9m by heading and benching method leaving about 8m wide sloping ramp adjacent to downstream wall No excavation has been done in the machine hall since September, 1993. The rocks in and around the cavern are strong and jointed (average block size $1m^3$) and dissected by shears

In situ stresses around the cavern have been determined by National Geophysical Research Institute by hydro fracturing test The major in situ stress is approximately 2.5 times the vertical stress and is parallel to the longer axis of the cavern. The intermediate principal stress perpendicular to the cavern axis is approximately 1 25 times the vertical stress. The minimum in situ stress is vertical and equal to depth below the surface times the unit weight of the rock The vertical stress is approximately 1.2 MPa and the horizontal stress acting perpendicular the cavern axis is approximately 1 5 MPa. The average compressive strength of rocks surrounding the power house cavern is more than 60 MPa

During benching operations sub-horizontal to low dipping cracks in the shotcrete of the upstream and downstream walls (between El. 4 and 38m) and vertical cracks in the pressure shafts (upto 10m distance) and bus galleries (upto 17m distance) have been observed. Except a few cracks on the walls majority of the cracks are not following geological features. First crack was noticed in the upstream wall when the bench level was at El. 14m at R.D. 1569m (Prakash & Sanganeria 1993) With progressive excavation in the machine hall down to El. (-) 1.9m, a number of cracks have been observed on both the walls. Widening of the existing cracks was also observed. No new cracks or widening of the existing cracks has been observed since the stop of the excavation (September, 1993) in the machine hall. However, breaking of the glass plates installed to monitor the cracks is continuing.

8 2 *Design Supports*

Roof supports include tensioned rock bolts of 25mm dia, 6m long and 1.75m center to centre (c/c) pre-tensioned to 14 tonne 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 Central Water Commission by adopting Barton's and Bieniawski's rock mass classification.

The excavation has been done by NATM to utilise the rock itself as principal structural material Analysis of the design supports show that length of the rock bolt designed for roof is adequate while that for the walls is inadequate. The wall rock bolt length from the various empirical approaches has been estimated to be at least 10m against 6 to 7m long rock bolts installed in the machine hall (Goel & Jethwa 1992) For 58m high cavern 11m long rock bolts or 20m long cables are required (Fig 4).

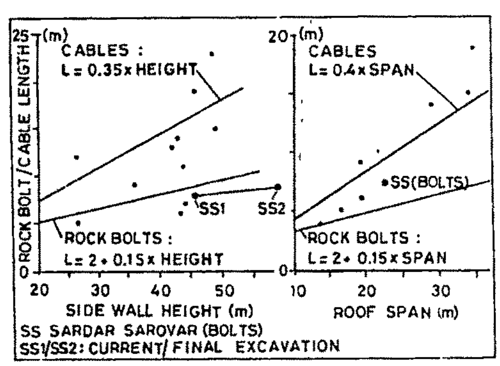

Figure 4 Plots of rock bolt and cable lengths for side wall and arch (roof) supports in various underground hydroelectric projects (Hoek 1995)

8 3 *Wedge and plane failure analysis of the discontinuities*

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) In the downstream wall these shears are diverging by virtue faults have been provided dental treatment varying in depth from 1.5 to 2.0m depending on the width of the fault zone.

6.5 *Weathered rock*

During foundation preparation, in general weathered rock has been removed from the foundation area. However, in some of the blocks (3, 15, 16 and 57) where depth of weathering was more than 5m, it was decided to retain and treat weathered rock mass. Reinforced concrete mat in single or two layers was provided depending on the nature of weathering and foundation topography to prevent differential settlement (Mehta & Prakash 1990).

6.6 *Limestone*

The limestone is occuring in the foundation of right bank spillway blocks 44 to 50 at about 40m depth. Thickness of limestone varies from 30 to 60m. It is of siliceous nature (average SiO_2 20%). On the surface, it has been brought by Mokhadi fault at about 500m upstream. During exploratory drilling poor core recovery, high permeability and heavy water losses have been observed at few locations in depth. Therefore, tracer studies were conducted to know the nature of limestone. Neither cavities nor interconnection between test holes was established by these studies This suggests that high permeability observed in few drill holes might be due to presence of local joint pockets. There is also possibility that intrusion of basalt and dolerite dykes in the foundation rocks may act as seepage barrier. However, depth of curtain grouting has been increased down to depth of entire section of limestone on right bank to seal local permeable zones.

7 PROBLEM DUE TO CONSTRUCTION STAGE FLOODS

The energy dissipation arrangement for 23 gates service spillway consists of sloping cum horizontal jump type stilling basin and for 7 gates auxiliary spillway, split-level chutes terminating into ski-jump bucket. The stilling basin and chutes are separated by about 50m high divide wall. Extensive damages have been observed in the foundation of divide wall and stilling basin during construction stage floods.

7.1 *Left divide wall block -28*

This wall is located between the lower chute and stilling basin. Major part of the foundation of cladding section of the wall remained open between the period 1988 and 1996 resulting in the progressive scouring of the rock face during successive floods. Maximum scouring of the order of 15m laterally inside has been observed in the weathered rock area during September, 1994 flood when the flood water level (El. 52m) was above the foundation level. Final scoured topography of the foundation is having trapezoidal section with narrow base and broad top. Longer anchors and reinforced columns and beams resting on good rocks have been provided besides additional drainage to stabilise the wall (Fig 3).

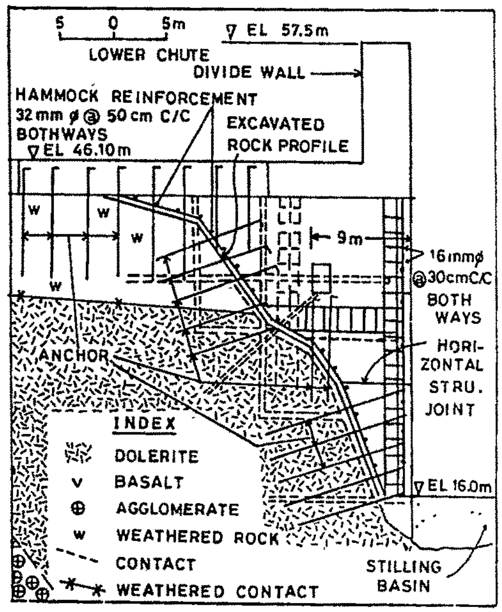

Figure 3. Typical section of left divide wall block-28 showing foundation treatment.

7.2 *Stilling basin bay -1*

The foundation of this bay was completely covered with structural concrete in June, 1994. Height of the spillway blocks in bay-1 varied between El. 69 and 87m at that time. Divide wall between bay-1 and bay-2 was also partly completed. About 2.5 million cusecs flood discharge passed over the partly constructed dam eroding about 35,000m³ of structural concrete and about 64,000m³ of underlying rocks in bay-1 in Sept, 1994. Geology was not responsible for this damage as neither selective scouring nor seepage was observed along the geological features in the scoured pit

The energy of the flood water was so high that it had sheared the anchor, uplifted and rounded concrete and rocks blocks (upto 300m³) from the stilling basin area and transported them beyond end sill of the stilling basin, covering about 300 - 400m distance. Reasons for the deep scouring can be attributed to the high concentrated discharge of 230 cumecs/m in the bay-1

5 GEOTECHNICAL INVESTIGATIONS

The geological and geotechnical investigations for the selection of Narmada dam site were initiated during the year 1948. Pre-construction stage investigations for the present site commenced in the year 1962 and construction stage investigations started in the year 1978. Detailed construction stage geotechnical investigations at site include drilling of about 30 running km cores, excavation of 90cm diameter calyx holes, shafts, adits, pits and trenches and their logging. Large scale geological mapping (on 1:100 scale) covering about 1000,000m² area for the assessment and evaluation of rock mass conditions of dam foundations and underground structures has been done Field testing of rock mass and laboratory testing of rock cores/ samples have been done (Prakash 1990) Important tests conducted at site include in situ deformation modulus tests on fault zone material and surrounding rocks, in situ shear tests for assessing shear strength of red bole, sheared contacts of sedimentaries and interfaces of rock and concrete, tracer studies for determining seepage losses through sedimentary rocks, blast tests for estimation of design seismic coefficient, hydrofracture tests to know the stresses around the underground power house. Laboratory analysis for deciding depth of concrete plug for the treatment of fault zone and 3-D stress analysis of rock mass surrounding the underground structures have also been done

6 GEOTECHNICAL PROBLEMS AND TREATMENT OF MAIN DAM

6 1 *Sub-horizontal weak layers*

A red bole layer on the left bank and sheared contacts of the sedimentaries and tuff layer on the right bank posing the foundation problem of sliding stability of the spillway blocks have been evaluated and treated.

1. Red bole layer: A red bole layer having rolling dip of the order of 5° to 15° due NE to SE i.e. towards upstream side was delineated about 10 to 30m below the foundations of left bank spillway blocks 28 to 42 between El.22 and (-) 10m. Shearing along red bole layer has been observed In situ shear tests on the red bole indicated negligible cohesion and value of angle of internal friction ('ø') as 17°. In view of the low shear parameters treatment to the red bole layer has been provided by excavating 3m wide drifts in a grid pattern across weak layer and back filling them with concrete to act against sliding of the dam blocks. Similar treatment to shear ('ø'=36°) traversing dolerite dyke in the continuity of red bole layer has also been provided in the foundation

2. Sheared contacts of sedimentaries : About 10 to 18m below the general foundation levels of spillway blocks-44 to 51, sedimentary rocks comprising of argillaceous sandstone, quartzitic sandstone, pebbly sandsone, shale and limestone underlain by basalt flows are present. Contacts of the argillaceous sandstone and quartzitic sandstone are sheared ('ø' = 11°). Treatment provided to the argillaceous sandstone layers was similar to the red bole. The drifts replacing the lower and upper drifts are lying one over the other separated by quartzitic sandstone (Prakash 1990)

3. Tuff layer: A tuff layer varying in thickness from a few centimetres to 3 metres is present at the contact of upper argillaceous sandstone and basalt. This layer has been provided treatment along with the upper argillaceous sandstone layer by removing it from the crown portion of the treatment drifts

6.2 *Low dipping shears*

During the excavation of shafts and drifts for the treatment of sedimentaries a low dipping shear ('ø'=30°) has been encountered in the overlying basalt in the foundations of spillway blocks 46 and 47 Based on the stability analysis treatment to the shear zone has been provided by constructing open concrete shear keys in these two blocks

6.3 *Main river bed fault*

A river bed fault aligned in N80°E-S80°W direction, dipping 60° towards N10°W is obliquely traversing the foundations of four spillway blocks 41 to 44 This fault has brought sedimentaries in juxtaposition with the basalt at the dam base (Fig 2 a, b, c) Width of the fault zone is about 10 to 12m. Fault is associated with 5 to 15cm thick gougey material Rock mass adjacent to fault zone is sheared and fractured

In situ test results have indicated low values of modulus of deformation for the fault zone (0 05 X 10^5 kg/cm²). High values of modulus of deformation has been obtained for the basalt (0.52 X 10^5 kg/cm²) and sandstone (0.38 X 10^5 kg/cm²). In view of the low 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 river bed blocks 41 to 44 was apprehended 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 judgement of various geotechnical consultants Depth of the concrete (reinforced) plug is 34m in the upstream and 26m in the downstream (Mehta & Prakash 1990)

6.4 *Minor faults*

Sympathetic to main fault, seven minor faults are obliquely traversing the foundation of dam blocks 4-5, 21-24, 25-27, 30-34, 45-47, 45-48, 48-55. These local

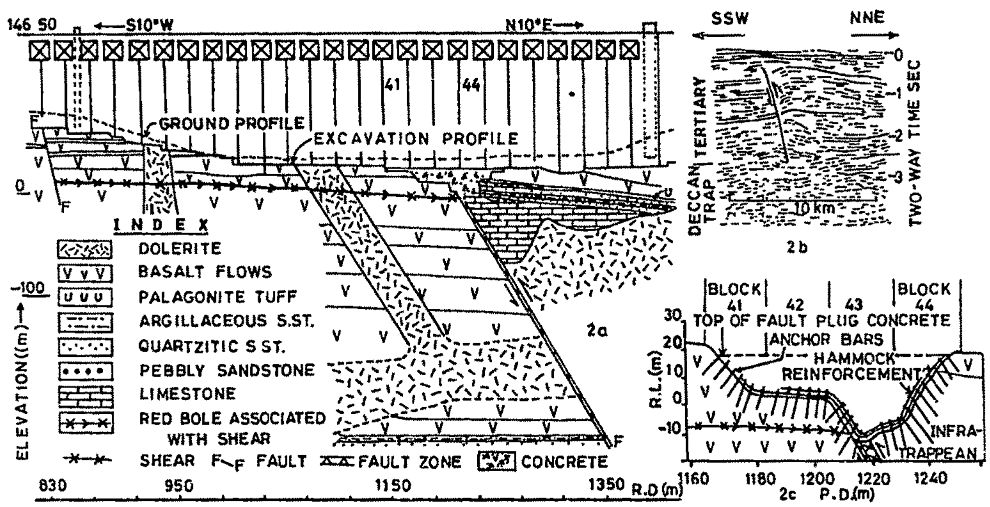

Figure 2(a) Longitudinal section of spillway blocks, 2(b) Seismic section across north Tapti anticline in onland Tertiary sequence in western part of 'SONATA' zone (RaviShanker 1993) and 2(c). Section depicting fault zone treatment.

Distinct phases of post Deccan trap tensional and compressional deformations are seen in the SONATA zone and adjoining region (RaviShanker 1993). Normal and reverse faults have been observed in the area The area is at present under compression. Seismotectonic study in the NSL zone after Jabalpur earthquake (M 6 0) of May 22, 1997 has shown that it was caused by a reverse fault mechanism in a compressive regime due to post collisional northward movement of the Indian plate (Acharyya et al. 1998). Neotectonic activities in the lower Narmada valley have already been reported (Srinivasan et al. 1981).

3 GEOLOGY

The project site is occupied by the Deccan basalt flows underlain by sedimentary sequence of Bagh beds (infra trappeans). The sedimentaries and basalt are profusely intruded mainly by dolerite dykes aligned in ENE-WSW direction. A river bed fault has brought the sedimentaries in juxtaposition with basalt at the dam base (Fig.2). This fault is en echelon type reverse fault having displacement of the order of 210m with upthrow side towards north i.e towards right bank. Contacts of the sedimentaries and some of the basalt flows are sheared. Basalt flows at the project site are sub horizontal to low dipping (upto 25°). Dykes in the foundation area are displaced along low dipping shears/ faults. Dipping of the basalt flows, occurrence of dyke swarms, displacement of dykes and emergence of the Bagh beds from underneath the Deccan traps in juxtaposition with basalt suggest post Deccan trap activities in the area.

4 SEISMICITY

The project lies in a "mobile" belt of about 20 km width, bounded by ENE-WSW trending fault towards North and Piplod fault towards South, in the SONATA zone. Micro seismic studies in the area show evidences of sub-zero and very low magnitude micro earthquakes activity with very shallow depth of foci (upto 5 km) and epicentres randomly distributed around dam site (Mehta & Prakash 1982). Evaluation of the seismic data for the period 1974 to 1996 revealed that there is no perceptible increase in the number of events before and after the partial impounding (since Feb., 1994) upto El. 80.3m However, two events of magnitude (M) greater than 4 i.e. M 4.2 (March, 1994) and 4.5 (Nov., 1996) are located in the downstream with epicentres at 18km and 87km on the southern bank, respectively. These two events might be part of the regional seismic activity and not necessarily related to the impounding of reservoir as a number of seismic events have been reported recently in the NSL zone. The Son-Narmada-Tapti rift zone is considered to be seismically active (Acharyya et al. 1998).

The project area falls in the zone III of the seismic zoning map of India (IS 1893:1975). The maximum magnitude of the earthquake felt in this zone is 6 5 Horizontal seismic coefficient adopted for the dam design is 0.125g. Monitoring of the project site by a network of eight seismological observatories is being done by project authorities.

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Geotechnical investigations of Sardar Sarovar (Narmada) Project, India

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ABSTRACT · The Narmada project is located on the Deccan basalt in the Son-Narmada-Tapti (SONATA) rift zone Identification, delineation and evaluation of the geological features affecting stability of structures have been done. Based on detailed investigations and analysis, treatment to weak geological features have been provided in dam foundations Effects of construction stage floods in the foundations of stilling basin and left divide wall have been analysed Construction of underground structures is being done by adopting New Austrian Tunneling Method (NATM) Rib supports have been introduced in parts of tunnels and longer rock bolts/ cables/ tendons have been installed in the area affected by cracks in the machine hall after observing behaviour of the rock mass during construction stage investigations

RÉSUMÉ. Le projet Narmada est situé sur le Deccan Basalt dans le rift du Son-Narmada-Tapti (SONATA) L'identification, la délinéation et l'évaluation des propriétés géologiques affectant la stabilité de la construction ont été faites. Fondé sur des recherches et des analyses détaillés le traitement des propriétés géologique faibles a été fourni lors de la fondation du barrage Les effets de l'inondation causée en cours de construction sur la fondation du bassin de retenu et le mur mitoyen gauche ont été analysés La construction des structures souterraines est en cours de réalisation le New Austrian Tunneling Method (NATM) Le support des tendeurs formeiets a ete introduit dans certaines parties des tunnels et des boulons rocheux plus longs/ câbles/ ont été installés dans les zones affectées par des crevasses dans la salle des machines après avoir observé le comportement de la masse rocheuse pendant les recherches faites en cours de construction.

1 INTRODUCTION

The Sardar Sarovar (Narmada) Project is being constructed to harness the vast irrigation and hydroelectric potential of the Narmada River at terminal gorge The Narmada project is a multi-state and multi-purpose river valley project Construction of 129m high (above the average bed level) and 1270m long concrete gravity dam to irrigate 3 4 million hectare area through a net work of canal system is in progress Construction of surface power house to generate 250 (5 X 50)MW is completed and construction of underground power house to generate 1200 (6 X 200)MW power is in progress Identification and delineation of geological features affecting the stability of surface and underground structures and evaluation of the foundations and rock mass surrounding the underground structures based on the geomechanical properties have been done

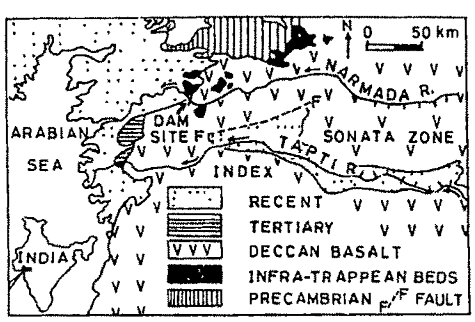

Figure 1 Location plan of the Dam site

2 GEOLOGICAL SETTING

The Narmada project is located on Deccan basalt flows in a graben bounded by faults parallel to the Narmada-Son lineament zone (NSL) aligned in ENE-WSW direction (Fig 1) This zone transects the shield area of peninsular India into northern and southern blocks It has been reactivated several times in the geological past

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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 cbserved 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 multi-purpose 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 under-ground 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).

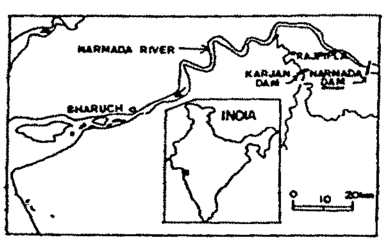

Fig. 1-Location plan of Sardar Sarover (Narmada) Dam Project.

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.

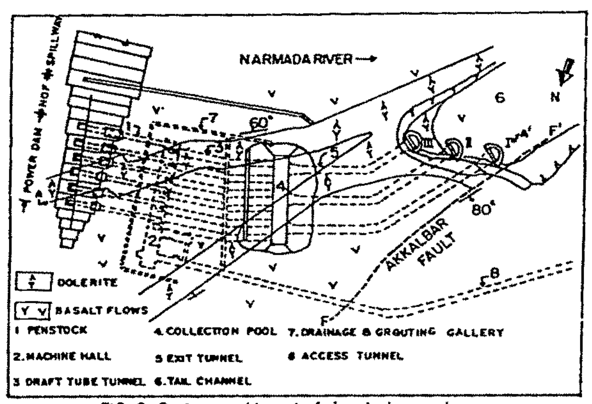

FIG 2. Geology and layout of river bed powerhouse

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 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	1.195×10^{4} MPa	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 El.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 El.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

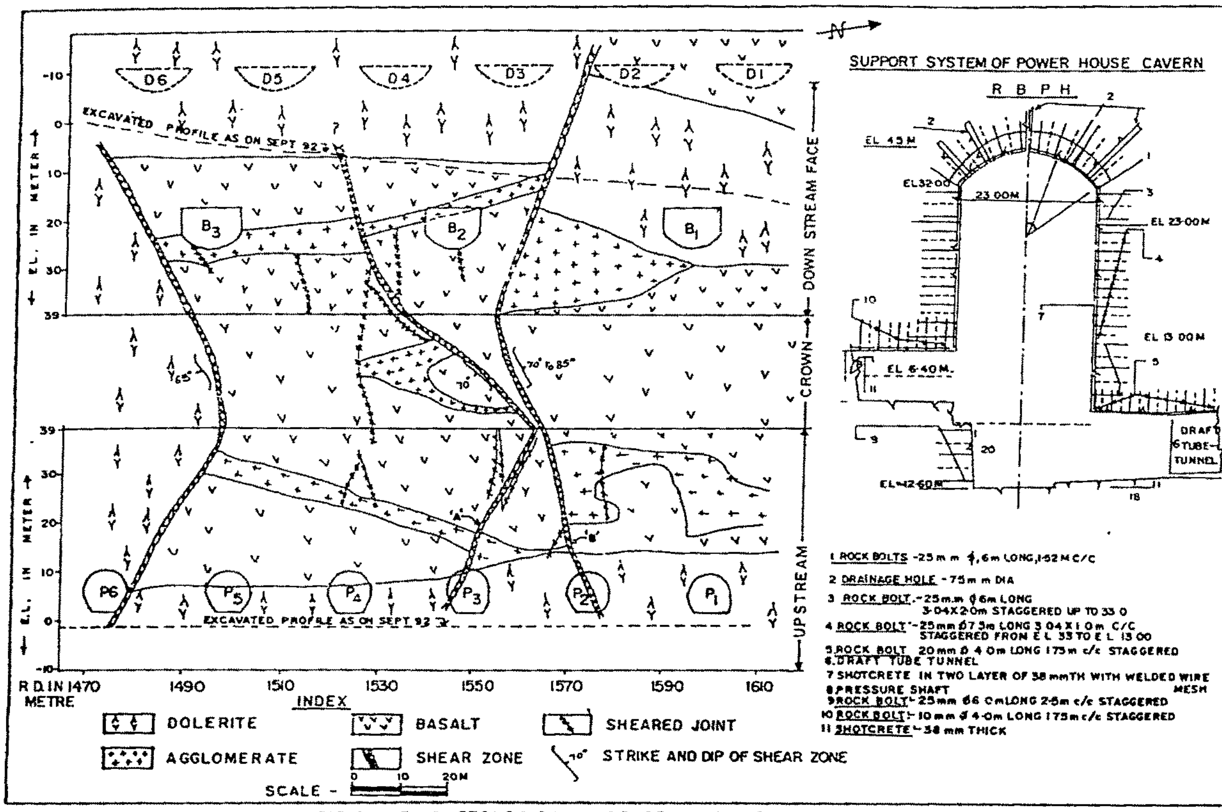

FIG 3. 3-D GEOLOGICAL LOG OF MACHINE HALL

cracks are developed along and adjacent to shear zones 'A' and 'B' above pressure shafts - 2 & 3 between El.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 El.36.5 and 37.5 m, that is just below the spring line El.39 m (Fig.4).

Deformation of the rock mass has continued since March, 1991. Reappearance and development of new cracks in the shotcrete and widening of the existing cracks during benching operation from El.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

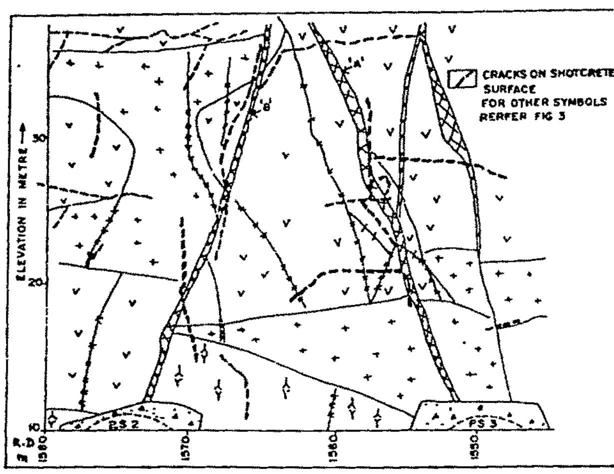

FIG.4 — PART OF UPSTREAM FACE OF THE MACHINE HALL SHOWING DISPOSITION OF CRACKS

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 El.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 El.66 m. The seepage was anticipated to increase manifold when the reservoir reaches its FRL at El.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.

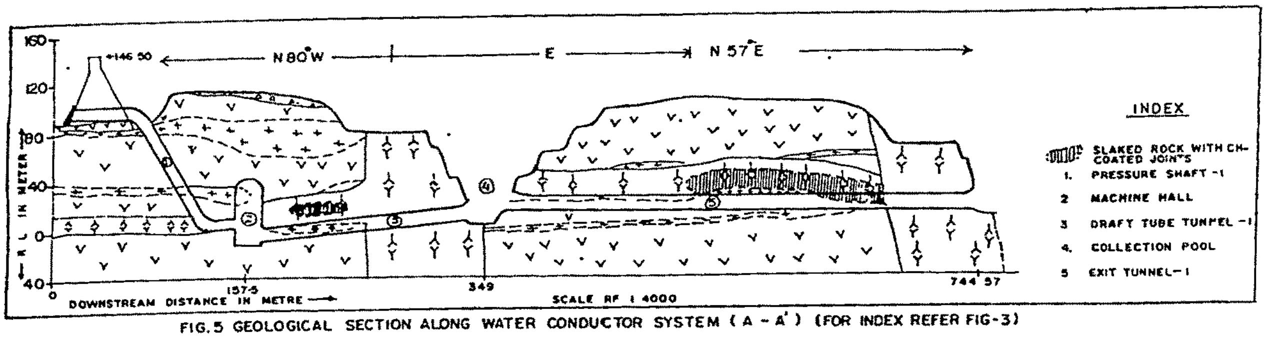

FIG.5 GEOLOGICAL SECTION ALONG WATER CONDUCTOR SYSTEM (A - A') (FOR INDEX REFER FIG-3)

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

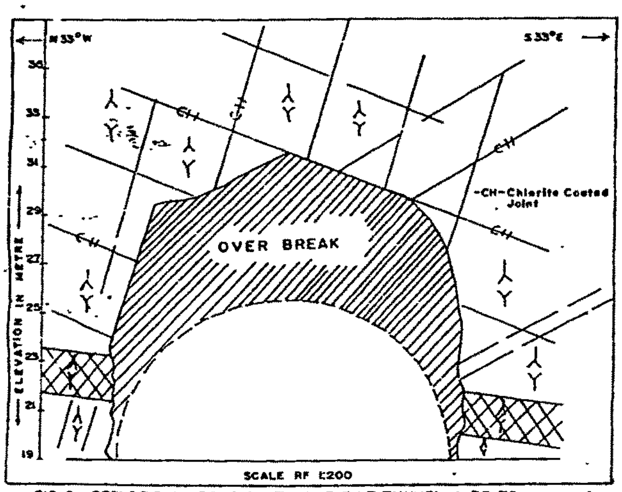

FIG.6 GEOLOGICAL CROSS SECTION OF EXIT TUNNEL-I. RD.79 m. (FOR INDEX REFFER FIG 3)

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 studies is of prime importance in such projects.

Acknowledgement:

The authors wish to record their sincere thanks to the Director General, Deputy Director General (Western Region) & Deputy Director General (Operations Gujarat), Geological Survey of India for the kind permission to publish this paper.

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Appendix-VII

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EXPERIENCE OF EXCAVATION FOR UNDERGROUND STRUCTURES THROUGH DOLERITE AT SARDAR SAROVAR (NARMADA) PROJECT, GUJARAT

By

Indra Prakash*

ABSTRACT

The underground power house is located in the basait flows intruded by dolerite dykes and sill. Major part of the tunnels are passing through slaked and chloritised dolerite posing problem of roof collapses and rock falls. Rock-bolt supports have been reviewed and rib supports have been introduced in the tunnels after observing behaviour of the dolerite rock mass during progressive excavation.

Introduction :

The Sardar Sarovar (Narmada) Project, under construction, is a multipurpose river valley project located in Gujarat. The project envisages construction of 1270m long and 128m high concrete gravity dam, 1200 MW underground power house and 250 MW surface power house. The underground power house is located at the toe of the main dam on the right bank. The construction of six draft tube tunnels is near completion. Other ancillary structures including six pressure shafts, an access tunnel and the exit tunnels are already completed. The cavern and tunnels have been opened by New Austrian Tunnelling Method (NATM). Geotechnical problems encountered during the excavation of tunnels through dolerite rocks are discussed.

Geology :

The underground power house is located in the sub-horizontally disposed Deccan basalt flows intruded by vertical and inclined dolerite dykes and

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sills (Prakash, I, 1993). These dykes are 40 to 45m thick, aligned in NNE-SSW to ENE-WSW direction The dolerite sill is about 25m thick, aligned in NE-SW direction, dipping 20° to 25° towards South East. The vertical dyke is traversing the access tunnel, draft tube tunnels 1 and 2 and all the exit tunnels. Inclined dolerite dyke is traversing the machine hall and exit tunnels (Fig. 1). Major part of the crown of the exit tunnels, draft tube tunnels and bottom of the machine hall including foundation of the Turbo-generator units are loacated in the dolerite sill dissected by chlorite coated joints, shears and slaked zones (Fig. 2).

Petrograpy of the Dolerite :

Dark coloured, medium to coarse grained rock compossed of laths of plagioclase feldsper and clinopyroxene. Secondary minerals include magnetite, chlorite, serpentine and olivine Apatite occurs as accessory mineral. Clinopyroxene poikilitically encloses plagioclase feldspar giving rise to ophitic to sub-ophitic texture, intergranular at places and holo-crystalline

Alteration of dolerite rock :

Alteration of feldspar to sericite and augite/olivine to chlorite have been noticed in some of the rock samples of the vertical dyke and sill. A few cracks observed in these rocks are of branching type infilled with chloritic material.

Laboratory test of slacked rock :

Rock lumps of slaked dolerite have been subjected to alternate wetting and drying cycles of 24 hours each in the laboratory. Chlorite flakes started separtating along joint planes after two cycles. Rock lumps started crumbling after 5 and 16 cycles. Slaked rock crumbled into flakes or granular particles after exposing to air depending upon the nature and degree of alteration

X-ray analysis of the slaked dolerite :

Saponite has been identified as a major constituent in the slaked dolerite rock. Chlorite occurs in small amount along joints (Table-1).

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Table-1

X-ray analysis of dolerite rock.

Part of the sample	Constituents	Remarks
Powder	Saponite (Smectite group)	Major
Thin black platy material	Saponite	Major
	Chlorite	Small amount
	Calcite, quartz	Minute
	Stellerite	Minute traces
Hard material	Saponite	Major
	Stellerite (Na) (Zeolite group)	Good
	Feldspar	Small amount
	Clay	Trace
Brittle soft material	Plagioclase	Major
	Chlorite	Small amount
	Amphibole	Minute traces

Physico-engineering properties of Dolerite :

Test results of the physico-engineering properties determined in the laboratory revealed that Uniaxial Compressive of strength of unaltered rock is almost double of the moderately altered rock (Shah, K. N., 1991). Porosity and water absorption percentage increases with the increase in alteration of dolerite rock. However, specific gravity remains almost same (Table-2).

Table-2

Physico-engineering properties of Dolerite

Types of the rock samples	Properties			
	% water absorption	% porsity	True specific gravity	Uniaxial compressive strength (Saturated) MPa
Fresh	0.9		2 95	77.6
Slightly altered	0.5 - 1.4	1.4 - 3.9	2.85 - 2.95	60
Moderately altered	0.8 - 2.7	2.3 - 7.5	2.80 - 2.9	34

Shear Parameters of the slickensided Chlorite coated joints :

Laboratory shear tests of rock samples having chlorite coated slickensided joint surfaces have been done to obtain 'C' and 'ϕ' values of weak planes. After 90 days of saturation 'C' value obtained is 0.20 MPa and 'ϕ' value is 18°, 'ϕ' value remained unchanged even after 30 days of saturation (Table-3).

Table-3

Shear parameters of the joints coated with chloritic material

Period of saturation	Shear parameters	
	'C' Mpa	'ϕ' degree
Dry	0.011	22.6
3 days	0.143	20
30 days	0.095	18
90 days	0.020	18

Table-2

Physico-engineering properties of Dolerite

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Fresh	0.9		2.95	77.6
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90 days	0.020	18

Rock-mass Classification :

The rockmass has been evaluated by adopting Barton's 'Q'-system and Beiniawski's RMR method. Four units of the dolerite rock have been classified for designing the supports (Table-4).

Table-4

Rock mass classification of the dolerite rock

Unit No.	Description	Bieniawski's RMR	Barton's 'Q'
I	Fresh dolerite	50 to 70	3 to 14
II	Slightly altered dolerite Chlorite coating along joints	40 to 55	1.5 to 3
III	Moderately altered dolerite. Joints infilled with thick chloritic material	35 to 45	1 to 1 5
IV	Moderately to highly altered dolerite (Slaking of the rock observed on exposure to air)	30 to 40	0.6 to 1.25

Support System :

The underground structures have been opened by New Austrian Tunnelling Method (NATM). The basic principal of NATM is to utilise the rock itself as principal structural material. The tunnelling quality index 'Q' and Geomechanics classification (RMR) system have been adopted for arriving at the basic support system consisting of pattern rock bolts and shotcrete. Ribs have been introduced in the exit tunnels and draft tube tunnels after collapses

Geotechnical problems of the dolerite rock :

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 Dolerite dyke is clo,ely to moderately spaced and dolerite sill is moderately to widely spaced jointed. Most of the joints are slicken-sided. Sub-horizontal shears are traversing the dolerite sill in enechelon pattern. Spacing, orientation, thickness and nature of the infilling material of discontinuities have played important role in the stability of the structures besid presence of shears and slaked zones in the rock mass.

Access tunnel :

The D-shaped 8.5m wide and 9m high, access tunnel passes through basalt and agglomerate for a length of 230m and dolerite dyke/sill in the remaining 630m length. The initial suport system comprised 25mm dia 6m long pattern rock bolts at 1.75 m. C/C with 2 layers of 38mm thick shotcrete with wiremesh in between.

Roof of the tunnel passing through dolerite sill in 67m length has become flat due to presence of sub-horizontal shear near the crown. Step like profile at haunches is formed due to the intersection of vertical and sub-horizontal joints (Fig. 3). As a remedial measure spacing of the pattern rock bolt has been reduced from 1.5m to 0.75m in this reach.

Part of the tunnel passing through vertical dolerite dyke, in 10m length, is located in closely spaced chlorite coated jointed rockmass (RMR=35.-Q=0.6). Support system was reviewed and steel ribs were installed in this reach.

Draft tube tunnels :

The draft tube (D.T.) tunnels of 10m diameter are passing through dolerite sill dissected by chlorite coated joints and sub-horizontal to low dipping shears. Initial support system for the tunnels comprised 20mm diameter and 4.0m long rock bolts at 1.75m C/C and 38mm thick shotcrete. These tunnels were proposed to be lined by 300mm thick concrete.

Major part of the tunnels are passing through dolerite sill. Subsurface exploration has revealed the presence of slaked zones in about 10% tunnel length. Roof falls have occurred in the draft tube tunnel-2 and 3 near interconnecting galleries. Rock mass (RMR=41, Q=0.63) in this are is dissected by chlorite coated joints and shears. Major rockfall occurred in the D.T-2 on 6 11.90 between R.D. 74 and 86m along the sub-horizintal shears involving overbreaks of the order of 4.5m in height even after the installation of rock bolt supports. Overbreaks in flat roof have been noticed in all the tunnels passing through dolerite sill (fig. 4). Rib supports have been installed in all the tunnels after the collapses except in about 80m length of the D. T-I and 14m length of D. T-2 where chlorite coating along joints was almost negligible. Reinforced lining has been done in the reaches supported by rock bolt.

Exit Tunnels :

Horse shoe shaped exit tunnels of 12.5m diameter are passing through basalt, agglomerate, dolerite dyke and sill. The initial support system consisted of 25 mm dia, 6m long expansion shell type of rock bolts, spaced at 1 to 1.5m centres with two layers of shotcrete each 38mm thick and welded wiremesh in between.

The Akkalbar fault runs parallel and close to the alignment of the Exit tunnel (E.T.)-1 from the outlet portal (RD 0m) to kink point (RD 222m). Joints sympathetic to the fault are traversing all the three tunnels but they are more porminent in the E. T-1. Sub-surface explorations of the tunnels have revealed that about 60% tunnel length of E.T.-1 and 30% tunnel length of E T-2 are occupied by slaked dolerite rock.

Major part of roof collapses have been observed in the E. T-1. Heading part of the E T-1 was initially excavated from the outlet portal side by providing rock bolt and shotcrete. A few cracks (about 4 to 10mm wide) were observed in the crown at R.D. 20, 34 to 36 and 55 to 57m in the month of May 1990 i.e. about one and half months after the excavation of the tunnel. Slipping of the rock bolts during tensioning was earlier noticed between R. D. 70 and 96m in the area of slaked rock zone. Steel ribs (ISMB 300 x 140mm at 350mm centres) were erected in the excavated tunnel section to support the distressed rockmass. This tunnel was twice filled up with flood water in the month of June and September, 1990 prior to the proper back packing of the ribs. Roof fall was observed in 67m length on 29th. Sept., 1990 after dewatering of the tunnel. Overbreaks of the order of 4 to 5m have been noticed. Steel ribs were found twisted. Some of the rock bolts fell along with rock block and a few remained in position with their anchorage inside the rock. Rock falls extended with time in the entire excavated and rock bolt supported tunnel section between R. D. 0 and 133m. Roof falls continued even after reducing the tunnel section to 7m in the slaked rock area. Finally entire tunnel was supported by rib supports. Rib supports have also been provided in the E. T-1 and E. T-2 in view of the similar rock conditions.

Discussions :

Extensive geological investigations have been done for the tunnels to delineate the weak feature and classify the rock mass. The NGI'S Tunnelling Quality Index (Q) and CSIR Geomechanics classification (RMR) systems were adopted for arriving at the basic support system involving rock bolts and shotcrete. Accordingly, the tunnels were opened by New Austrian Tunnelling Method (NATM) as per advice of the Central Water Commission. Roof falls occurred in the tunnel section of E. T.-1 passing through slaked dolerite even after providing rock bolt supports. About 50% rock bolts slipped during tensioning. This necesitated review of the support system. Roof falls also occurred in the draft tube tunnels passing through dolerite sill dissected by sub-horizontal shears causing over excavation and flat roof at places. Pattern rock bolts could not prevent rock falls in the exit tunnels and draft tube tunnels in the rockmass dissected by moderately to widely spaced cholorite coated slickensided joints. In view of the continuation of the rock falls, rib supports have been introduced in the exit tunnels and draft tube tunnels to stabilise the rock mass.

Conclusions :

Support system based on the rock mass classification consisting of pattern rock bolts and shotcrete was designed initially for all the uuderground openings. After observing the behaviour of rock mass during progressive excavation, rib supports were installed after the collapses in the tunnels. Designing of the support system by adopting the available emperical methods like Barton's 'Q' and Bieniawski's 'RMR' system has not resulted in the safe execution of the tunnels in the chloritised and slaked dolerite rock. Rib supports are recommended for such type of rock mass.

Acknowledgement :

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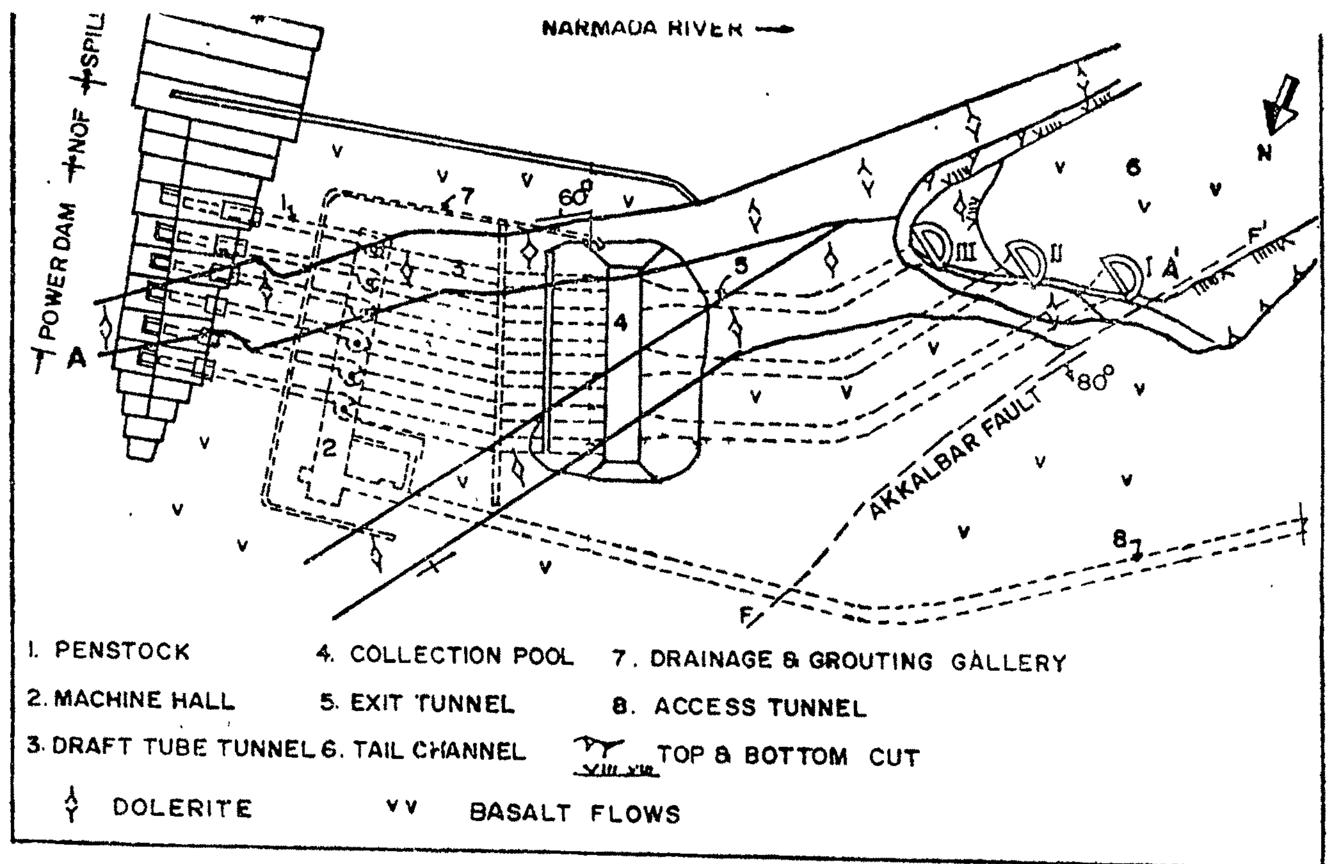

Fig I GEOLOGY AND LAYOUT OF UNDERGROUND POWERHOUSE

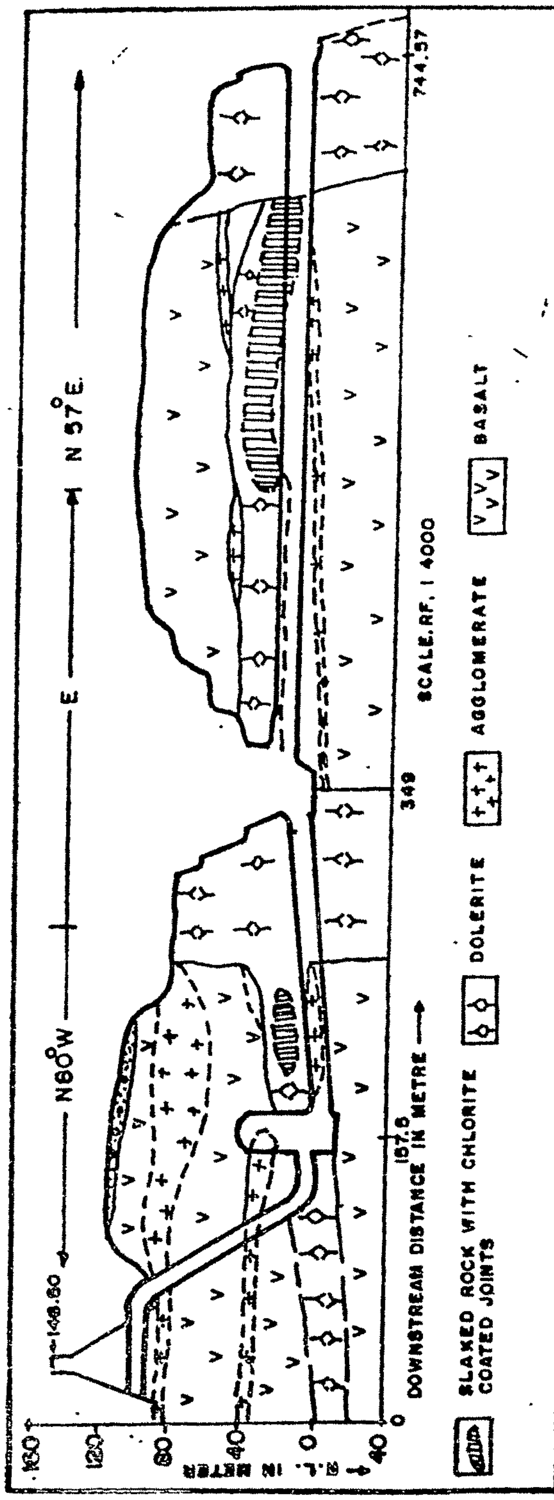

Fig. 5 - GEOLOGICAL SECTION ALONG WATER CONDUCTOR SYSTEM (A - A')

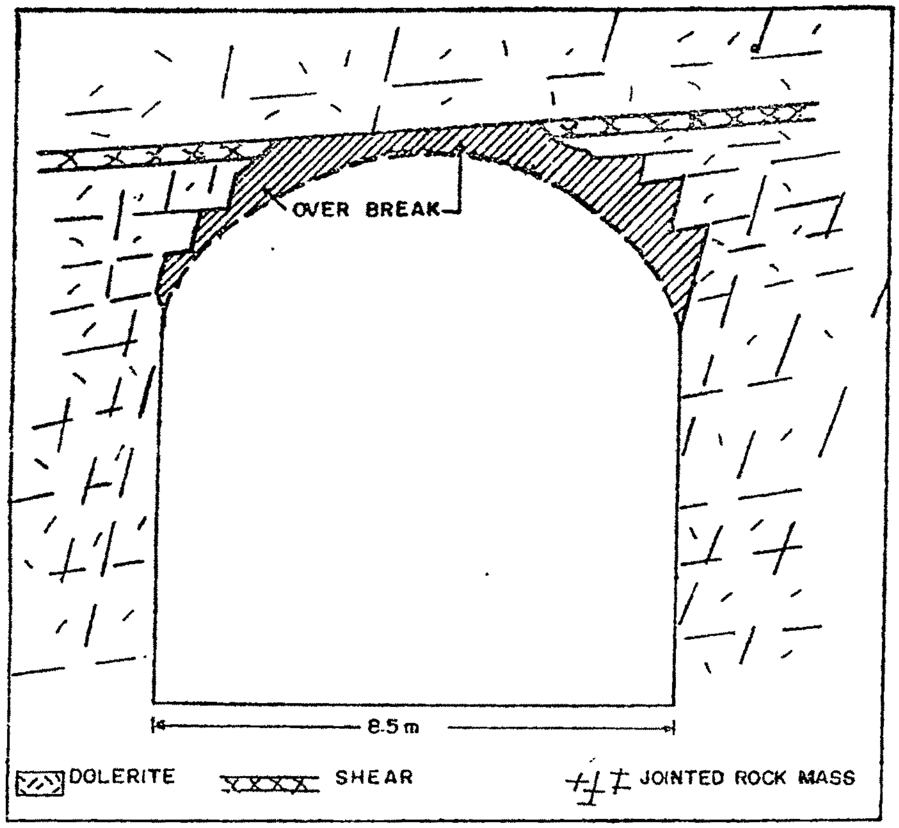

Fig.3 GEOLOGICAL CROSS-SECTION OF ACCESS TUNNEL SHOWING OVER BREAKS

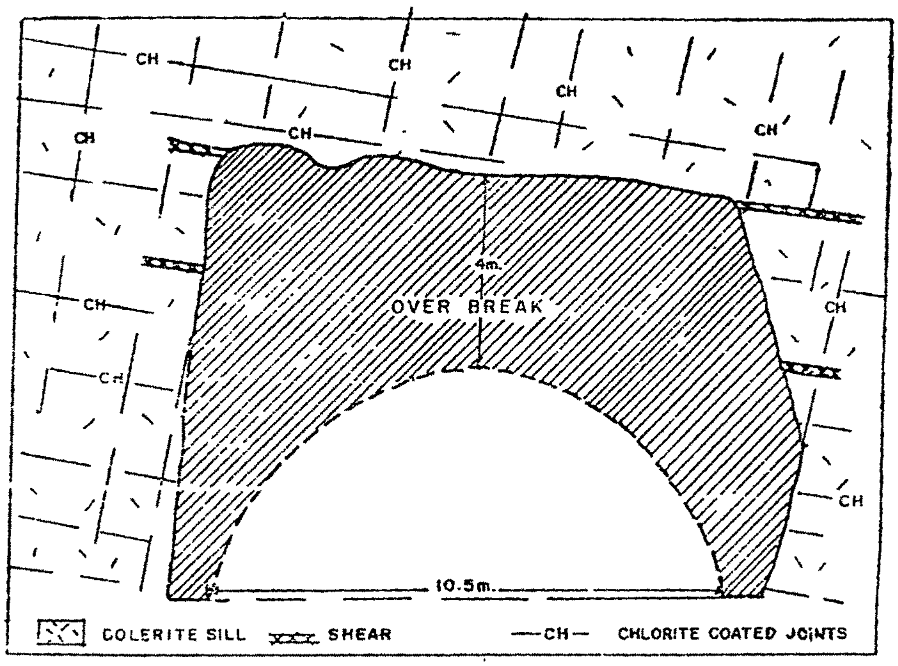

Fig. 4 - GEOLOGICAL CROSS SECTION OF PART OF THE DRAFT TUBE TUNNEL-2 SHOWING OVER BREAKS ALONG SUB-HORIZONTAL SHEARS