4. Engineering geological study of dam sites, Identification of Geotechnical problems and treatment

4.1 Engineering geological investigations at Narmada dam site

The first reconnaissance survey for a suitable site in the Lower Narmada Valley in Gujarat was carried out in the year 1947. Total nine sites were investigated. Out of these nine sites detailed investigations were carried out for three sites between Mokhadi and Limdi village. Finally present site near Navagam village was selected based on the topographical and geological conditions. Pre-construction stage investigations for the present Navagam site commenced in the year 1962 and construction stage investigations started in the year 1978. Detailed construction stage geotechnical investigations at site included drilling about 30 running km rock 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 1000,000m^2 area for the assessment and evaluation of rock mass conditions of dam foundations and underground structures was done.

Weak geological layers/ features identified in the Deccan basalt flows included tuff, agglomerate, red bole, highly jointed zones, shears and faults. Weak sheared argillaceous sandstone layers (Infra-trappean Bagh beds) also occur in juxtaposition with basalt at shallow depth in the foundations of Narmada dam (Fig. 3) (Appendix-III).

Field testing of the rock mass and laboratory testing of the rock cores samples were done to assess the physico-mechanical properties of rocks as well as rock mass (Prakash 1990). Major tests conducted at site included in situ deformation modulus tests on fault zone material and surrounding rocks, in situ shear tests for red bole, sheared contacts of sedimentary rocks and interfaces of rocks and concrete. Tracer studies were done for determining seepage losses through

Limestone. Blast tests were conducted for estimation of design seismic coefficient. Hydro-fracture tests were done to know the stresses around the underground powerhouse. 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 were also done.

Based on the engineering geological investigations following geotechnical problems were identified at Narmada dam site:

	Main geological features responsible for the problems
(1) Sliding	Red bole, tuff layer, argillaceous sandstone layers and sub-horizontal to low dipping shears
(2) Settlement	River channel fault, minor faults and jointed weathered rocks
(3) Seepage	Limestone and river channel fault
(4) Seismicity	Faults and shears especially Piplod fault

4.2 Engineering geological investigations at Karjan dam site

The first reconnaissance survey for a suitable site in the Karjan river valley was done in the year 1952. Total two sites were investigated near village Jitgadh and Nani Bej in Nandod Taluka, near Rajpipla town during the period 1968-69. Site near village Jitgadh was finalised on the basis of topography and geology. Pre-construction stage investigations for the present Jitgadh dam site were started in the year 1970 and construction stage investigations in the year 1978. Surface and sub-surface investigations were done. Number of exploratory drill holes in conjunction with exploratory shafts conclusively established occurrence of weathered rock seams at 4 to 10m intervals within a depth of 20m from the general foundation level of the dam. These weathered rock seams posed foundation problems of sliding, settlement and seepage (Appendix-IV).

4.3 Sliding stability

The vertical forces (mainly weight of the dam) and horizontal forces (mainly lateral pressure of reservoir water) acts on the foundation of the dam. Thrust of the water in the reservoir tends to push the dam horizontally, towards the downstream side. The resultant of these two mutually perpendicular forces is inclined downstream. The exact amount of this resultant is dependent on the magnitude of the two forces involved and ranges from 10° to 30° with respect to the vertical (Thomas 1979). The buoyancy force and pore pressure acts upward and often decreases vertical forces (i.e. the sum of the static forces acting downward) with detrimental effect. The sliding of a dam is resisted by friction and cohesion, either at the boundary of the dam and the foundation, or along seams within the foundation. The dam may slide if the horizontal forces are excessive i.e. more than the forces resisting the slide.

4.3.1 Influence of sub-horizontal to low dipping weak features in the sliding stability

Stability against sliding along any of the weak litho-units/ layers/ seams/ shears/ faults is to be examined in relation to:

(a) The in-situ shear strengths of material in weak litho-units/ layers/ seams/ shears/ faults.
(b) The probable extent and location of the maximum scour on the downstream side resulting in reduction of downstream passive resistance due to the exposure of the weak litho-units/ layers/ shears/ seams/ shears/ faults in the anticipated scour pits.
(c) The resistance offered by the masonry to rock and concrete to rock contacts.
(d) The extent of simultaneous mobilisation of the strengths of the weak litho-units/ layers/ seams/ shears/ faults material and the concrete/ masonry to rock interfaces.
(e) The geometry of the weak litho-units/ layers/ seams/ shears/ faults.

(f) The position and stages of the actual construction work already carried out in the spillway blocks.

4.3.2 Sliding factor

The sliding factor may be defined as the ratio of the sum of all horizontal forces and components of loading that tend to cause sliding of the dam on its foundation to the sum of all vertical forces and components of loading (Thomas 1979). This ratio should lie within the range of 0.65 and 0.75 for normal loading with a higher value up to, say, 0.85 under extreme loading combinations, i.e.,

$\Sigma H/\Sigma V$=0.65 to 0.75 for normal loading

$\geq$0.85 for extreme loading.

The actual value permissible at a particular site will depend upon the soundness of the rock, the slope of the foundation and the keying effect provided.

4.3.3 Shear Friction Factor

The Shear Friction Factor (SFF) may be defined as the ratio of the total resistance to shear failure within the dam, at its contact with the foundation or within the foundation to the total horizontal load. This can be represented by the formula:

$SFF= (V \tan\phi + CA)/ H$

Where SFF= the Shear Friction Factor

V= the sum of the vertical loads (or loads normal to the plane)

A= the are of the plane of contact

H= the some of the horizontal loads (or loads parallel to the plane)

ϕ= the angle of internal friction

C= the ultimate shear resistance of concrete or rock.

In general $\tan\phi$ will lie in the range of 0.6 to 0.75 but may be lower along joints, shears, faults or seams in the foundation (Thomas 1979). If such features are present, in situ tests are performed to know the shear characteristics of the infilling material of discontinuities.

The shear friction factor should not be less than 5 for normal loading and may be acceptable as low as 4 for the extreme combinations of loading. However, some countries specify values of 4 and 3, respectively for SFF but each site should be considered as unique, and investigation should be conducted appropriate to the size and importance of the dam (Thomas 1979).

4.3.4 Factors of safety against sliding

The factor of safety (FOS) of dam is defined as the ratio of the sum of the resisting forces tending to prevent sliding divided by the sum of the active forces tending to produce sliding. Thus the Shear Friction Factor (SFF) of Safety has a certain critical value beyond which a structure is considered safe. The minimum values of factor of safety laid down in the Indian standard (IS: 6512), for the different load conditions are as below (Table 21):

Table 21: Shear friction factor values under different load conditions

No.	Load condition	Shear Friction Factor values
1	Full Reservoir Level (FRL) without earthquake and with drains operative (condition B)	4.0
2	FRL with earthquake and drains operative (Condition E)	3.0
3	FRL with earthquake and drains inoperative (condition G)	1.5

4.3.5 Sliding stability of Narmada dam

A number of sub-horizontal to low dipping weak layers like red bole, tuff, agglomerate, shale, argillaceous sandstones and shears were encountered in the foundation of spillway blocks having low values of shear parameters (Table 22).

Table 22: Average values of shear parameters of the foundation rocks and weak zones

Rock/ interface	Cohesion 'C' MPa	ϕ degree	Remarks
Quartzitic sandstone	-	44	Tests indicated very low values of cohesion hence 'C' was considered zero and neglected in the design.
Argillaceous sandstone	-	17	
Upper contact of argillaceous sandstone and lower contact of quartzitic sandstone	-	11	
Lower contact of argillaceous sandstone and upper contact of quartzitic sandstone	-	26	
Contact of basalt and agglomerate	-	18	
Pebbly sandstone	-	45	
Red bole	0.08	17	
Contact of dense basalt and amygdaloidal basalt	-	47	
Weathered joint in basalt	-	27 to 39	
Concrete rock interface			
(a) Dolerite rock	0.71	53	
(b) Basalt rock	1.02	66	

Stability analysis indicated that the Narmada dam does not satisfy criteria for design of solid gravity dams for factor of safety against sliding as per IS specification (6512). Therefore, to achieve the desired factor of safety against sliding concrete shear keys were provided for the treatment of argillaceous sandstone layers and red bole layer. Treatment was on the similar line of treatment provided for the Itaipu dam (Brazil). Itaipu dam was having identical foundation problems of sub-horizontal weak layers associated with basalt flows (Moraes et.al. 1982 and Parmar and Java 1990).

a. Treatment for safety against sliding of red bole layer: Sliding stability problem was posed by red bole layer in the foundations of spillway blocks 28 to 42 on account of its low shear parameters (ϕ=17^o). To increase shear friction factor and thus to prevent sliding of these blocks underground treatment was done for the red bole layer by excavating 3m wide drifts through approach shafts in grid pattern at right angle to each other leaving 4.5 x 8.5m rock pillar between them (Fig. 17). The drifts were excavated in such a way that red bole layer was intercepted at mid height of the drifts and back filled with concrete. Open concrete shear keys were also provided where rock cover was less than 5m (Plate 10). Total area of the treatment of red bole in each block is approximately seventy percent of the foundation area. Total rock excavation for the red bole treatment through shafts and drifts aggregating to 4858m running length was 47590 cum.

b. Treatment for safety against sliding for sedimentary rocks: Sedimentary rocks are present at relatively shallow depth (about 10 to 18m below the general foundation levels) in the foundations of Right Bank spillway blocks-44 to 51 in juxtaposition with basalt flows. Riverbed fault has brought these sedimentary rocks at higher level. Contacts of argillaceous sandstone with quartzitic sandstone are sheared (ϕ =11^o). Argillaceous sandstone layers are having phi (ϕ) value 17^o. In view of low values of shear parameters of argillaceous sandstone layers as well as their contacts with quartzitic sandstone layers, underground concrete shear keys were provided to increase shear friction factor of the right bank spillway blocks-44 to 51. Average thickness of both the argillaceous sandstone layers is about 2.5m. These layers are separated by about 3m thick quartzitic sandstone. Upper argillaceous sandstone layer is overlain by 0.5 to 2m-tuff layer having sheared contact. Treatment to these weak layers was provided by excavating 3m wide and 3.6 to 6m (average height 4.5m; 2.5m+2m) high drifts through approach shafts in a grid pattern leaving rock pillar of size 8.5 x 8.5m (Plate 11 to 16 and Fig. 18).

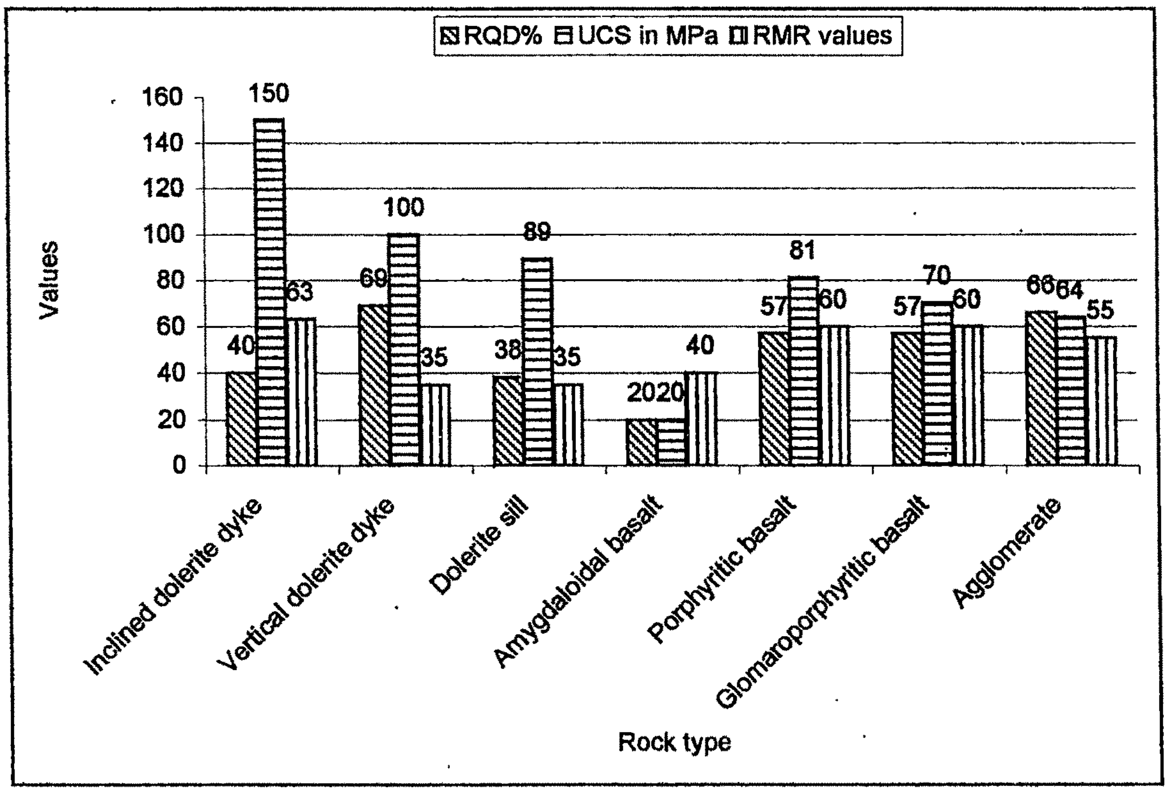
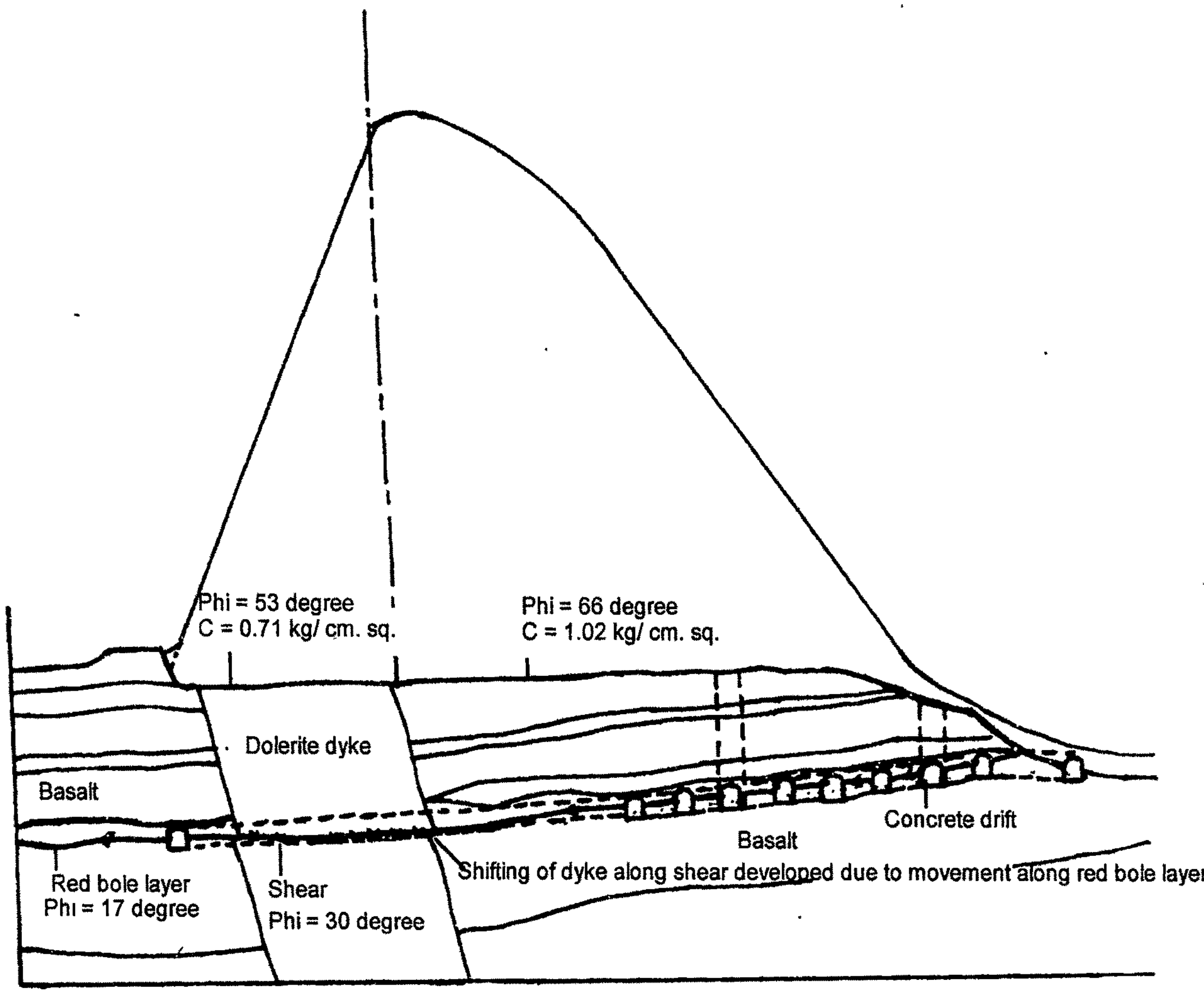
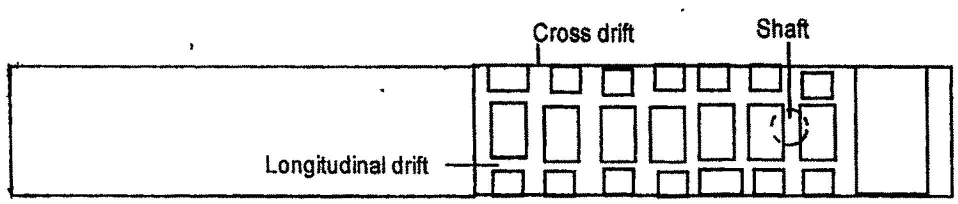


Fig. 16: Rock Quality Designation (RQD), Unconfined Compressive Strength and Rock Mass Rating of underground powerhouse, Sardar Sarovar (Narmada) Project

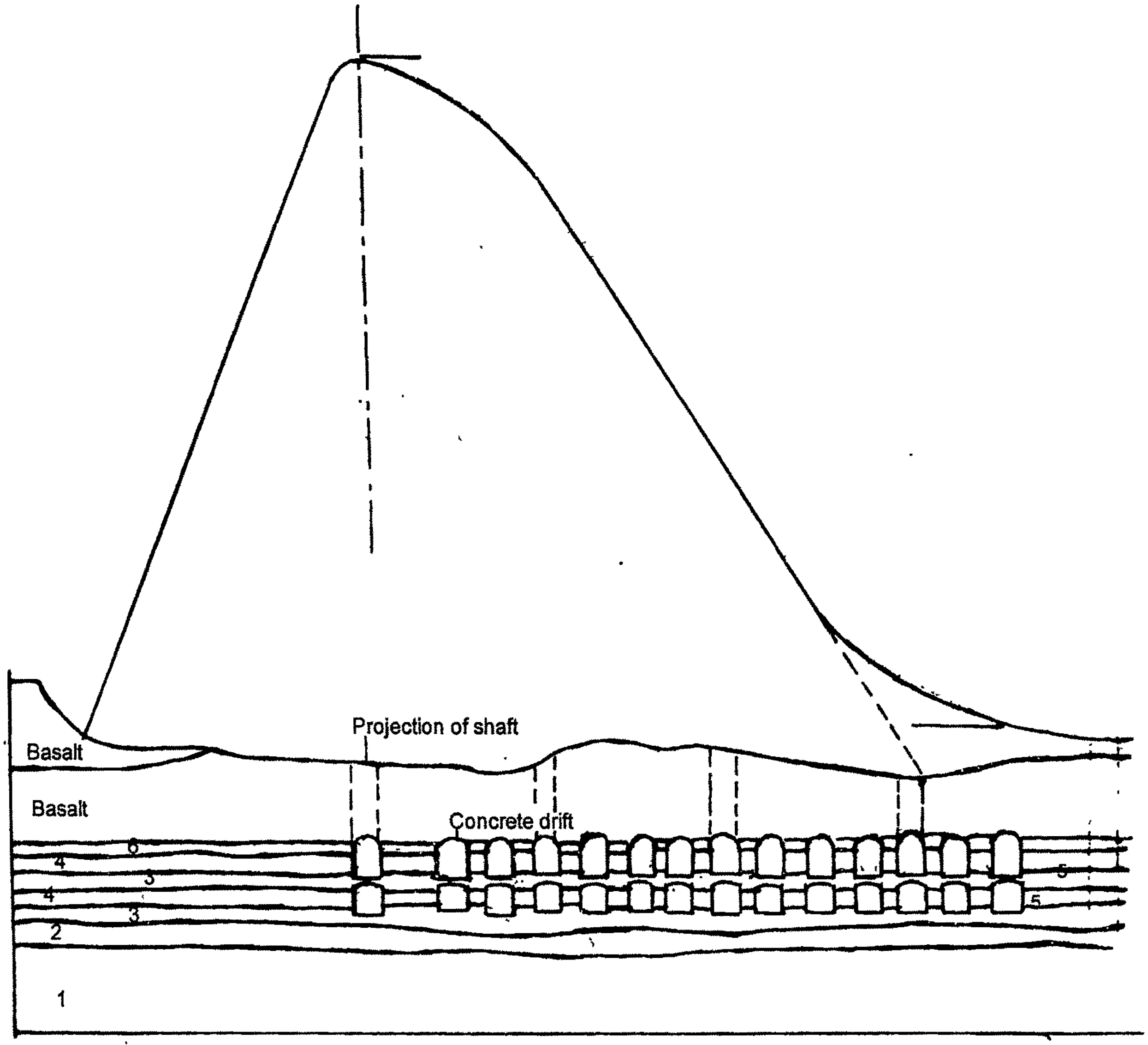


I. Geological cross-section of spillway block showing treatment of red bole layer



II. Foundation plan of treatment drift

Fig. 17: Treatment of red bole layer in the foundation of Narmada dam left bank spillway block



	Phi (degree)
6. Tuff	-
5. Contact of argillaceous sandstone and quartzitic sandstone	11
4. Argillaceous sandstone	17
3. Quartzitic sandstone	44
2. Pebbly sandstone	45
1. Limestone	-

Fig. 18: Geological section of spillway block showing treatment of argillaceous sandstone layers at Narmada dam

(a) Disposition of red bole layer in the foundation of spillway block

(b) Open concrete shear key in the foundation of spillway block

Plate 10: Disposition and treatment of red bole layer at the toe of spillway Block, Sardar Sarovar (Narmada) Project

Plate 11: Treatment of the argillaceous sandstone layers in the Right Bank foundation of spillway blocks through shafts, Sardar Sarovar (Narmada) Project

Plate 12: Treatment of upper argillaceous sandstone through drift and shaft in the foundation of spillway block, Sardar Sarovar (Narmada)Project

Plate 13: Sub- horizontal shear exposed in the treatment drift traversing the argillaceous sandsone, Sardar Sarovar (Narmada) Project

(a) Sheared tuff layer exposed in the crown of the drift

(b) Colcrete in the upper (crown) part of the drift

Plate 14: Treatment drift for the upper argillaceous sandstone layer, Sardar Sarovar (Narmada) Project (a & b)

Plate 15: Excavation of approach drift for the treatment of weathered rock seam in the foundation of block L-5, Karjan dam

Plate 16: Pump concrete in the crown of treatment drift in the foundation of spillway block, Karjan dam

Tuff layer was removed from the crown of the Upper argillaceous sandstone treatment drifts. Crown of the upper drifts was excavated in the sound basalt and of lower drifts in the quartzitic sandstone. The bottom of the drift was kept just below the argillaceous sandstone in the quartzitic sandstone for proper keying. The drifts replacing the lower and upper argillaceous sandstone are lying directly one over the other separated by upper quartzitic sandstone. Location of drift one above the other was planned to avoid concentration of stress on the weak layers and for easier drilling for contact grouting. In blocks 49 and 50, thickness of quartzitic sandstone in between argillaceous sandstone layers is less than a meter; hence, some of the lower and upper drifts were combined from safety consideration. The maximum height of the combined drifts is 12.5m.

Shape of the roof of drift is semicircular. Experience of the Karjan dam and experimental drifts at Narmada dam showed that there was large shrinkage gap in case of pump concrete in the crown. Therefore, colcrete was done in the crown (i.e. 1/3 height) of the drift for better concrete and rock contact (Plate 14). Consolidation-cum-contact grouting through holes spaced at 2m centre to centre (c/c) was done in grid pattern in the foundation of treated blocks to ensure good contact of the concrete, colcrete and rock. Grout will also fill voids, if any, remaining due to concrete shrinkage and/or inadequate filling of irregularities in the rock surface, besides sealing of open joints (Prakash 1990).

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

c. Treatment for safety against sliding of Low dipping shears: During the excavation of shafts and drifts for the treatment of sedimentary rocks, a low dipping shear ('ϕ' = 30°) was encountered in the overlying basalt in the foundations of Right Bank spillway blocks 46 and 47. Based on the stability analysis treatment to the shear zone was provided by constructing open concrete shear keys in these two blocks.

d. Alternative measures: Following alternative methods were provided to increase stability of the dam:

(i) Curvature in the axis of dam: A mild curvature in the axis of the dam was provided of power dam, non-overflow and spillway blocks to mobilise shear resistance of all monoliths together to ensure greater safety against sliding.

(ii) Designing of suitable energy dissipation arrangement: Energy dissipation arrangement for the service spillway has been designed sloping-cum-horizontal jump type stilling basin to retain downstream rock mass overlying weak layers to act as passive resistance against sliding. Energy dissipation arrangement for the auxiliary spillway is split-level chutes terminating into ski-jump bucket.

4.3.6 Sliding stability of Karjan dam

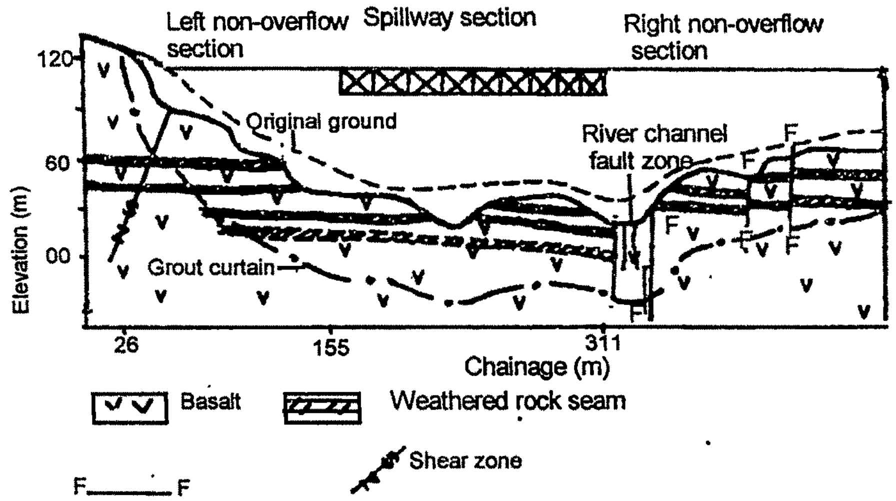
Numerous weathered rock seams encountered in the foundation of spillway blocks at 3 to 5m intervals and in the Non over flow blocks at 4 to 10m intervals. These seams posed the problem of sliding of the dam blocks. In situ tests of seams indicated values of cohesion almost negligible ((C)$\simeq$0) and angle of internal friction (ϕ) =22°, 24° and 26° varying at different locations. Shear values obtained for concrete over rock contacts were C= 4 kg/cm^2 and ϕ=50°. Factor of safety against sliding (F1) 1.5 and shear friction factor (F2)=3.0 was considered in the design. Minimum value of SFF was scaled down from 4.0 for condition B (FRL with earthquake and drains operative) as stipulated in IS: 6512-1972 to 3.0 as per United State Bureau of Reclamation (USBR) practice. This was done in view of the fact that intensive investigations had been carried out narrowing down the margin of uncertainty. Moreover, the shear values were based on actual tests carried out on site. The earthquake factor of 0.125g (identical to Narmada dam) was considered in the design.

a. Treatment considered to prevent sliding of dam blocks resting over weathered rock seams: In order to make various spillway blocks safe against sliding various remedial measures were considered including curvature in the alignment of dam, flattening of upstream batter, roughening of the foundation base and combining of two or more blocks besides concrete shear keys.

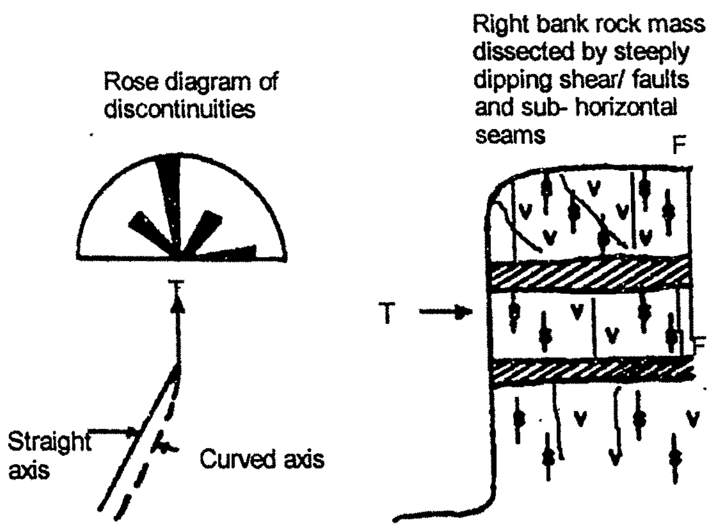
(i) Curvature in the alignment of dam: It was thought to lay the blocks in the riverbed in the manner of an arch with a flat mild upstream curvature in order to induce the blocks to act as a monolith for mobilising greater combined resistance against sliding. Idea of curvature in the alignment of dam was dropped due to two reasons firstly, all the blocks had seams below them secondly, abutment rocks were not found geologically suitable for arch action (Fig. 19).

(ii) Alternative measures: Alternative measure for increasing the stability of blocks over weathered rock seams such as flattening of upstream batter, roughening of the foundation base for greater friction, combining of two or more blocks to act as monoliths were adopted. Although all these measures did improve the factors of safety against sliding, they fell short of required minimum values of sliding (F1) and shear friction factor (F2) necessitating provision of concrete shear keys (open or underground drifts/concrete plugs).

Treatment to weathered rock seams in various blocks was provided by open concrete shear keys where seams were located at shallow depth and in the form of concrete drifts (plugs) wherever sufficient rock cover was available over the seam and also where dam blocks were partially constructed (Prakash and Vyas 1998) (Table 23).



I. Longitudinal geological section of dam



T- Direction of resultant thrust in case of curved dam axis

II. Critical geological abutment conditions

Fig. 19: Geological sections of Karjan dam showing foundation condition

Table 23: Rock mass classification of the foundations of blocks-L3, L5, Sp0-Sp7, R0, R1A and R1B and treatment of weathered rock seams

Block No.	Flow units and associated weak features	RMR	Class No.	Remarks	Treatment
L3	F3, blocky, thick weathered rock seam	40	IV	Block size (BS2-BS3), Low value of RMR due to weathered rock seam (W3-W4, UCS4)	Removal of seam from 2m wide strip adjacent to block L3 and L4 joint.
L5	F1, weathered rock seam	44	III	Weathered rock seam (W3-W4, R5-R6)	Concrete shear keys.
Sp0 – Sp7	F1, weathered rock seam	44	III	Weathered rock seam (W3-W4, R5-R6)	Concrete shear keys.
		24	IV	Weathered rock seam (W3-W4, R5-R6) (For sliding consideration lower RMR)	Concrete shear keys.
R0	F1, main river channel fault	44-48	III	Fault zone healed (HL1-HL2); seams are thinning out (A5-A6).	-
R1A	F1 to F3, main river channel fault, weathered rock seam	44-48	III	-do-	Concrete shear keys.
R1B	F3, weathered rock seam	65	II	-do-	Concrete shear keys.

Note:

1. Flow unit–1 is dissected by a number of weathered rock seams. These seams (ϕ=22° to 26°) are considered potential weak planes for sliding but may not pose problem of settlement as they are of pinching and swelling nature. Hence, different rating adjustment values have been adopted for discontinuities for sliding (-25) and settlement (-5).
2. Treatment was provided for sliding. No treatment was provided for settlement.

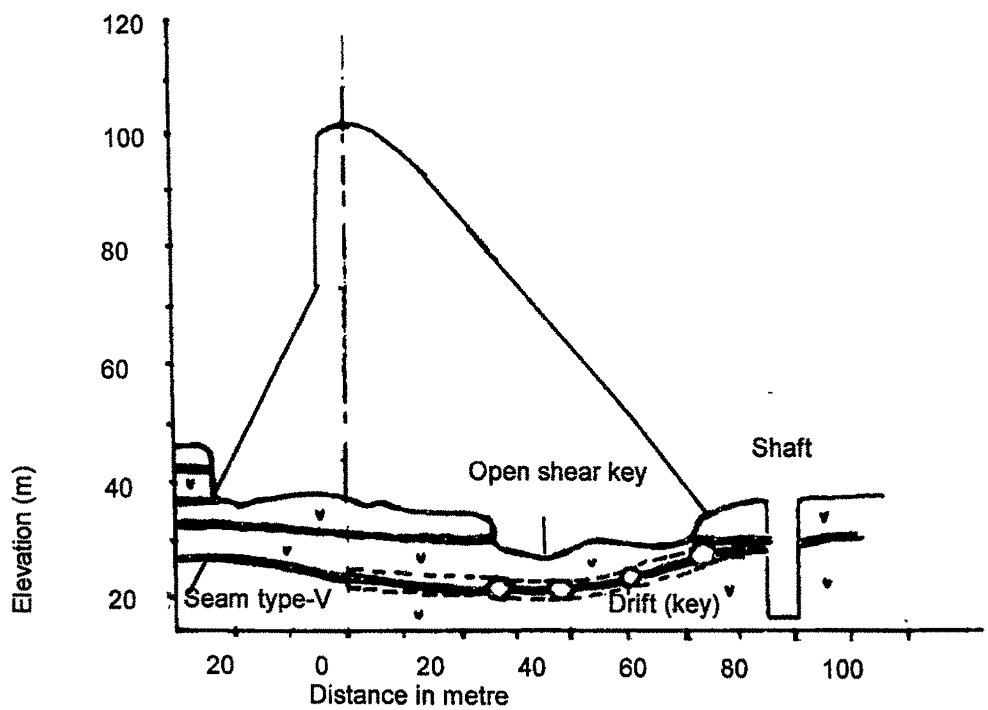
b. Treatment of Left non-overflow (LNOF) blocks: Weathered seams encountered in the foundation of LNOF blocks at different elevations were analysed. Removal of the weathered seams was done in the foundation of block L-4 where rock cover was shallow (3 to 4m) and concrete shear keys were provided in block L-5 for deeper seam (Plate 15).

c. Treatment of spillway blocks: Spillway blocks-1 to 7 are resting over various weathered seams located at varying depths in the foundations. Sliding analysis showed that three to four drifts/plugs of 6 to 8m widths were required for the spillway blocks-1 to 5, for ensuring minimum required values of factors `F1' and

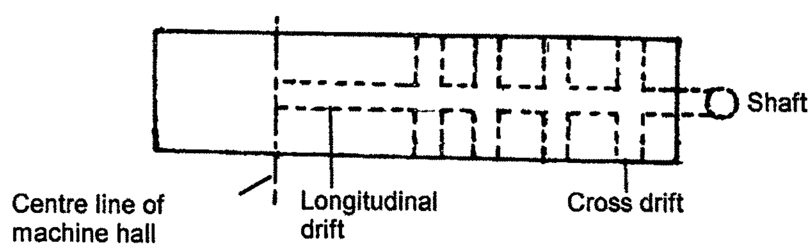
`F2'. Accordingly, concrete drifts were provided in the foundation of spillway blocks-1 to 5 and open concrete shear keys in the foundation of spillway blocks-6 and 7 (Fig. 20). Pump concrete was done in top 1/3 rd. portion of the drift (Plate 16). Shear keys were provided in the foundation of spillway blocks-6 and 7, without any joint in between to take the advantage of the available cross shear though it has not been considered in the stability analysis.

d. Treatment of right non-over flow (RNOF) blocks: The factors and criteria involved in the stability analysis of the RNOF blocks over the weathered seam were similar to those adopted for the left NOF and spillway blocks. Shear failure was considered through the downstream rock cover along a plane at an angle of 22° 33' (45°-ϕ/2) for design purpose. Treatment to the R-0, R-1A, R-1B and R-2 blocks, resting on weathered seams, was provided by excavating the drifts across the weak layers and back filling them by concrete.

e. Additional treatment: Analysis showed that to obtain extra resistance through shear keys/drifts alone, very large shear keys/drifts were required to be provided. This would have involved not only high costs but also considerable constructional difficulties as concrete was already laid in a few blocks. To keep this treatment to the minimum and also to reduce the width of the keys/drifts, it was decided to provide a flatter upstream batter of 1:2 below El. 75m in the spillway blocks, instead of original 1:15 below El. 85.70m. This would increase the vertical load due to concrete plus water coloumn thus helping in mobilising larger frictional resistance along the weak seams (Purohit et al. 1983). In order to obtain extra frictional strength (resistance) the surface of the foundation rocks were roughened; seam material was scooped out laterally to possible maximum depth inside the rock mass and contact grouting was done between concrete and rock face in the spillway and NOF blocks.



I. Geological cross-section of spillway block



II.Foundation plan of treatment drifts

Fig. 20: Treatment of Weathered rock seams in the foundation of Karjan dam spillway block

4.3.7 Treatment for sliding at other projects:

Shear keys were provided to treat sub-horizontal to low dipping shears/ fault/ weak layers on the foundations of Ichari dam (Jalote et. al. 1975), Srisalam dam (Rao and Narsimham 1975), Kadana dam (Mistry and Kulkarni 1977), Eddabhi Hydro-electric scheme, Morocco (Thomas 1979) and Itaipu Hydroelectric project, Brazil (Moraes et. al. 1982) (Table 24).

Table 24: Example from some of the dams in India and abroad where concrete shear keys were provided for the foundation treatment

Name of the project	Type of structure	Rock type	Geological features responsible for sliding problem	Treatment
Ichari dam (Uttarakhand, India)	55m high concrete gravity dam	Thinly bedded slates and quartzites	Low diping (2° to 5° downstream)beds	Upstream shear key going down to 7.5m
Srisalam dam (Andhra Pradesh, India)	143m high masonry-cum-concrete gravity dam	Quartzite, with thin interbands of shales	Interbedded incompetent shale (ϕ=30° to 33°)	Slightly arching the axis of the dam, construction of a single monolithic block, construction of an RCC toe block anchored into the foundation and four concrete shear intercepts
Kadana dam (Gujarat, India)	58m high composite earthfill-cum-masonry gravity dam	Quartzite, phyllite, quartz-biotite-schist and schist	Faults with gentle dips and associated with gougey material (ϕ=14° to 30°)	Concrete shear keys (drifts) along low angled faults (<30°) in the foundation of power dam blocks 7 to 9 and 15 were and thrust blocks in the spillway blocks 8, 9 and 14 to 19
Eddabhi Hydro-electric scheme (Morocco)	Arch dam	-	Low dipping fault dipping upstream crossing the left abutment hill	Part removal of fault material by driving number of drifts and backfilling them with concrete
Itaipu Hydroelectric project (Brazil)	196m high composite dam (Concrete-cum-rock fill and earth fill dams)	Basalt flows	Sub-horizontal weak layers (ϕ=20° to 30°) located at the contact or at the base of the transition zone	Underground Concrete shear keys

4.4 Settlement problem

The dam may settle under the action of its weight and other vertical forces (including forces due to filling of the reservoir) imposed upon it. The settlement problem is simple if the foundation rock is sound and strong and of one type. If the dam is placed on varying lithounits and foundation is traversed by shears and faults, differential settlement and rebound could be expected. The internal stresses thus imposed on the structure could be disastrous. Therefore, identification and delineation of lithounits and geological features having variances in the physico-mechanical properties are essential for the consideration in design.

4.4.1 Significance of faults in engineering

The main significance of faults in engineering lies as plane of weakness, possible source of further movement and in their effect on the physical properties of surrounding rocks. Differential movement in a fault occurs along fault plane. Surface or zone of failure is characterised by polished surfaces, striations, slickensides, gouge, breccia, mylonite, or any combination of these. A fault/ shear zone may be healed or recemented by one or more episodes of subsurface mineraliztion or precipitation of soluble materials (Table 15).

Faulting may bring different types of rocks in juxtaposition to each other as in the case of Narmada dam where a portion of the dam foundation is overlying on hard basalt and another portion on relatively soft sedimentary rocks. This may create problems of differential settlement of the foundations of dam blocks. In such cases internal stresses imposed on the structure due to variances in the modulii of elasticity of rocks as well as fault zone material are to be considered in the design.

4.4.2 Treatment of fault

Dental treatment is mainly done to treat the fault zone in the foundation. It involves selective removal of weak fault zone material from the foundation and back filling the excavated trench with concrete (plug). Treatment depends on the width of the fault zone, type of material of the fault zone and height of the structure. The depth of the concrete plug has to be such that the stresses within the plug should be compressive and the contact stresses generated on the interfaces of the plug with the supporting rock should be within permissible limits and there should not be excessive deformations.

The function of concrete plug in case of vertical or steeply dipping fault is to provide support to the bridging portion of the concrete dam over the fault and by transferring the stresses to the adjacent sound rock. This will prevent development of tension zone in the concrete dam. For an inclined fault, the concrete plug will act as a strut to counter the inward deformation of the overhanging rock mass under the load of the dam.

Theoretical studies based upon foundation conditions and stresses at Shasta and Friant Dams, have resulted in the development of following approximate formulas (United States Bureau of Reclamation (USBR) formulas) for the depth of dental treatment:

$d = 0.002\ bH + 1.524$ for $H \geq 46$m (150 feet)

$d = 0.3\ b + 1.524$ for $H < 46$m (150 feet)

Where, H = height of dam above general foundation level in metre, b = width of weak zone in metre, and d = depth of excavation of weak zone below surface of adjoining sound rock in metre, (In clay gouge seams, d should not be less than 0.1 H)

These formulas have been used in most of the dam constructed in Gujarat like Kadana, Ukai and Karjan and elsewhere in India and abroad in slight modified forms, as per local site conditions.

The above formulas are suitable for application to foundations with a relatively homogeneous rock mass with nominal faulting. Presence of different lithounits and large zones of faults affects overall strength and stability of the foundation. It requires a definitive analysis (CBIP 1988). Such a study can be performed by finite element analysis. Data required for the finite element method of analysis include dimensions and composition of the lithologic units and geologic discontinuities, deformation modulii for each elements incorporated in the study and loading pattern imposed on the foundation by the dam and reservoir.

4.4.3 Narmada Main River channel fault

Narmada Main River channel (bed) fault is aligned in N80°E-S80°W direction, dipping 60° towards N10°W. This fault has brought sedimentary rocks in juxtaposition with the basalt at the dam base (Plate 6 and Fig. 3). It is an en echelon type reverse fault having displacement of the order of 210m with up throw side towards north i.e. towards Right Bank. Width of the fault zone is about 10 to 12m (Fig. 4). The fault is associated with 5 to 15cm thick gougey materials. Rock mass adjacent to fault zone is sheared and fractured. This fault is obliquely traversing the foundations of four spillway blocks 41 to 44.

Fault zone material consists of sheared basalt, basalt dyke and sedimentary rocks. Basalt flows dissected by red bole form the left abutment and basalt flows underlain by tuff rock and sedimentary rocks form the right abutment. Red bole layer on the left abutment and sedimentary rocks and tuff rocks are associated with sub-horizontal shears. USBR criteria have been adopted for the description of characteristic of the fault zone (Fig. 4).

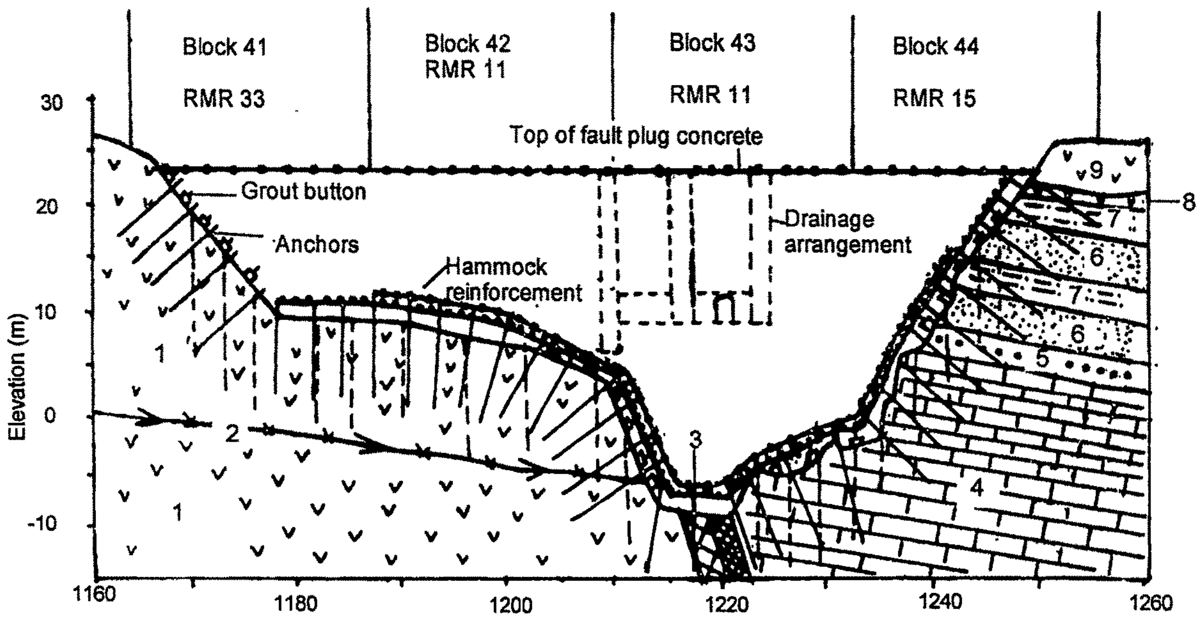
a. Rock mass classification of fault zone and foundation rocks traversed by fault: Foundations of the spillway blocks 40-44 are traversed by river channel fault. Rock mass rating of the fault zone was done to assess and evaluate rock mass conditions of these blocks affected by fault (Fig. 21) (Table 25 & 26). The Bieniawiski's 1976 rock mass classification system was adopted which gave RMR value of the fault zone as 11 and adjacent blocks 33 i.e. very poor to poor category rocks, respectively.

Table 25: Rock mass classification of the Narmada dam river channel fault zone

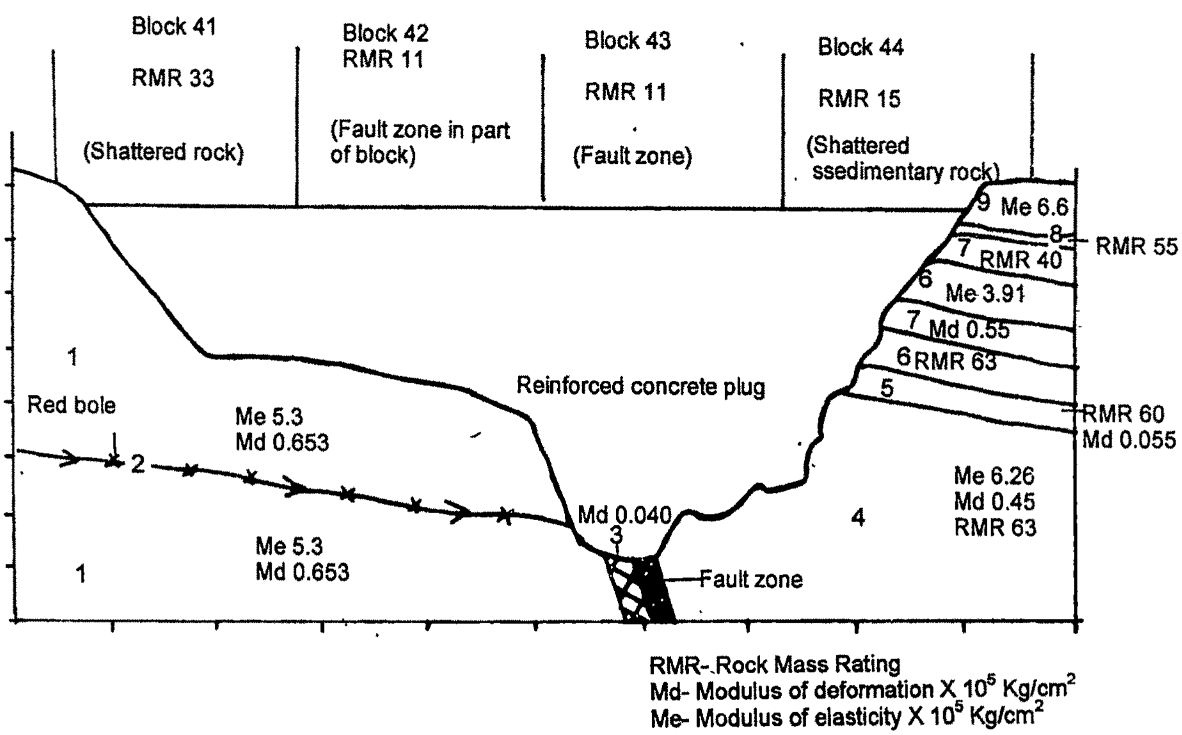
No.	Parameter description	Value	Rating
1	UCS	5-6MPa (Approx.)	2
2	RQD%	25	3
3	Spacing of discontinuities	<60mm	5
4	Condition of discontinuities	Gouge >1cm thick	5
5	Ground water condition	Dripping	4
6	Discontinuity orientation	Fair-Unfavourable	-8

Total rating (RMR): 11
Class No.: V
Class description: Very poor

b. Geotechnical assessment of the fault zone: Rock mass rating of almost unhealed (HL4-HL5) fault zone material is very poor (RMR 11). In situ test results indicated low values of modulus of deformation for the fault zone (0.04 x 10^5kg/cm^2) and relatively high values for the basalt (0.653x 10^5 kg/cm^2) and sandstone (0.38x 10^5kg/cm^2). Ratio of modulus of elasticity and modulus of deformation of the basalt adjacent to fault zone vary from 1.87 to 2.4 and of sedimentary rocks from 2 to 4 indicative of weathered and jointed nature of the rock mass (Fig. 10). In view of the low modulus of deformation of fault zone and high modulus ratio of the abutment rocks of varying physico-engineering properties (Fig. 9 to 11), problem of differential settlement in the foundations of riverbed blocks 41 to 44 was apprehended.



I. Treatment of fault zone



II. Rock mass rating and modulus properties of fault zone and adjacent rocks

Fig. 21: Rock mass characteristics and treatment of Narmada river channel fault at the dam base

Table 26: Rock mass classification of the foundation rocks of dam blocks (40 to 44) traversed by fault zone

Block No.	Flow/ Rock type and associated weak features	RMR	Rock mass classification	Category	Remarks
40 and 41	L-3, L-4, Agglomerate and red bole, shattered zone, closely spaced joints	33	Poor rock	IV	Low RMR due to shattered zone
42	Sedimentary rocks, L-2, L-3, fault zone, red bole	11	Very poor rock	V	Low RMR value due to fault zone
43	Sedimentary rocks, L-2, L-3, fault zone, red bole	11	Very poor rock	V	Low RMR value due to fault zone
44	Sedimentary rocks, tuff, R-1, minor fault, shattered zone , closely spaced joints	15	Very poor rock	V	Low RMR value due to shattered rock

c. Treatment of Main River channel fault: The treatment of the fault zone envisaged excavation of a trench (Plate 6) to remove fault zone material and sheared rock adjoining the fault zone from the dam base to a required depth and back filling it with pre-cooled mass concrete of suitable strength (Fig. 21). For determining depth of the plug two-dimensional and three-dimensional photo-elastic studies were done besides finite element analysis (Desai and Java 1983).

(i) Two dimension photo elastic studies: These experiments indicated that the maximum stress concentration decreases with increase in depth of plug and decrease was not appreciable after the depth value about 1.5b, where b is horizontal width of the fault zone. Thus plug depth of about 1.5b would be adequate for treatment of fault zone in the foundation of dam. For plug depth equal to 1.5b and 2.0b stress distributions observed were similar to those obtained in roof of the galleries in the dam. There were predominantly compressive stresses on sides of the fault zone and only minor tensile stresses in bottom of the plug. Shoulder level for reckoning depth of the plug was considered at El. 1.5m and width of the fault zone 12m. General foundation level was at El.18m. This shoulder level was considered on the basis that the plug should remain embedded below foundation rocks on all sides including upstream and downstream end.

(ii) Three dimension photo elastic studies: In the three dimension photo elastic studies ratio of depth of plug to width of fault zone varied from 1 to 3. For these values the maximum tensile stresses in the concrete plug vary from 3.0 σ_o to 1.0 σ_o and the maximum compressive stresses vary from 3.2 σ_o to 1.56 σ_o, where σ_o is vertical component of resultant load at El. 1.5m divided by area of the base of the dam. The value adopted for σ_o is 10kg/cm^2.

(iii) Limitations of photo-elastic studies: It is not possible to realistically represent elastic modulus of concrete and foundation rock in photo-elastic studies. In the above studies concrete and rock foundations were assumed to have the same modulus of deformation. Fault zone material was considered completely fractured having nil value of modulus of deformation. The fault zone was represented as complete cavity in photo-elastic studies. Therefore, as a better alternative finite element analysis was done to determine stress distribution in foundation in region of the fault and to estimate foundation deflections.

(iv) Finite element analysis: Two-dimension finite element analysis of the fault zone was done by the time lowest excavation levels in the fault zone were at El. -16.5m on the upstream and at El. –8.5m in the downstream. General foundation levels were at El. 18m. For 8 to 10m width fault zone the depths excavated to form a plug were (18 to –16m) 34m at upstream and (18 to –8.0m) 26m at downstream i.e. 2.6 to 3.4 times the maximum width against 1.5 times width arrived by photo-elastic study. Physico-mechanical properties of the fault zone material and adjacent rocks (Sedimentaries and basalt) forming shoulders were considered in the analysis. Shoulder level for reckoning depth of the plug was kept at El. 18m (i.e. at general foundation level) in the analysis against at El. 1.5m (scoured level of the fault zone) earlier considered in the photo-elastic studies. It was found that excavation of trench was more than adequate. Results of the analysis also showed that stresses and factors of safety against sliding are well within the permissible limits.

(v) Treatment for settlement/ differential settlement: Based on two dimensional photo elastic studies depth of fault treatment plug was initially designed to be 1.5 times width of the fault zone but the actual treatment was carried out to a depth varying from 2.15 to 2.83m times the width depending on the site conditions and geotechnical judgment. Fault zone material was removed by excavating a trench down to El. (-) 16.5m in the upstream and to El. (-) 8 m in the downstream of the dam. Thus depth of the concrete (reinforced) plug provided was 34.5m (El.+18 to –16.5m) in the upstream and 26m(El.+18 to -8.0m) in the downstream (Mehta & Prakash 1990). In view of very poor quality of fault zone material (RMR 11) and poor quality of abutment rocks (RMR 11 to 33) this 34.5m deep concrete plug was reinforced. Construction of reinforced concrete plug was done monolithically below five spillway blocks 40 (block 40 forms one of the shoulder of the plug) to 44 upto El.18m and above it construction joints were provided for raising respective spillway blocks separately (Fig. 21).

(vi) Details of Hammock reinforcement: Hammock reinforcement was provided in the fault plug. It consisted of four layers of 36mm diameter high yield strength deformed steel bars, spaced 25cm parallel to dam axis in one direction and parallel to strike in other direction. This reinforcement 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, which may result in tension in plug. For mobilising greater shear resistance high yield strength deformed anchor bars 36mm diameter, 8m long in a grid of 3 X 3m were also provided (Fig. 21).

(vii) Grouting of the foundation below the plug: For consolidation grouting of foundation below the concrete plug 35mm diameter (dia.) 15m deep drill holes in a grid of 3m x 3m were provided. For curtain grouting two rows of 100mm dia. pipes at 1.5m spacing one row along the heel of the dam and the other along the drainage gallery were provided from the bottom of the concrete plug. Similarly drainage pipes of 100mm dia. at 3m spacing along the drainage gallery were provided. In order to ensure positive contact between the concrete plug and rock surface of the trench, grout buttons duly connected with a system of grout pipes were provided along the contact for carrying out grouting. An inspection-cum-drainage gallery of size 1.5 (W) X 2.30(H) m at El. 4.0m in the concrete plug with its alignment parallel to the strike direction of the fault zone was provided.

(viii) Seismic considerations: The area falls in seismic zone III of India (IS: 1893). Seismic event in the area can have two aspects viz. vibrations due to shock and physical relative movements along the fault plane. Earthquake studies carried out in the area have not given any indication of seismicity along the river channel fault. However, in view of the regional seismicity, effect of vibrations was considered in the design of the Narmada dam for a factor of 0.125g (horizontal seismic coefficient).

4.4.4 Karjan River Channel Fault

Karjan River at the dam site before construction of the dam was flowing along N-S trending fault in a 12-14m wide and 14m deep gorge (Plate 9). This fault is obliquely traversing the foundation of two dam blocks at an angle of 60° with the dam axis. Fault zone consisted of hard sheared, fresh basalt and associated with thin discontinuous gougey layer. Width of the River Channel fault (RCF) zone varied between 12 to 14m (i.e. covering entire width of the river gorge). Shears and fractures associated with the fault zone were mostly healed (HL1 to HL2) by veins of Zeolite and Calcite. Minor displacement, of the order of 0.50m of these veins was observed indicating slight movement in the area after healing of the fault zone (Fig. 4). Rock mass rating value of fault zone is 48 (Table 27). Rock mass adjacent to fault zone forming sides of the river gorge was hard and fresh except presence of a few weathered rock seams. Sides of the channel were smooth but marked with a number of potholes ranging from $0.6m^3$ to $6m^3$ size.

Table 27: Rock mass classification of the Karjan dam river channel fault zone

No.	Parameter description	Value	Rating
1	UCS	50-100MPa	7
2	RQD%	25-50	8
3	Spacing of discontinuities	<40-60mm	10
4	Condition of discontinuities	Separation<1mm, slightly rough surface	20
5	Ground water condition	Damp	10
6	Discontinuity orientation	Fair	-7

Total rating (RMR): 48
Class No.: III
Class description: Fair rock

Rock mass of the fault zone and adjacent rocks are fresh and most of fractures and shears associated with the fault zone are nearly healed (HL1 to HL2). It was considered that mostly healed fault zone (RMR 48) may not adversely affect the load bearing strength of the rock mass. Therefore, on the basis of engineering geological analysis and judgment no treatment for settlement/ differential settlement was provided to RCF though the width (i.e. 12m) of the fault zone was identical to Narmada main river channel fault. The dam is functioning from last 14 years without showing any distress. However, from structural consideration following remedial measures were adopted:

(i) Deep River channel (DRC) was plugged with concrete up to El. 36m i.e. up to general foundation level of spillway blocks. A mat containing nominal reinforcement extending 3m on either side of the block joint with distribution bars was provided on top of the plug. This was done to bridge the possible shrinkage gap between mass plug concrete and steep rock faces of DRC and thus to prevent the cracking of the overlying block concrete at the block joint.

(ii) Contact grouting of the concrete plug and the side rock was done to seal the shrinkage gap.
(iii) Additional drainage arrangement was provided through additional longitudinal and cross galleries to prevent uplift pressure.
(iv) Consolidation grouting through closely spaced (3m-interval) grout holes in the DRC area was done to seal remaining naturally unhealed fault zone material.

Seismic coefficient identical to Narmada Project i.e. 0.125g has been adopted in the design as Karjan dam is located in same seismo-tectonic environment. It was apprehended that gates of the spillway blocks directly located on the fault zone might get twisted in the event of earthquake shaking. Therefore, spillway blocks were shifted towards left side of the fault and Non-overflow blocks were located on the fault zone.

4.4.5 Treatment of minor faults and shears

Minor faults and shear in the foundations of Narmada and Karjan dams have been provided dental treatment. It involved selective removal of weak fault/shear zone material from the foundation and back filling the excavated trench with concrete (plug).

4.5 Weathered rocks/ zones/ seams in the foundation

Treatment of weathered rocks/ zones/ seams in the foundation of Narmada and Karjan dam was done as detailed below:

4.5.1 Narmada dam

Excavation of the foundation of dam blocks was completed upto general foundation level based on the sub-surface exploration prior to construction of the dam. During construction of the dam it was noticed that foundation of some of the dam blocks especially blocks-3, 15, 16 and 57 were occupied either by weathered rocks or weathered rock zones/seam or by both. Depth of weathering in these blocks was generally more than 5m from the excavated foundation levels. Removal of weathered rocks from the foundation of these blocks was not considered desirable in view of the already constructed adjacent dam blocks. Therefore, weathered rocks were retained and treated in the foundation of these blocks based on the rock mass characteristics (Table 28).

Table 28: Rock mass classification of the foundation of the Narmada dam block-3,15,16 and 57

Block No.	Flow/ Rock type and associated weak features	RMR	Class No.	Description	Remarks	Treatment
3	L-7 with weathered rock pockets	40	IV	Poor rock	Low RMR value due to weathered rock (W4-W5)	Reinforced concrete mat at toe of the block (Single layer)
15	L-7 with weathered rock pockets	40	IV	Poor rock	Low RMR value due to weathered rock(W4-W5)	-do-
16	L-7 with weathered rock pockets dissected by steeply dipping and sub-horizontal shear zones	25	IV	Poor rock	Low RMR value due to weathered rock(W4-W5) and shear zones (HL3-HL4)	Reinforced concrete mat covering major part of the foundation area as per topography (Two layers)
57	Agglomerate, R-5 and R-6a, dolerite dyke, shears and weathered rocks	35	IV		Low value due to weathered rock (W4-W5) and shear zones (HL3-HL4)	-do-

a. Main dam blocks: In-situ tests conducted on the foundation of block-16 on highly jointed weathered basalt (DS4, W3, RQD4, RMR-40) gave average value of deformation modulus of weathered rock as 0.04×10^5 kg/cm^2 and modulus of elasticity as 0.075×10^5 kg/cm^2. The ratio of two-modulii (1.87) also suggests weathered and jointed nature of the rock mass. Major part of the block-16 was lying over weathered rock zone having 5 to 6m highly jointed rock cover (Plate 17

(a) Removal of weathered rock mass in part foundation of the block-16

(b) Hammock reinforced concrete in the foundation of block-16 and sloping concrete in the foundation of block-17

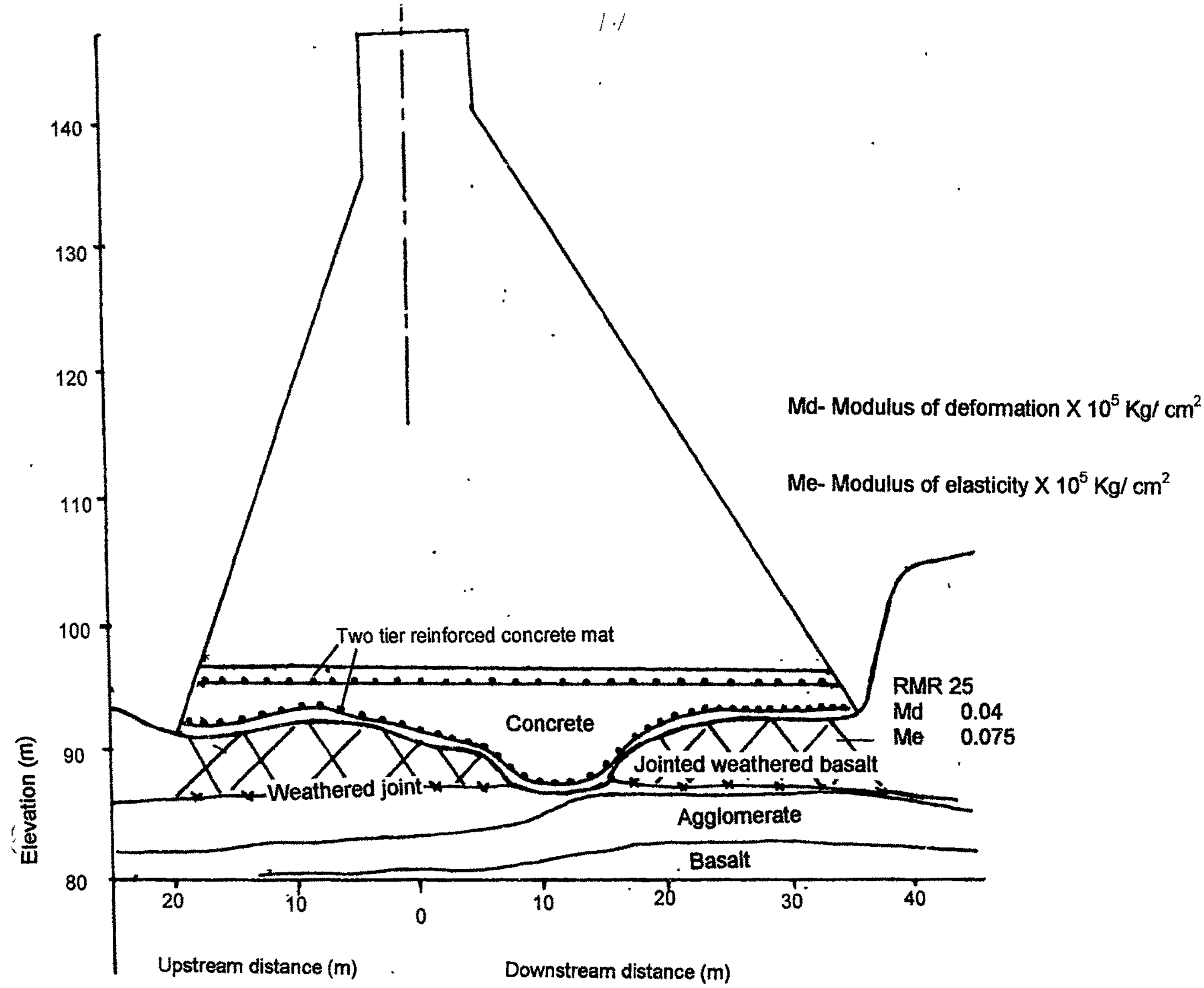
Plate 17: Foundation treatment of block-16 & 17, Sardar Sarovar (Narmada) Project

and Fig. 22). Stability analysis (FEM) indicated maximum 0.5cm settlement at the toe of the block with applied stresses, between 18 and 22 kg/cm^2. Corresponding tension observed near the upstream heel was 1.8 kg/cm^2. As a treatment concrete mat having two tiers reinforcement of 33mm diameter tor steel bars spaced at 0.6m c/c was provided in the entire foundation area of this block (Mehta and Prakash 1990) (Plate 17 and Fig. 22). Similarly, treatment in the foundations of other blocks (3, 15 and 57) was provided in the form of reinforced concrete mat in single or two layers depending on the nature of weathering and foundation topography to uniformly distribute the load and to prevent differential settlement.

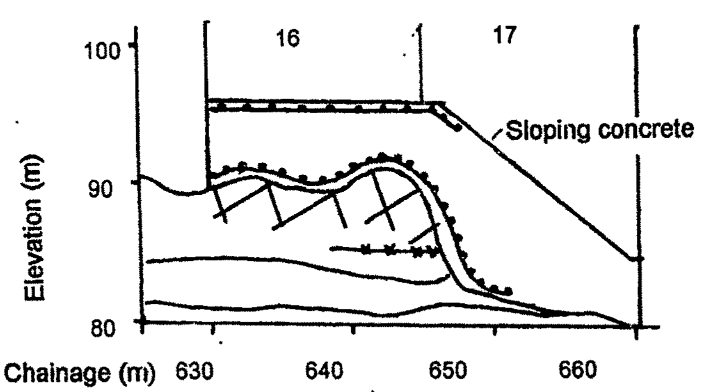
b. Left divide wall block-28: This wall is located between the lower chute and stilling basin. Basalt flows separated by red bole layer and intruded by dolerite rock form the foundation of the wall (Plate 18). Cladding section of the wall below the foundation of lower chute is weathered. Major part of the wall remained open between the period 1988 and 1996 resulting in the progressive scouring of rock face during successive floods. Maximum scouring of the order of 15m was observed laterally in the weathered rock area during September, 1994 flood when the flood water level was above the foundation level of the lower chute. The final scoured topography of the foundation was having trapezoidal section with narrow base and broad top. Longer anchors and reinforced coloumns and beams resting on good rocks were provided besides additional drainage to stabilise the wall (Fig. 23).
No such problem was observed in the foundation of right training wall where rock mass was fresh and construction was completed within two years of foundation excavation (Plate 4).

4.5.2 Karjan dam

Numerous weathered rock seams were encountered in the foundation of Karjan dam. It was not practicable to remove these seams from the foundation of blocks



I. Cross-section of block-16



II. Longitudinal section of block- 16 & 17

Fig. 22: Foundation treatment of weathered rock in the foundation of block-16 & 17

Plate 18: Disposition of red bole layer in the foundation of divide wall between Lower Chute and Stilling Basin, Sardar Sarovar (Narmada) Project

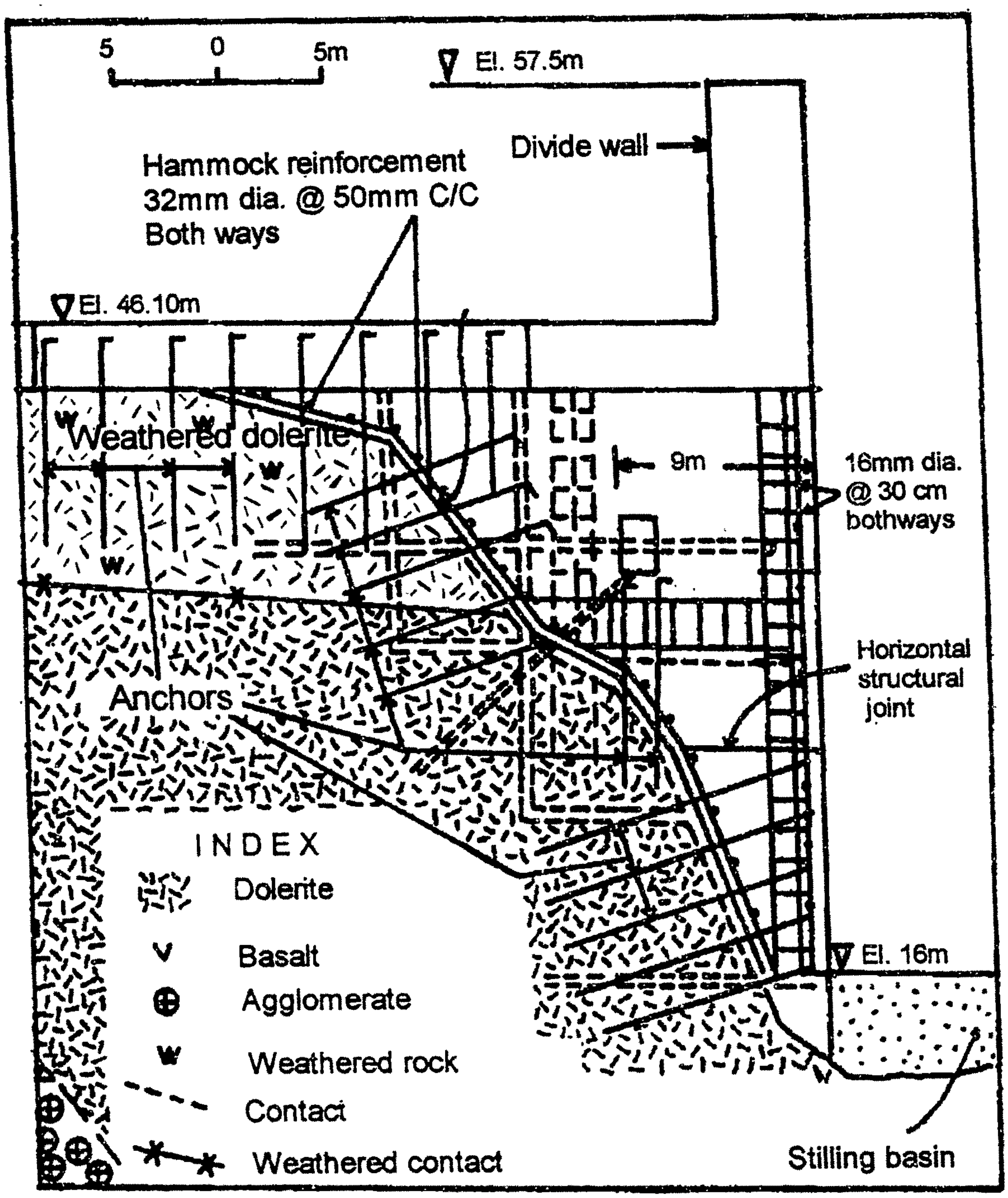


Fig. 23: Typical section of Narmada dam left divide wall block-28 showing foundation treatment of scoured rock

as they lie one over other at 3 to 5m intervals in the spillway blocks and at 4 to 10m intervals in the NOF blocks. These seams in general are of pinching and swelling type and of wavy disposition. Therefore, no treatment was provided to these seams from the settlement consideration as load of dam blocks would be transferred to underlying rocks at the points of rock to rock contacts of seam (Fig. 24). Moreover, these seams would be under confined condition due to construction of concrete shear keys. Treatment was also not provided even for 2m thick seam in the foundation of LNOF block-L3 on the consideration that dam was to be constructed slowly (i.e. slow loading of the foundation) and whatever settlement would have to take place would be completed during construction of the dam itself (Fig. 25). This seam is having shallow (3 to 4m) jointed (BS2-BS3) rock cover (Prakash and Mehta 1990).

No settlement problem has been observed in the foundations of Karjan dam blocks even after fourteen years of completion/ operation of the dam.

4.6 Seepage

There is always permanent movement or normal seepage of the water after the construction of the dam from the reservoir under and around the dam and at the rims of the reservoir. If this movement or escape of water from the reservoir through the fissures and openings in the rock, buried channels etc. becomes abnormally large it is known as leakage (flow >125 Ltr./ minute/ 10m length as per Bieniawiski's (1976) classification). Fissures characteristics of the rocks at Narmada and Karjan dam sites were investigated through pressure testing in drill holes and also by direct observations of foundation treatment shafts and drifts during their excavation. Water pressure tests in the basalt were misleading as adjacent holes often gave entirely different permeability values. Natural lava tunnels, caverns etc. were not found present in the area.

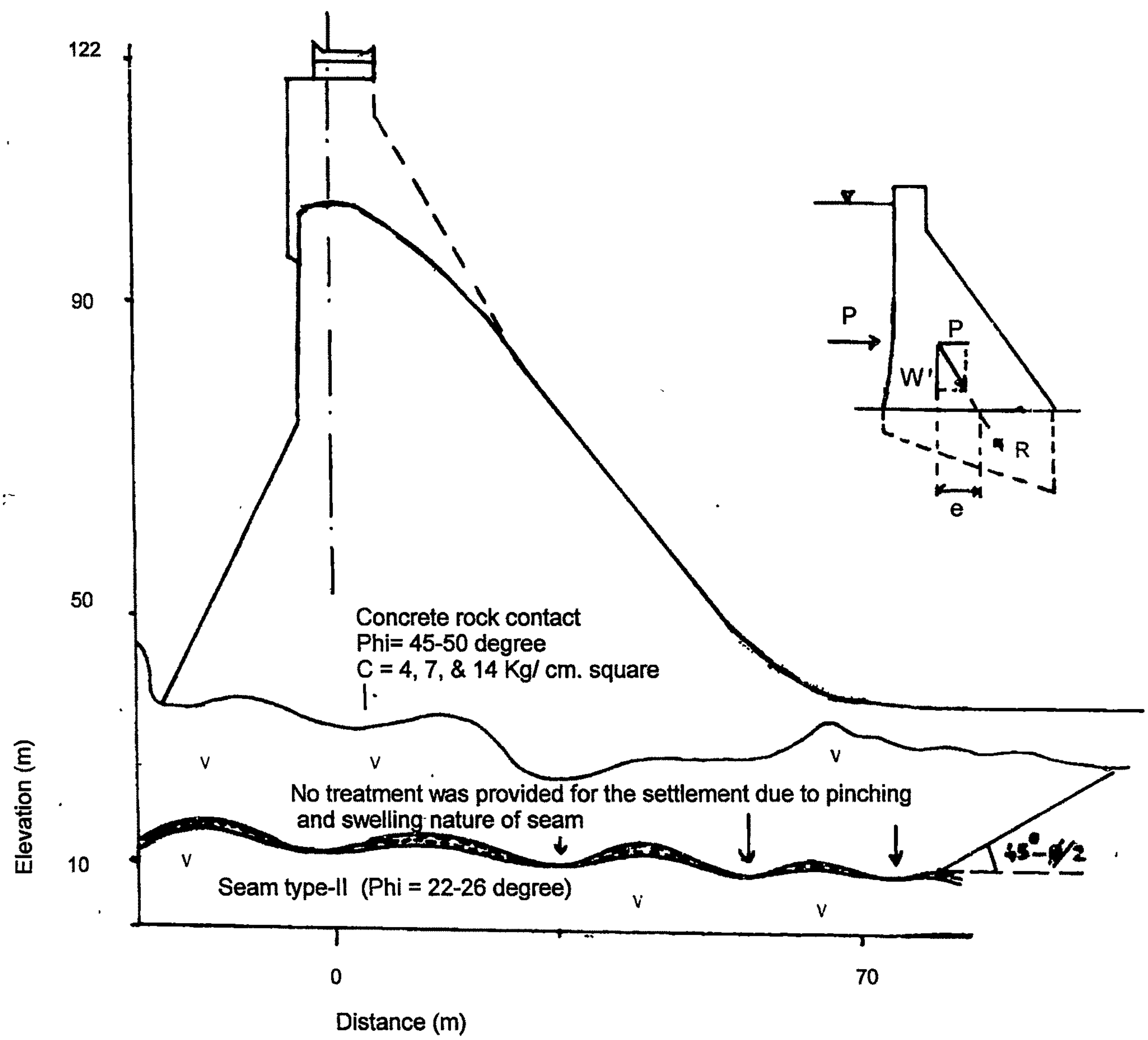


Fig. 24: Geological section of Karjan dam spillway block showing disposition of pinching and swelling type weathered rock seam

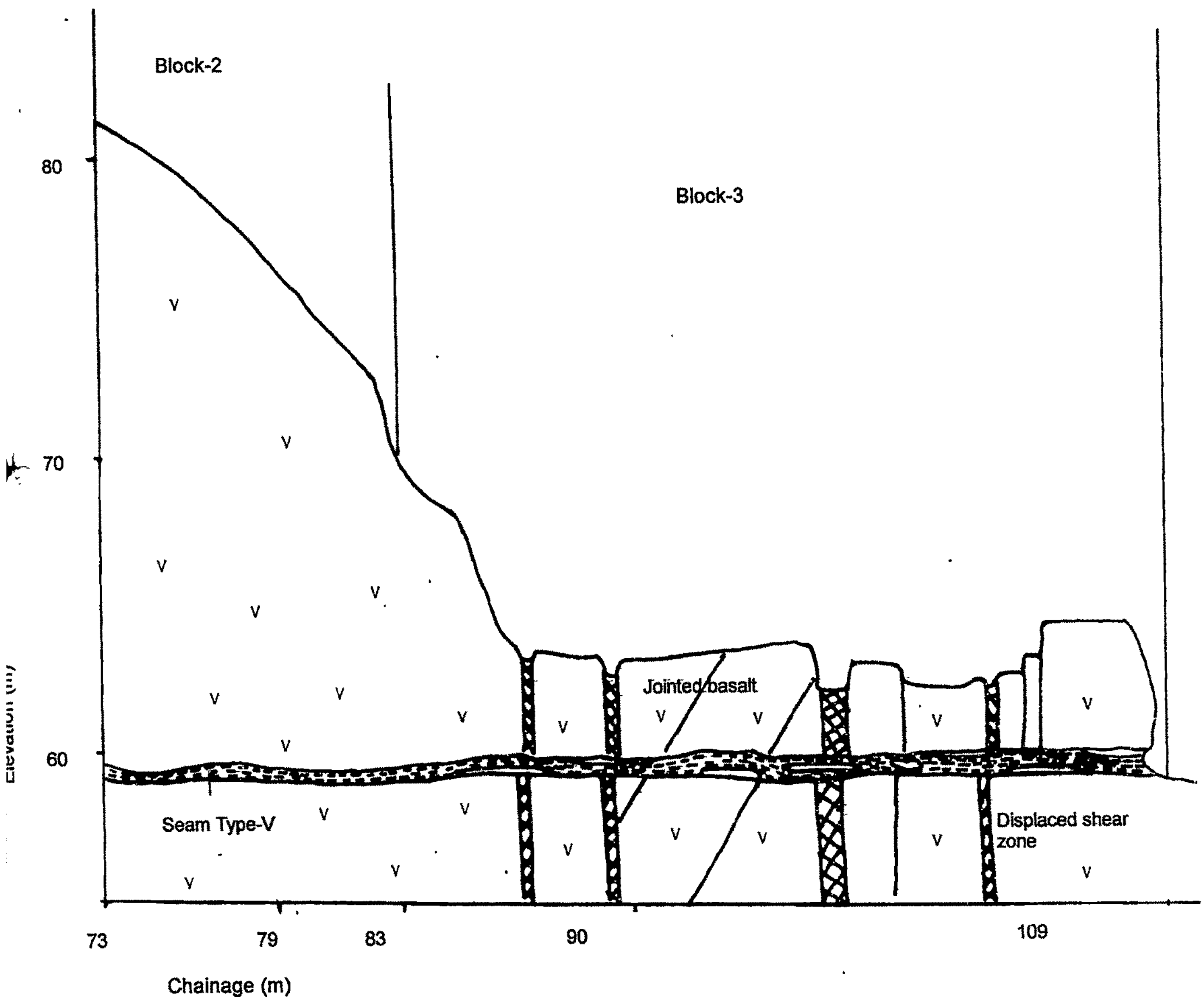


Fig. 25: Longitudinal geological section of Left Non Over Flow block showing disposition of weathered rock seam

4.6.1 Narmada dam

In general appreciable seepage was not observed in the foundation through basalt flows except at few locations where flow contacts were sheared and weathered (Plate 19). Seepage was observed through the contacts of infratrappean sedimentary rocks during the excavation of treatment shafts and drifts. These rocks are present in the foundation of Right Bank blocks at shallow depth. The limestone (30 to 60m thick) occurs in the foundation of spillway blocks 44 to 59 at about 40m depth (Fig. 26).

a. Seepage through limestone: It was apprehended that limestone occurring in the reservoir as well as in the foundation of dam blocks might lead to the leakage of reservoir. Therefore, extensive investigations were carried out to know the nature of the limestone including drilling of 14 exploratory drill holes down to depths ranging from 70 to 105m (El.15 to –95m). Chemical analysis of limestone showed that it is of siliceous nature (average SiO_2 20%) and thus there was no possibility of occurrence of chemical cavity. Cavities were also not detected during core drilling. However, drilling investigations of limestone suggested poor core recovery, high permeability and heavy water losses at few locations requiring tracer studies to ascertain the seepage path and to establish inter-connectivity of holes.

Entire limestone section was divided into three parts viz. upstream of the dam (Upstream of the Mokhadi fault), below the dam seat and downstream of the dam (225m downstream of dam toe) for tracer studies. Following are the results of tracer studies carried out for the limestone:

(i) Sodium Fluroscein and Tritium were injected in the drill holes located in the upstream of Mokhadi fault. Even after 15 days tracers were not observed in the boreholes located in the downstream of the fault.

(ii) Sodium Fluroscein was injected in the hole located 5m upstream of the dam axis (Fig. 26). Average velocity of 20 to 30m per day was observed on the basis

Plate 19: Seepage along flow contact, Sardar Sarovar (Narmada) Project

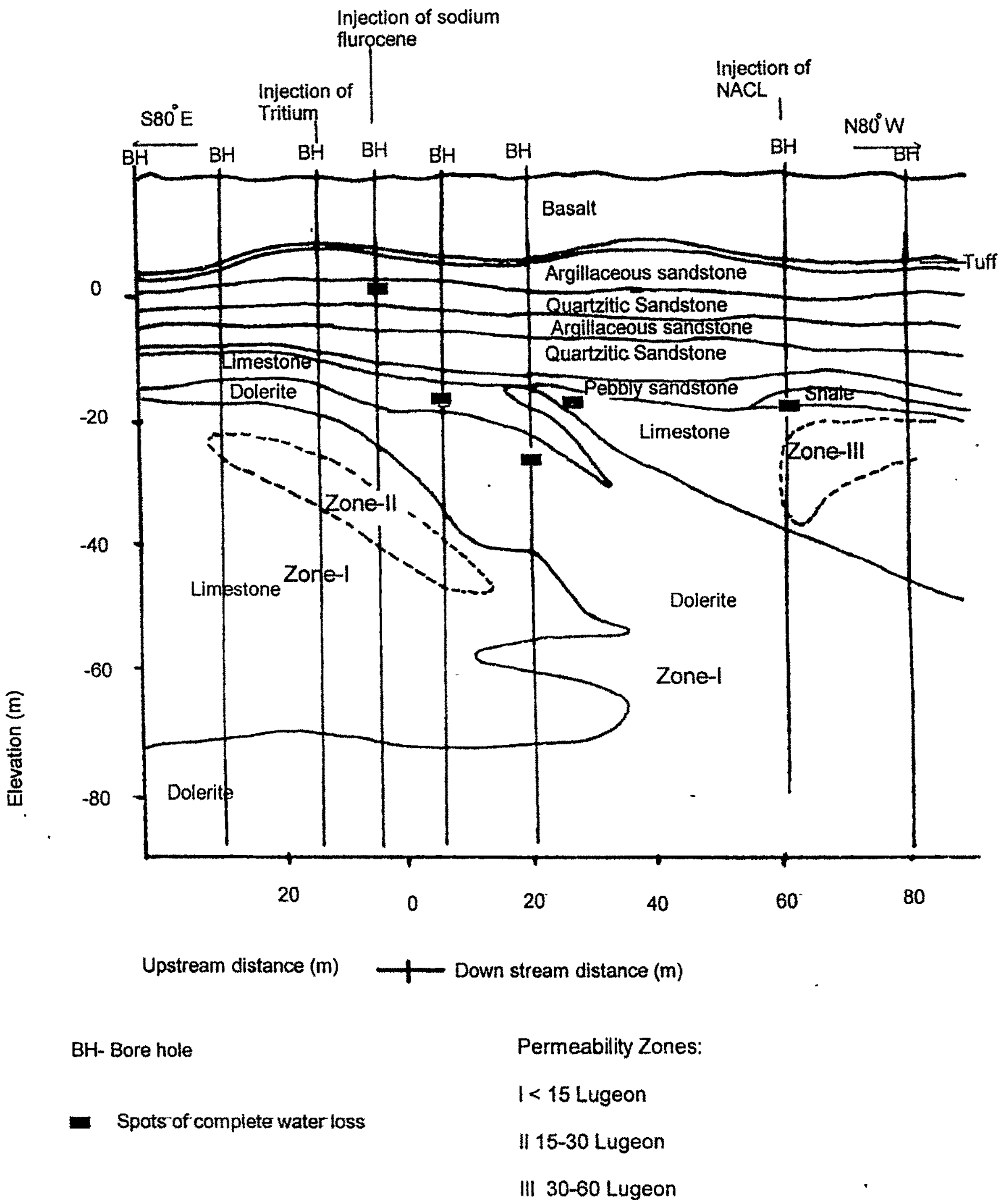


Fig. 26: Geological section (at Chainage 1275m) of Narmada dam foundation showing tracer study holes and permeability results of limestone rock

of observation in two bore holes located at 20m and 80m downstream, respectively. In another hole at 30m upstream of dam axis sodium chloride was used. No tracer was noticed even after 5 days in the observation holes located 5 and 60m away. Similarly, Tritanium was used in other holes but no tracer of Tritanium could be traced even after 8 days.

(iii) In view of high velocity observed in one of the section further Fluroscein studies were done in the area. Out of three observational bore hole, porous velocity 2.16x 10^{-3} cm/ sec was observed in one of the hole. It might be due to presence of local permeable pocket.

(iv) Sodium Fluroscein was injected in a hole located at 225m downstream of the dam and observations were made in the drill holes located at 15, 18, and 120m away but no tracer was observed in the observation holes.

(v) Neither cavity in the limestone nor interconnection between test holes was established by tracer studies. It might be due to breaking in the continuity of permeable zones by faulting and intrusion of basic dykes acting as seepage barriers.

High permeability observed in few drill holes appears be due to presence of local permeable pockets. These permeable pockets/zones were treated by increasing depth of curtain grouting down to depth of entire section of limestone below dam base.

b. Protection against piping of fault zone material: Fault zone material is highly sheared basalt rock with crushed and brecciated rock pieces interspersed with 5 to 15cm clay gouge. Adequate length of path of seepage under the dam required for providing treatment against piping for such type of fault zone would be 2.5H (Height of the water coloumn). Normally, cut off shafts on upstream and downstream of plugs are required from hydraulic considerations. It was felt that these shafts are not necessary in this case, as required path of percolation of 2.5H would be available under the dam and stilling basin. As per the design requirement trench on upstream was back filled with concrete for a distance of 10m and by impervious soil in the remaining distance. In the downstream, a 4m deep trench was excavated along fault zone and back filled in the foundation of entire stilling basin.

4.6.2 Karjan dam

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 foundation rock mass, initial curtain grouting was done in four stages with 5,10,15 and 20kg/cm^2 pressures, gradually increasing with depth when the reservoir level was at minimum. Depth of curtain grouting in spillway section varied from 42 to 60m. It was observed that in five spillway blocks post-grouting seepage was more than 100 litres/minute. It clearly indicated ineffectiveness of initial curtain and consolidation grouting. Therefore, to reduce the seepage and to seal remaining gaps/permeable windows in the grout curtain additional curtain grouting was done with uniform high pressure of 20kg/cm^2 in all the stages, after filling of the reservoir upto El.78m. Seepage was reduced in general by about 70 to 90% after providing additional curtain grouting (Prakash and Srikarni 1998) (Fig. 27).

4.7 Seismoteconic studies and problem of seismicity

The Narmada and Karjan dams are located in the Son-Narmada-Tapi Rift Zone, which is considered to be seismically active. The project area falls in the seismic zoning map of India (IS: 1893).

4.7.1 Micro-seismicity

Micro-seismic survey carried out by Central Water and Power Research Station, Pune, and School of Earthquake Engineering, University of Roorkee in the area

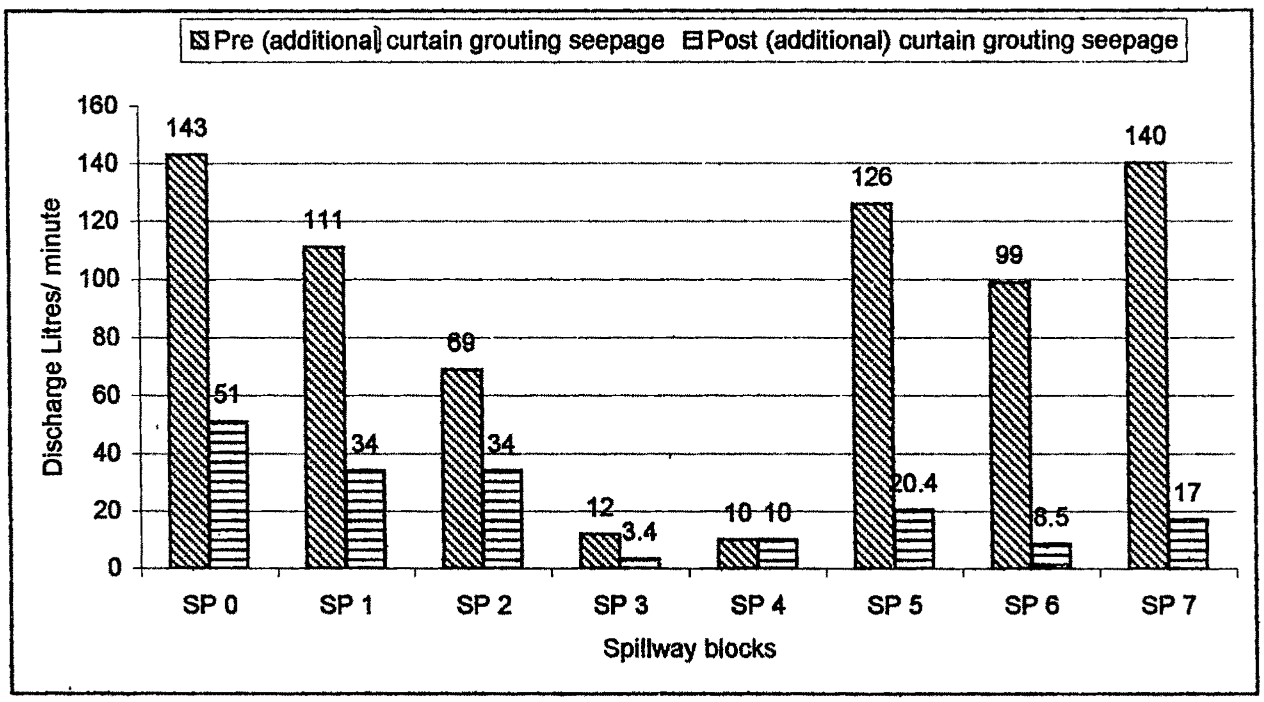


Fig. 27: Post grouting seepage and additional curtain grouting of the foundation of spillway blocks, Karjan dam

in 1970-75 and 1980 respectively showed evidences of sub-zero and very low magnitude micro earthquakes activity with very shallow depth of foci (upto 5km) and epicenters randomly distributed around Narmada dam site. No seismic activity was recorded along the river bed fault crossing the dam site (Krishnan 1976, Mehta & Prakash 1982) (Appendix-V).

4.7.2 Consideration of seismotectonics in the aseismic design of dam

The data on earthquake occurrence in Peninsular India show that the Maximum Credible Earthquake (MCE) in this area can have a maximum magnitude of 6.5 and the aseismic design of the dam is based on this. As per the published statistics, idealised length of fault rupture to cause an event of magnitude 6.5 has been estimated to be more than 15 km (i.e. about 20-25km) and the vertical dimensions of the slip area to be about 20km. Based on the experience of 1967 Koyna event focal depth of 10 km has been assumed for any major seismic event that may occur in the vicinity of the Dam. The Piplod fault, which is a major closest fault (i.e. at 12km shortest distance) to the Dam site, has been assumed as causative fault for the aseismic design of the dam (Krishnan 1976) (Plate 1 & 20 and Fig. 2). The horizontal seismic coefficient 0.125g was considered for the static analysis and 0.25g EPGA (Effective Peak Ground Acceleration i.e. mean-plus-one-standard deviation of the peak values of Peak Ground Acceleration), for the dynamic analysis in the final design of the dam. Accordingly, the horizontal seismic coefficient adopted for the Narmada project is 0.125g. Same seismic coefficient has been adopted for the Karjan dam located about 25km downstream in the same seismotectonic province.

4.7.3 Monitoring of the project sites

Project area is monitored by a network of nine seismological observatories with the following objectives:

(a) Straight course of Tarav Nala near Piplod

(b) Near vertical disposition of infra-trappean sedimentaries (Bagh beds) adjacent to fault

Plate 20: Surface manifestation of the Piplod fault (a & b)

(i) To provide background information about seismic activity in the area before and during the impoundment.

(ii) To assess the probable source(s) of seismic activity in the vicinity of the Dam.

(iii) To monitor the seismic activity around the reservoir.

(iv) To evaluate seismic activity during the filling of the reservoir.

(v) To provide real time detailed information on the intensity and frequency characteristics of ground motions due to earthquake(s) in the region.

(vi) To precisely determine the epicentral location and focal depth within 200 km radius for any earthquake of engineering significance.

The Narmada dam was partly constructed upto El.85m till October 2000. Maximum depth of water in the reservoir was 65m. Seismic events of magnitude below 3 are generally occurring in the area except a few events between magnitude 4 and 4.5. Seismo-tectonic study revealed that about 15% of the epicenters of all earthquakes fall on the northern side of Narmada River and the rest 85% on the southern side of the river mainly along and adjacent to Piplod fault which has already been considered in the aseismic design. No activity along river channel fault, located at the dam base, has been observed prior and during the present stage of construction of the dam i.e. after partial filling of the reservoir. No adverse effect of recent Bhuj (Kachchh) earthquake of 26 January 2001 of magnitude 6.9 (as per Indian Meteorological Department) has been noticed on Narmada and karjan dams.