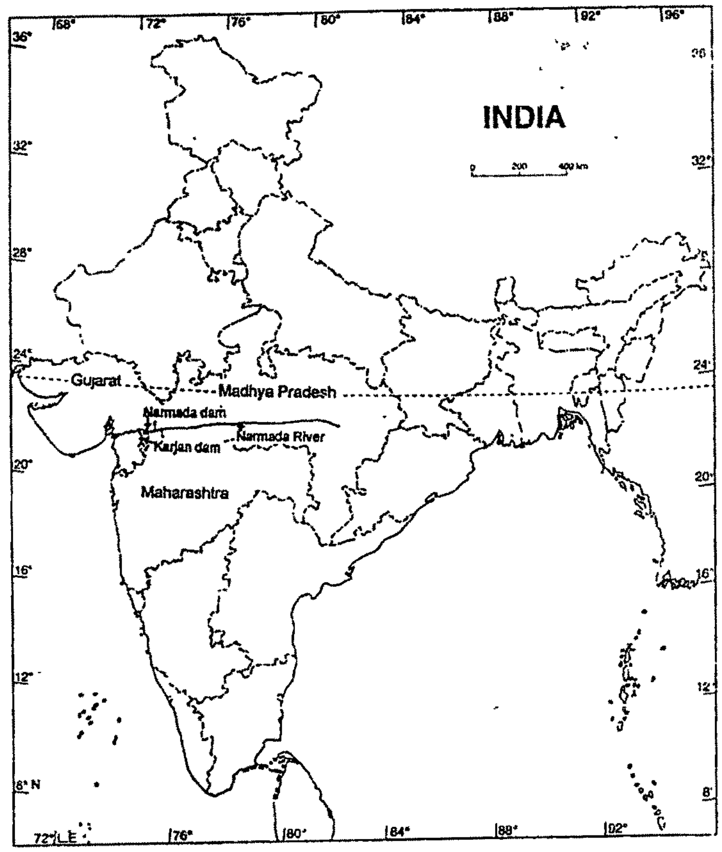
ENGINEERING GEOLOGICAL EVALUATION OF THE MAJOR DAMS IN LOWER NARMADA VALLEY IN GUJARAT STATE

1. Introduction

1.1 Background information

The Narmada is the longest west flowing river in India. It is the fifth largest river in the country and the largest one in Gujarat. It originates at Amarkantak in Madhya Pradesh (M.P.). It enters the Gulf of Cambay in the Arabian Sea. The total length of the river from source to sea is about 1312km. Physiographicaly the Narmada valley is divided into three zones namely "Upper zone" between Bilaspur and Mandla in Madhya Pradesh (M.P.), Middle zone" between Mandla and East Nimar in M.P. and "Lower zone or Lower Narmada Valley" between East Nimar (M.P.) and Bharuch (Gujarat). In the Lower Narmada Valley the Narmada River flows in gorge mainly through Deccan trap (Basalt) and infra-trappean (sedimentaries) in straight course upto Gora (about 6 km downstream of Narmada dam). The area exhibits faulted block topography with alternate ridges and valleys aligned almost in ENE-WSW direction.

The Narmada River has maximum-recorded flood discharge of 69,375 cumecs and minimum being 8.5 cumecs. In order to harness the vast irrigation and hydro-electrical potential of Narmada River number of dams are being constructed across Main River and its tributaries. In the Madhya Pradesh part of the Lower Narmada Valley some of the major dams under construction are Narmada Sagar, Omkareshwar and Maheshwar. The Sardar Sarovar (Narmada) dam located across main Narmada River is a terminal reservoir under construction in the Gujarat State (Fig. 1 & 2 and Plate 1 & 2). The Karjan River is a S-N flowing major tributary of Narmada River in Gujarat. The Karjan dam has been constructed across this tributary about 25 km downstream of the Sardar Sarovar dam (Plate 1 & 3 and Fig. 1 & 2).



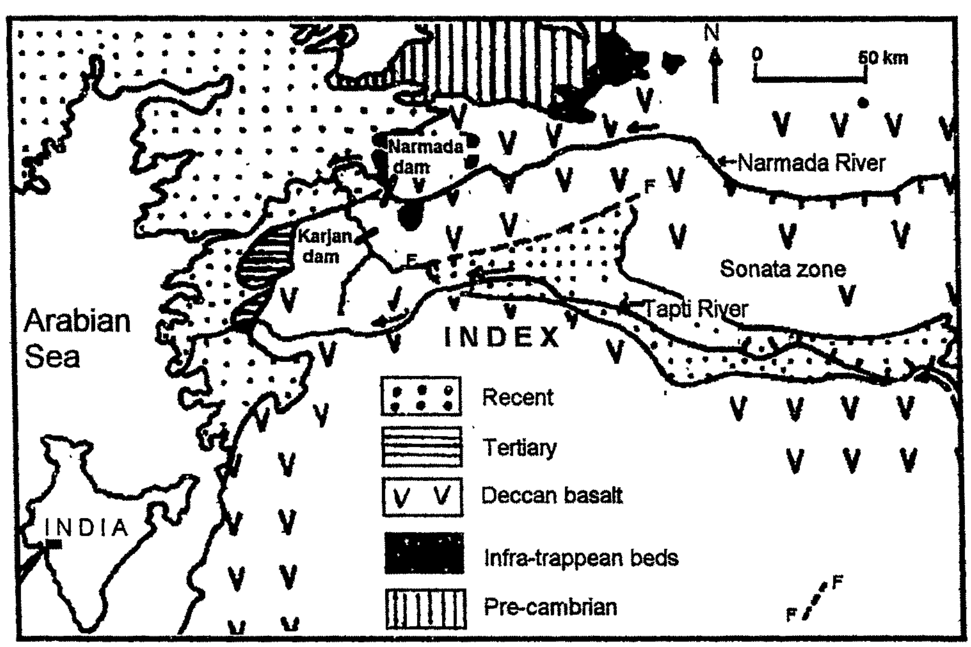


Fig. 1: Location plan of Sardar Sarovar (Narmada) and Karjan dams

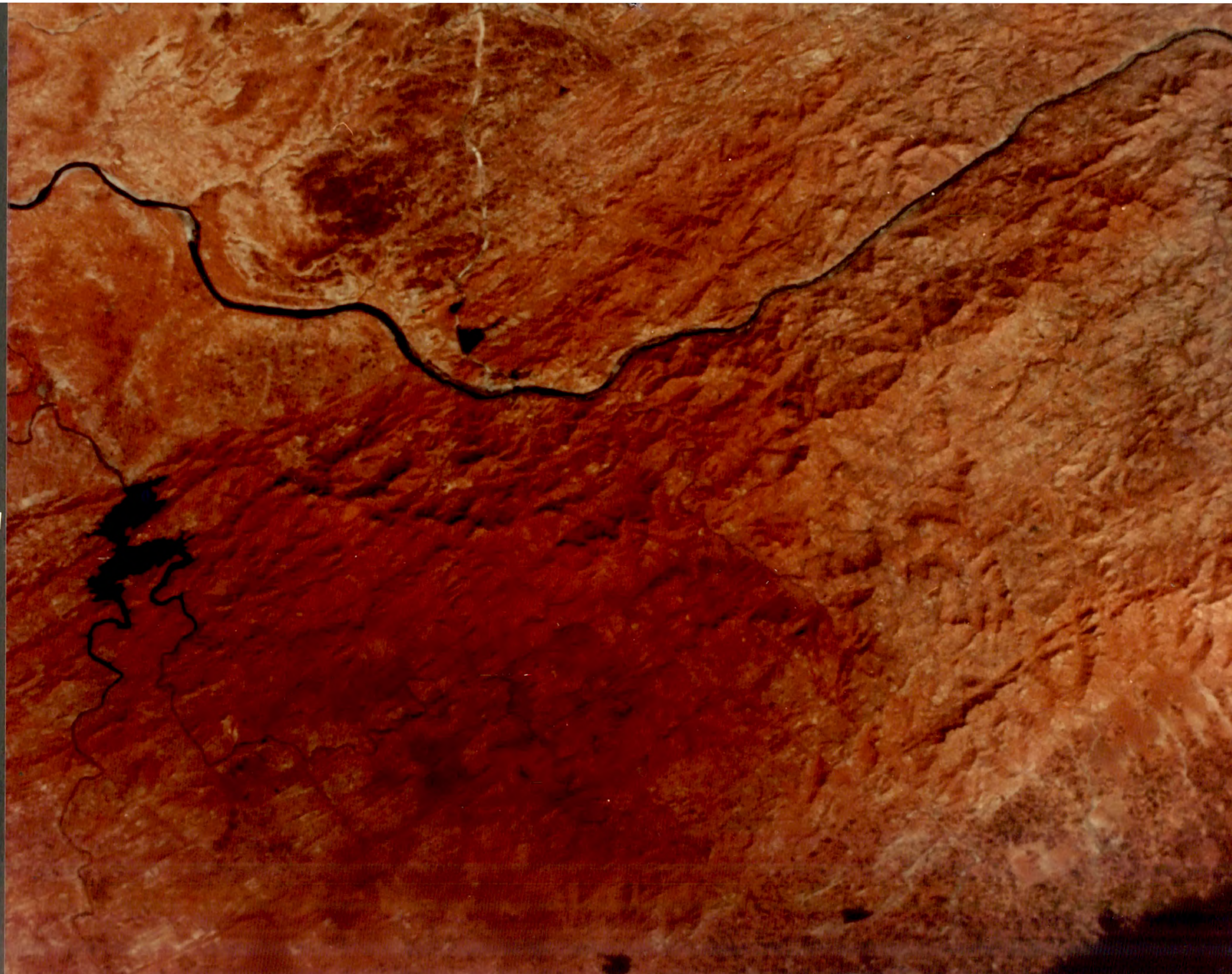


Plate 1: Satellite imagery showing location of Narmada and Karjan dam sites and Piplod fault

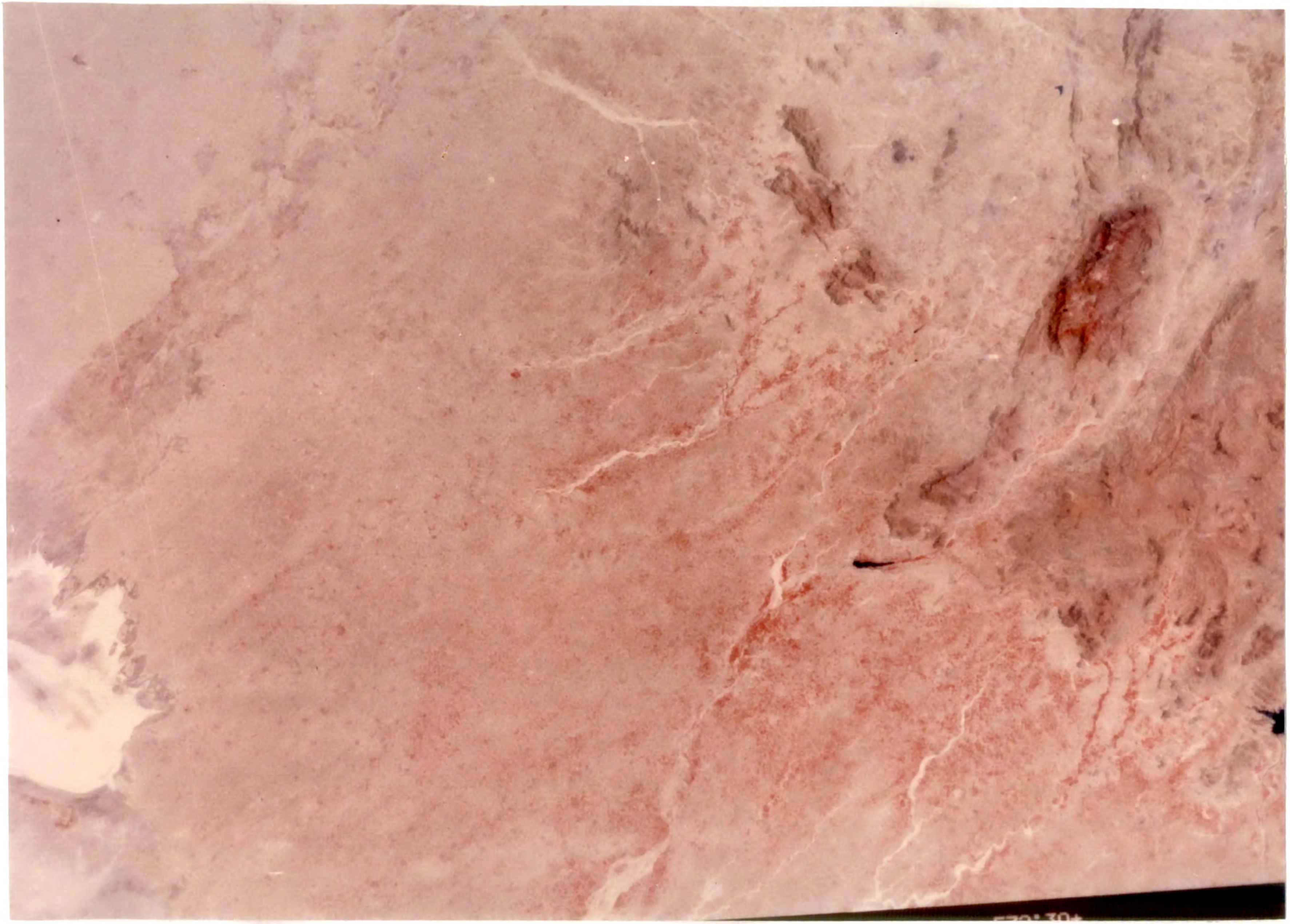


PLATE [illegible]-2 SATELLITE VIEW OF THE NORTHERN EXTENTION OF BANASKANTHA SHOWING TERRAIN FEATURES

(a) Excavated foundation of dam

(b) Partly constructed dam

Plate 2: Site condition prior and after the construction of the dam (view from left bank), Sardar Sarovar (Narmada) Project (a & b)

(a) Basalt flows mainly covered by dumped material

(b) Partly constructed stilling basin with end sill

Plate 3: Site condition prior and during the construction of stilling basin, Karjan dam (a & b)

The Narmada Valley is geologically complex and structurally disturbed (Fig. 3). The ideal sites for major projects are now almost exhausted. Safety viability and cost of the dam are dependent upon geology. The main purpose of engineering geological evaluation of major structures is to identify weak geological features and to reduce the factor of ignorance in the design for safe and economic construction. This is possible only with improved intensive geotechnical investigations. Two large major dams located in the Lower Narmada Valley in Gujarat State namely **Sardar Sarovar (Narmada) and Karjan dams and Narmada underground powerhouse have been selected for the present study** (Fig.1). The knowledge obtained by synthesis of the available information of other projects located in India and abroad has been utilised in the foundation evaluation and treatment of these two dams and underground structures. Thus, experience gained at these two project sites would also be helpful in further advancement of the knowledge of engineering geological investigations to be carried out for similar projects in future.

1.2 Purpose and scope of the study

The studies were carried out with the following main objectives:

(i) To study in detail surface and sub-surface geology of the project sites

(ii) To identify and delineate geological features like shears, faults, joints, lithological contacts and litho-units, weathered rock seams/ zones.

(iii) To study and evaluate physico-mechanical properties of rock mass and individual litho-units, litho-contacts, joints, shears and faults.

(iv) To study in detail influence of geological features in the stability of structures.

(v) To obtain sufficient knowledge and understanding of geological conditions at Narmada and Karjan project sites for ensuring economic design, construction and subsequently operation of these projects as well as other similar structures likely to be taken up in the future.

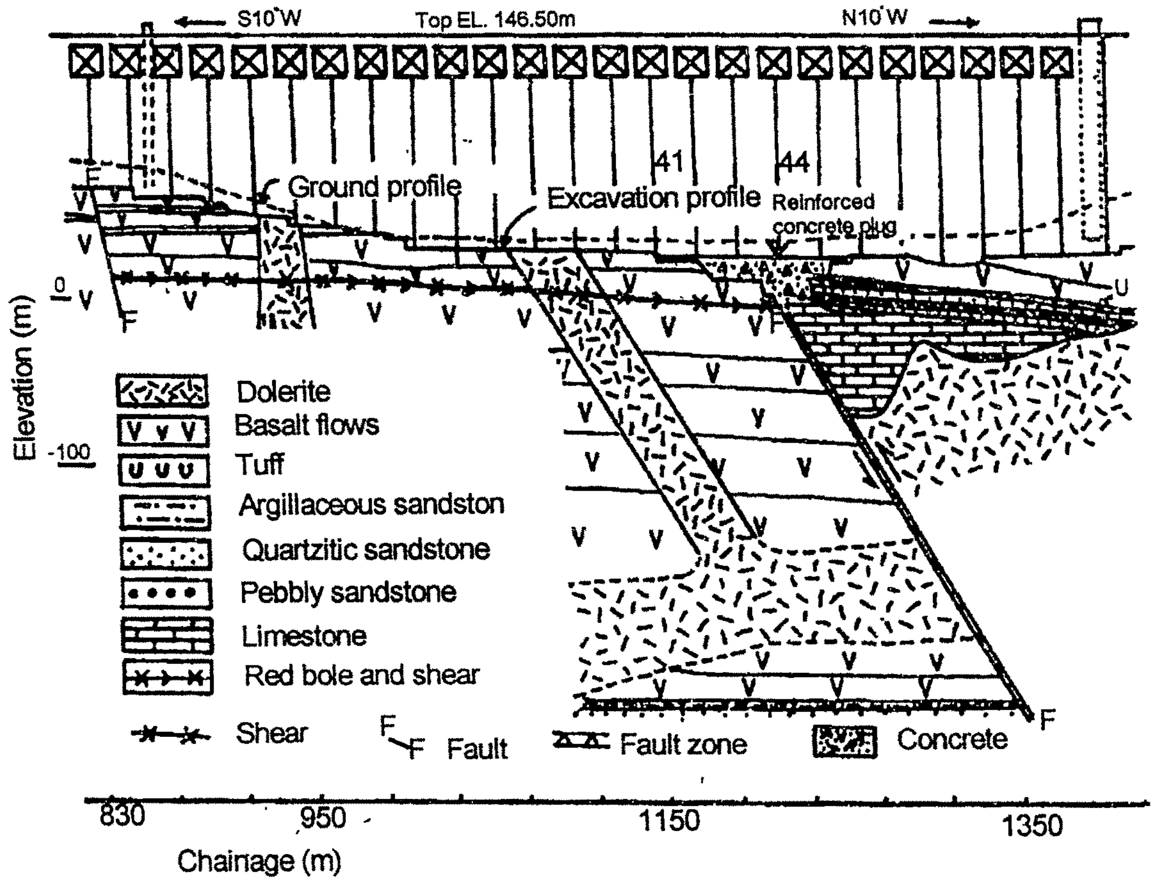


Fig. 3: Geological longitudinal section of Narmada spillway blocks

1.3 Methodology

Adequate knowledge of ground conditions and reaction of ground to civil engineering structures is essential for the successful investigation. There are many techniques of investigation, each more appropriate to some situations than others. The choice of methods for a particular investigation depends upon the geological circumstances and the nature of the engineering work, and limitations of the site investigation techniques. The methodology adopted for the present study is described below:

1.3.1 Study of literature

The available geological, seismological and engineering geological information was collected from the published and unpublished reports, records and papers of various Central and State Government Department / Institutes / Universities. An important part of general study is to recognize those natural and man-made hazards, which may affect the site. In this study data is compiled and studied in order to recognize potential problems, coming either from geological or other circumstances, or from lack of available knowledge.

Information regarding geological and engineering geological investigations was collected from the reports of Geological Survey of India (GSI) and Irrigation Department (ID), Government of Gujarat. Seismological and seismotectonics data was obtained from the Roorkee University, Roorkee, ID, Gujarat and GSI. Central Water and Power Research Station (CWPRS), Poone, Central Mining and Research Station (CMRS), Dhanabad and Gujarat Engineering Research Institute (GERI), Vadodara conducted Laboratory and in-situ tests on the rocks and rock mass at Narmada project and GERI at Karjan project. Laboratory and in-situ test results were studied in detail. Instrumentation reports of the CMRS and Central Soil and Material Research Station, New Delhi were consulted for evaluating underground powerhouse performance. Information regarding in-situ stresses in and around underground powerhouse was obtained from the reports of GERI and National Geophysical Research Institute (NGRI). Reports of the National Institute of Rock Mechanics (NIRM), Kolar on 3-D model studies of the underground powerhouse were referred for the evaluation of behaviour of rock mass during construction. Reports of Hydraulic model tests carried out for various types of energy dissipater arrangement by CW&PRS and GERI were referred to know scour depths and location.

1.3.2 Geological Study

The geological study included surface and sub-surface examination and mapping of rock-types and discontinuities including joints, shears and faults. Satellite imageries and aerial photographs were used for the identification and delineation of major lineaments. Regional geological mapping was done on scale 1:63,360 and 1:50,000 and detailed geological mapping of dam sites area was done on scale 1:1500 and 1:500. Geological features of finally excavated dam foundations and its ancillary structures and underground structures were recorded on scale 1:100 (covering about 10,00,000m^2 area). Selected geological features were also recorded on scale 1:50. Sub-surface geology was established based on the logging of test pits, trenches, 90 cm Diameter calyx holes, rock cores (about 30km running length), shafts, drifts, tunnels and underground chambers/cavern. Geological investigations were carried out at site in different stages closely linked with the progress of engineering design and construction.

1.3.3 Seismotectonic studies

The earth's crust is in a state of stress. Earthquakes occur on the surface of the earth due to sudden release of stresses. The energy released in an adjustment of stresses in the earth crust causes propagation of shock waves of varying wavelengths and frequencies from the epicenter. The short period waves can produce resonance in the dams because frequencies of these waves are mostly within the range of natural frequencies of large dams. The short period waves attenuate more rapidly within 80 to 120km distances from the focus (Thomas 1979). Therefore, seismic events occurring within 120km radius of dams in conjunction with faults and lineaments are generally considered in the seismotectonic studies.

1.3.4 Engineering Geological Study

The knowledge about the rock mass and its reaction to the superimposed civil engineering structures is obtained by the engineering geological studies. Local geological conditions vary from place to place requiring identification, delineation and evaluation of the geological features at each site considering types of the structure to be constructed in the area. The techniques adopted for the engineering geological studies are described as below:

a. Geological mapping: Geological mapping was undertaken in order to understand the geological character, history, and structure of the area. These maps show litho-stratigraphical units and indicate the main structural features of the area, such as major faults, shears and folds. Geological maps (Topo sheet Nos. 46G/9 and 11) formed the base for the Engineering Geological study as they show composition and structure of the area. Site specific engineering geological maps were built upon geological maps.

b. Engineering geological mapping: Base of the engineering geological map is geological map. Engineering geological mapping was undertaken in the area with the objective to characterise and classify geological materials based on engineering properties. Different scale maps were prepared during different stages of investigations (Table 1). Ground water conditions were also recorded as water pressure plays important role within the ground and thus in turn on the structure.

Table 1: List of Engineering geological maps used for the different purpose

Scale of map	Examples of uses	Types of Information shown
1:15,000 TO 1: 63,360	Planning of dams and powerhouses sites. These maps formed basis for reconnatory site investigations.	General zoning of litho-units, main structural geological trends, major faults and shears. General drainage characteristics.
1:1000 to 1: 15,000	Maps were used for the selection (alternatives) of engineering locations for dams, powerhouses and its ancillary structures. They formed basis for the planning of a detailed program of site investigations.	Zoning of litho-units in qualitative engineering units based on engineering geological properties. Mapping of discontinuities including major joints. Details of drainage system, flooding areas, saturated zones, springs, seepage and infiltration
1: 100 to 1.1000	Maps were used for detailed study of foundation conditions, remedial measures for unfavourable geological features and for stability considerations of surface and underground excavations. These maps formed the basis for the design of the engineering structures.	Litho-units were described in terms of rock mass strength e.g. combinations of material strength, jointing intensity and nature and depth of weathering. Additional information on depth of ground water table/ levels and permeability of ground.
1:50 to 1:100	Maps used for storage of data obtained during foundation excavations of dam and its ancillary structures, tunneling and underground powerhouse excavations. This data was analysed and compared with conditions as predicted from previous investigation phases and accordingly treatment of weak features and construction programs were modified wherever found necessary. Monitoring of the behaviour of the geological environment during construction and subsequently operation of the engineering structures was done.	Additional site specific geological and hydrological information during and after construction of the structures was obtained.

c. Remote sensing techniques: Remote sensing techniques like aerial photographs (scale 1: 25,000 and 1: 50,000) and satellite imageries (IRS-IA) were used in the identification and delineation of major geological features and assessment of ground conditions in and around project sites.

d. Boreholes, excavations and sampling: Sampling of the ground was done to know the nature and engineering properties of rocks and rock mass. Disturbed and undisturbed samples were collected, studied and analysed. Access to sampling locations was gained by trial excavations, shafts, and drifts and by boreholes. Rock samples for detailed examination and testing were recovered by single, double and triple core drilling methods. Types of core barrel used were mostly NX (Core Diameter (CD) 54mm). Other types of core barrel used were BX (CD 41.3mm) and AX (CD 32.2mm).

e. Laboratory and field testing of foundation media: Testing of foundation media both in the field and on samples of materials in the laboratory is done to determine various characteristics of the foundation media. The characteristic of the weaker portion of the rock mass is essential as it may guide and control the overall behaviour of the rock mass. It is like the proverb that the strength of the chain lies in the strength of its weakest link.

Some workers considered laboratory methods of testing as more accurate, cheaper, easier to carryout in comparison to field tests which are limited to the particular set of circumstances prevailing at the time of the test (Farmer 1968). However, rock properties can change over a small area as rock mass is heterogeneous and a pronounced joint or fault system (discontinuities) in a large project may affect rock reactions in a way, which could never be estimated by laboratory tests. Discontinuities are generally absent in laboratory test samples. Therefore, in design of structure founded on rock or in rock, it is necessary to understand that rock or rock mass has definite limitations when used as an engineering material.

f. Laboratory testing: Laboratory testing was undertaken to determine the engineering properties of the ground material and discontinuities recovered by sampling for the description and classification, to know deformation characteristics and for the suitability of rocks as construction material (Table 2).

g. In-situ testing: Rock mass is heterogeneous because of presence of discontinuities. Such features are generally absent in laboratory samples. Hence to obtain realistic design parameters in-situ tests were conducted (Table 3). Physico-mechanical properties obtained from the laboratory and field tests were applied within as well as outside boundaries of geological units for the quantitative assessment of rock mass.

Table 2: List of laboratory tests conducted as per IAEG Commission (Price 1981) on ground (rock and soil) material and discontinuities

Test/ Technique	Principles of technique	Remarks
Density	The weight and volume of samples or rock are determined and the density (specific gravity) is calculated. The density may be determined as "wet" (at natural moisture content or "dry" after oven drying).	The true specific gravity is larger than bulk (apparent) specific gravity. In rock of low porosity this difference is negligible as weight of water filling the pores would also be negligible.
Natural moisture content	By weighing samples before and after drying their natural moisture content is assessed and expressed as a percentage of the weight of solid material	
Porosity	By measuring the volume of the sample and the saturated moisture content the porosity may be expressed as the volume of voids as percentage of the total volume	The test method assumes voids are interconnected which is always not true.
Permeability	A sample is placed in a permeameter or triaxial test cell and water is passed through the sample under a hydraulic head The loss of head caused by the resistance to flow of the sample is measured.	Most material are anisotropic with regard to permeability and if tube samples are tested along their long axis values of coefficient of permeability obtained may not be those required for groundwater calculations.
Ultra-sonic velocity	The velocity of sound waves through a sample of rock is measured, commonly using a velocity tester developed for concrete testing. Compression wave velocities are commonly measured but shear wave velocities may be measured if special transducers are used.	The velocity of sound waves through rock is related to density, porosity and strength of the material. The test provides useful correlation data and has the benefit of being non-destructive. The test results depend upon the frequency of the signal
Unconfined tensile strength (UTS)	A cylinder or irregular lump of rock is subjected to longitudinal tension until failure occurs.	To subject specimen to tension caps must be glued to specimen. There are great difficulties in adequate sample preparation.
Brazilian test	A disc of rock is loaded across a diameter until failure occurs. Failure occurs in tension across the loaded diameter.	Widely used to measure tensile strength in rocks on discs cut from the rock cores. Generally the specimen may fail under intense compressive stresses developed at platen contact, special platens used.
Point load strength (Is)	A cylinder of rock is loaded between two 5mm radius spherical points set in a 60° cone, until failure occurs.	It may be correlated with UCS. Test can be undertaken on rock cores and irregular lumps. Accuracy of correlation with UCS uncertain.
Shear box test	A sample of soil or rock is placed in a split box. The top part of the box and the sample are subjected to a horizontal (shear) load. The sample may be sheared under a vertical load. If three vertical (normal) loads are applied to three similar samples values of cohesion (C) and angle of shearing resistance (ϕ) may be deduced from the material.	Used for Shear strength (T), C and ϕ determination in soils. Commonly used for discontinuity strengths in rocks and between rock concrete.
Triaxial test	1 A sample of soil or rock is placed in a split box. The top part of the box and the sample may be sheared under a vertical load. If three vertical (normal) loads are applied to three similar samples values of cohesion (C) and friction (ϕ) may be deduced. Various types of triaxial test may be undertaken, principally the "undrained" test and the "drained" test. 2.Measurements of modulus of deformation (E) of the sample under increasing stress and poissons ratio are also done.	1. The purpose of the triaxial test is to determine the reaction of rock specimens to confining pressures similar to those found in the earth's stress field. 2. C and ϕ of intact material is determined. The most common soil strength test. Little used for rocks (except very weak rocks) because of hydraulic pressures required. Generally applied to most bearing capacity problems. 3. E-vales determined mostly in rocks for settlement and deformation calculations. In soils for immediate settlement calculations. Results sensitive to rates of loading and strain.
Tilting table test	A table is slowly tilted to an angle at which the upper part of a sample slides off along a pre-existing separation plane. Friction angle along the separation plane.	Testing is limited to low values of normal stress.

Table 3: In-situ testing of the discontinuities and rock masses as per IAEG Commission (Price 1981)

Test/ Technique	Principles of technique	Remarks
Plate load test	On the surface of soil or rock, at the earth's surface or in a tunnel or shaft a plate is pressed and the penetration into the soil or rock is measured.	Modulus of deformation and modulus of elasticity of a rock mass is determined. Results of test are used to predict rock and soil mass settlement and/or deformation under influence of stresses caused by engineering works.
Flat jack test	In a slot in rock a thin flat jack of large dimension is pumped up with oil under pressure. Deformation of the surrounding rock is measured.	Modulus of deformation and modulus of elasticity of a rock mass is determined. Results of this test are used to predict rock mass deformation and to determine the residual stress in a rock mass.
Direct shear test	A large sample is prepared in situ in soil or rock. A normal load is applied to the top of sample and shear load is increased until the sample is sheared through intact material or along discontinuity at a certain level of normal stress. If various tests of identical blocks at different normal loads can be executed the cohesion and angle of friction of the tested material or discontinuity can be determined.	Used for the determination of shear resistance of intact material and discontinuities in dam foundations or slope stability studies. Test is also executed along interface of rock and concrete. Rate of strain strongly influences the result.
Deformation openings	A slot of hole is opened in rock. Deformation of the rock around the opening is measured with extensometers.	Deformation of rock upon unloading from the natural existing stress is obtained. It is used to predict unloading phenomenon around large underground openings. Rock mass should not be intensely jointed.
Deformation of borehole after overcoring	Diameter of borehole is carefully measured before and after coring. Deformation of rock upon unloading from the natural existing stress is obtained.	Used for determination of natural stress condition in a borehole. Rock mass should not be intensely jointed.
Restoration of stress	Open slot in rock is extended with help of a flat jack to balance the original condition before the opening of the slot. Pressure of a fluid in flat jacks or borehole is considered to balance natural stress condition when rock around the slot or borehole is brought back to original condition.	Used to determine natural stress condition in rock. Rock mass should not be intensely jointed.
Hydro-fracture test	Fluid pressure is applied to a test section of a drill hole isolated by packers. The fluid pressures required to generate, propagate, sustain and reopen fractures in rock at the test horizon are measured and are related to the existing stress field. Directions of measured stress are usually obtained by observing and measuring the orientation of hydraulically induced fracture (hydro-fracture) plane. The drill hole direction is assumed to be a principal stress direction. Vertical pressure is considered from the overburden weight.	The method is more suited to measurements at depth > 50m that are beyond the capabilities of most other techniques. It has the advantage of requiring no advance knowledge of the elastic properties of the rock and being able to be carried out without difficulty below the water table.
Permeability tests	Water is pumped out from or injected into a borehole in the ground to obtain permeability of ground mass of different type. In pumping test a section of borehole is closed with packers at top and bottom and water is pumped in under pressure; water losses or recorded. Alternatively a single packer is used to test the part of the borehole below it.	The value of the results will be related to frequency and aperture of the joint systems in relation to the size of the borehole.
Grouting tests	The packer test is undertaking using cement grout as injection fluid to determine the grout acceptance of the groundmass.	Used for ground mass improvement.
Rock anchors, ground anchors and rock bolt tests	Rock anchors, ground anchors and rock bolts are properly installed. Pulling force is then applied up to the level that anchor or bolt is pulled out or broken. Maximum load applicable to anchor or bolt is obtained.	Large variations of the value from anchor to anchor are possible when ground conditions are in homogeneous.

h. Monitoring of movements in rock mass: Monitoring of engineering structures is the surveillance, either visually or with the help of instruments. Monitoring is important to check the rock mass response and as a consequence to adjust the overall designs or takes remedial measures. The different devices are used in monitoring of movements and pressures of rock mass/ ground (Table 4).

Table 4: List of In-situ instruments as per IAEG Commission (Price 1981)

Test/ Technique	Principles of technique	Remarks
Extensometer	Length changes between the borehole mouth and one or more fixed points along the borehole are determined with transmission rods or tensioned wires. Relative movements between the borehole mouth and the fixed points are obtained.	Extensometers are used to determine movement and deformations in soil and rock mass. Length of extensometer may vary from 10m to 200m. Maximum number of fixed point is eight.
Earth pressure cell	A cell is embedded in the soil of a fill, cemented in the wall of a tunnel between rock and tunnel lining. Hydraulically or electrically the stress acting in the ground or tunnel lining is recorded. Earth pressure at location of the cell are obtained	Use to determine stresses in soil, rock and concrete. Used in soil fills in retaining walls, dams, in lining of tunnels, etc. These equipments cannot be retrieved after tests.
Anchor load cell	An elastic element is fastened between the anchor plate on the surface and the anchor head. Elastic deformation of the cell can be translated to anchor tension	Used for measurement of anchor load over longer time intervals and for the determination of anchor failure load. These cells can be retrieved after use.
Glass plate	Simple device to know the movement along the cracks. Glass plate breaks easily under tension.	Used for rough visual observations.
Demac gauge	Six points each on both sides of cracks were established for observing the distance between gauges.	Used to study movement along cracks.
3-D Crack monitor	Relative displacement of two rock or concrete masses in any particular direction is determined as change of distance between a Sensor plate mounted on one of the masses and a Dial gauge mounted on the other rock mass. The readings are taken in the direction of X-axis which is along the crack, Y-axis which is perpendicular to the X-axis and in the plane of the surface of the crack, Z-axis which is perpendicular to both X and Y axes and is also perpendicular to the plane of the surface of the crack.	Used to register the crack displacement readings in three directions.

1.4 Characterization of the rock mass

Quantitative description and assessment of the rock mass is essential for design purpose. Classification of rocks and soils should be based on principle that the physical or engineering geological properties of a rock in its present state are dependent on the combined effects of mode of origin, subsequent diagenetic, metamorphic and tectonic history, and on weathering processes (UNESCO-IAEG, 1976). The orientation, location, persistence, water pressure, wall strength, degree of weathering, type of in filling material and shear strength of critical discontinuities are some of the important parameters which can be described quantitatively. These parameters can be used in conjunction with physico-mechanical properties in correct assessment and evaluation of rock mass and in the stability analysis.

Rock mass classification should effectively combine the finding from observation, experience and engineering judgment to provide a quantitative assessment of rock mass conditions, support requirements and foundation treatment. Rock mass classification schemes have been developing for over 100 years since Ritter (1879) attempted to formalise an empirical approach to tunnel design for determining support requirements. Terzaghi (1946) used the descriptive classification in which the rock loads, carried by steel sets, are estimated for the design of tunnel support. Lauffer (1958) proposed that stand-up time for an unsupported span is related to the quality of the rock mass in which the span is excavated. The Rock Quality Designation index (RQD) was developed by Deere (Deere et al. 1967) to provide a quantitative estimate of rock mass quality from drill core logs. Palmström (1982) estimated RQD from the surface exposures. Wickham et al. (1972) described a quantitative method for describing the quality of a rock mass and for selecting appropriate support on the basis of their Rock Structure Rating (RSR) classification. Bieniawski (1976) developed a rock mass classification called the Geomechanics Classification or the Rock Mass Rating (RMR) system and refined successively on the availability of more case histories (Bieniawski 1989). Bieniawski's classification is based on the case histories of civil engineering projects. Laubscher (1977, 1984), Laubscher and Taylor (1976) and Laubscher and Page (1990) have described a Modified Rock Mass Rating system for Mining. Cummings et al. (1982) and Kendorski et al. (1983) have also modified Bieniawski's RMR classification to produce the MBR (modified basic RMR) system for mining. Barton et al. (1974) of the Norwegian Geotechnical Institute (NGI) proposed a Tunneling Quality Index (Q) for the determination of rock mass characteristics and tunnel support requirements on the basis of an evaluation of a large number of case histories of underground excavations.

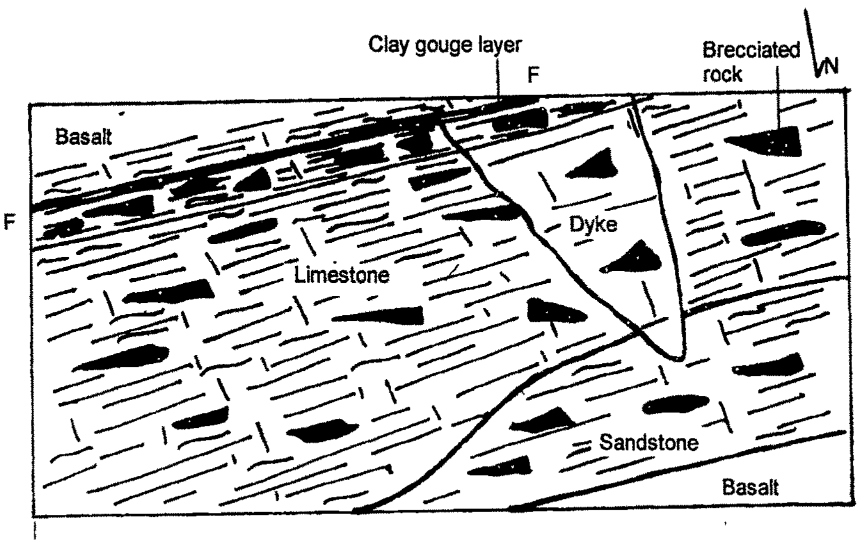
Barton et al. (1980) provided additional information on rock bolt length, maximum unsupported spans and roof support pressures to supplement earlier support recommendations. Hoek et al. (1997) developed a new index called the Geological Strength Index (GSI) especially for very poor rock (RMR <25). Bieniawski's RMR (1976) and Barton et. al's Q (1974) classification systems have been mainly used for the present study. These two systems are most widely used today.

1.4.1 Description of rock mass

The complete specification of a rock mass requires descriptive information on the nature and distribution in space of materials, which constitute the mass. Description is the initial step in an engineering assessment of rocks and rock masses. The behaviour of a rock mass is determined by the type, spacing, orientation and characteristics of the discontinuities present. As a consequence, the parameters which ought to be used in a description of a rock mass include the nature and geometry of the discontinuities as well as it's overall strength, deformation modulus and secondary permeability.

A discontinuity is a surface within the rock mass that is open or potentially openable under the stress levels applicable in engineering because the tensile strength across the surfaces is lower than that of rock material. Discontinuity may be filled or "healed" entirely or over a significant portion of their aerial extent by quartz, calcite, or other minerals (Fig. 4). Even larger fissures are sealed due to intrusion of magma. Veins may be present without healing the Discontinuity or may have been broken again forming new surfaces. Soluble fillings such as gypsum may cause foundation or structural degradation during the expected lifetime of structures.

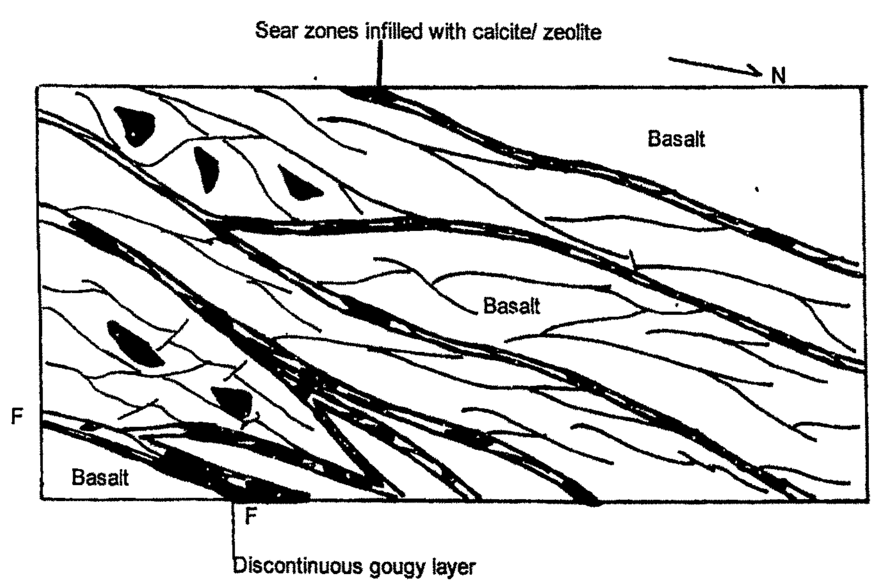
Some of the descriptors useful in the classification of the rock mass are tabulated below (USDI 1991):



Alphanumeric description of Narmada fault zone:

Rock strength	UCS5
RQD	RQD5
Weathering	W2
Discontiuity spacing	DS4-DS5
Roughness	R5-R6
Aperture	A5-A6
Block size	BS4-BS5
Discontiuity continuity	C4-C5
Filling thickness	T1-T2
Moisture conditions	M4-M5
Healing	HL4-HL5

I. Mostly unhealed (HL 4-5, RMR 11) Narmada River channel fault zone at Narmada dam base



Alphanumeric description of Karjan fault zone:

Rock strength	UCS2
RQD	RQD4
Weathering	W1
Discontiuity spacing	DS1-DS2
Roughness	R3
Aperture	A6-A7
Block size	BS2-BS4
Discontiuity continuity	C4-C5
Filling thickness	T1-T2
Moisture conditions	M4
Healing	HL1-HL2

II. Mostly healed (HL 1-2, RMR 48) Karjan River fault zone at Karjan dam base

0 2 4 6m
Scale

Fig. 4: Description of Narmada and Karjan dam River Channel Fault Zones

Table 5: Rock Strength (Unconfined Compressive Strength)

Alphanumeric symbol	Description	Compressive strength (MPa)	Remarks
UCS1	Extremely strong	>250	
UCS2	Very strong	100-250	
UCS3	Strong	50-100	
UCS4	Moderately strong	25-50	Soft rocks are weaker than 50MPa and strong rocks are stronger than 50MPa
UCS5	Weak	1.5-25	Rocks with strength under 12.5 MPa, as a rule, hard soils.

Table 6: Rock Quality Designation (RQD)

Alphanumeric symbol	Description	Value	Remarks
RQD1	Excellent	90-100	
RQD2	Good	75-90	
RQD3	Fair	50-75	
RQD4	Poor	25-50	
RQD5	Very Poor	0-25	Where RQD is reported or measured as ≤ 25 (including 0), a nominal value of 10 is used to evaluate Q (Barton et al. 1974)

Table 7: Weathering Grades for the Rock Mass

Alphanumeric symbol	Description	Characteristics	Degree of change (%)	Grade
W1	Fresh	No visible sign of rock material weathering. Slight discolouration on major discontinuity surfaces.	0	I
W2	Slightly weathered	Discolouration indicates weathenng of rock material and discontinuity surfaces.	>1-10	II
W3	Moderately weathered	Less than 35% of the rock material is decomposed and/or disintegrated to a soil, Fresh or discoloured rock is present either as a continuous framework or as corestones	10-35	III
W4	Highly weathered	More than 35% of the rock material is decomposed and/or disintegrated to soil, Fresh or discoloured rock is present either as a discontinuous framework or as corestones	35-75	IV
W5	Extremely weathered	All rock material is decomposed and/ or disintegrated to soil. The original mass structure is still largely intact.	>75	V
W6	Residual soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is large change in volume, but the soil has not been significantly transported.	100	VI

Table 8: Discontinuity Spacing

Alphanumeric symbol	Description	SPACING
DS1	Very widely spaced	> 2m
DS2	Widely spaced	600mm-2m
DS3	Medium spaced	200mm-600mm
DS4	Closely spaced	60mm-200mm
DS5	Very closely spaced	< 60mm

Table 9: Roughness Categories for Discontinuity Surfaces

Alphanumeric symbol	Description	CATEGORY
	STEPPED	
R1	Rough (or irregular)	I
R2	Smooth	II
R3	Slickensided	III
	UNDULATING	
R4	Rough(or irregular)	IV
R5	Smooth	V
R6	Slickensided	VI
	PLANAR	
R7	Rough(or irregular)	VII
R8	Smooth	VIII
R9	Slickensided	IX

Table 10: Aperture (Opening) Of Discontinuity Surfaces

Alphanumeric symbol	Description	CATEGORY
A1	Very wide	>200mm
A2	Wide	60-200mm
A3	Moderately wide	20-60mm
A4	Moderately narrow	6-20mm
A5	Narrow	2-6mm
A6	Very narrow	>0-2mm
A7	Tight	Zero

Table 11: Block Size

Alphanumeric symbol	Description	Block size	Equivalent discontinuity spacing in blocky rock	Volumetric joint count (J_v) (After Barton 1978) (Joints/m^3)
BS1	Very large	> 8 m^3	Extremely wide	< 1
BS2	Large	0.2-8 m^3	Very wide	1-3
BS3	Medium	0.008-0.2 m^3	Wide	3-10
BS4	Small	0.00002-0.008 m^3	Moderately wide	10-30
BS5	Very small	< 0.0002 m^3	< Moderately wide	>30

Table 12: Discontinuity Continuity

Alphanumeric symbol	Description	Category
C1	Discontinuous	>1m
C2	Slightly continuous	1 to 3m
C3	Moderately continuous	3 to 10m
C4	Highly continuous	10 to 20m
C5	Very continuous	>20m

Table 13: Filling Thickness

Alphanumeric symbol	Description	Thickness
T0	Clean	No film or coating
T1	Very thin	<1mm
T2	Moderately thin	1to 3mm
T3	Thin	3 to 10mm
T4	Moderately thick	10 to 30mm
T5	Thick	>30mm

Table 14: Moisture Conditions

Alphanumeric symbol	Criteria
M1	The Discontinuity is dry. It is tight or filling (where present) is sufficient density or composition to impede water-flow. Water flow along the Discontinuity does not appear possible.
M2	The Discontinuity is dry with no evidence of previous water-flow. Water-flow appears possible.
M3	The Discontinuity is dry but shows evidence of previous of water-flow such as staining, leaching and vegetation.
M4	The Discontinuity filling (where present) is damp, but no free water is present.
M5	The Discontinuity shows seepage. It is wet with occasional drops of water.
M6	The Discontinuity emits a continuous flow (flow rate is to be estimated) under low pressure. Filling materials (where present) may show signs of leaching or piping.
M7	The Discontinuity emits a continuous flow (flow rate is to be estimated) under moderate to high pressure. Water is squirting and/or filling material (where present) may substantially washed out.

Table 15: Healing Of Fractures/ Shear/ Fault Zones

Alphanumeric symbol	Description	Criteria
HL0	Totally healed	Fracture is completely healed or re-cemented to a degree at least as hard as surrounding rock.
HL1	Near totally healed	Greater than 75% of fractured material. Discontinuity surfaces and/or strength of the healing agent are nearly as hard as surrounding rock.
HL2	Moderately healed	Greater than 50% of fractured material. Fracture surfaces; and/or strength of the healing agent is less hard as surrounding rock.
HL3	Partly healed	Less than 50% of fractured material, filling, or Discontinuity surface is healed or re-cemented.
HL4	Slightly healed	Less than 25% of fractured material, filling, or Discontinuity surface is healed or re-cemented.
HL5	Not healed	Discontinuity surface, Discontinuity zone, or filling is not healed or re-cemented; rock fragments or filling (if present) held in place by their own angularity and/ or cohesiveness

1.4.2 Geomechanics Classification

The geomechanics classification system is also known as the Rock Mass Rating (RMR) system. The classification can be used for estimating the unsupported span, the stand-up time or bridge action period and the support pressures of an underground opening. It can also be used for selecting a method of excavation and permanent support system. Estimation of cohesion, angle of internal friction and elastic modulus of the rock can be done (IS: 1998). In its modified form RMR can be used for predicting the ground condition for tunneling and also for foundation.

In the present study Bieniawski's 1976 Geomechanics Classification of the Rock Mass Rating (RMR) system has been mainly used. In applying this classification system, the rock mass is divided into a number of structural regions and each region is classified separately. The following six parameters are used to classify a rock mass using the RMR system:

(i) Uniaxial compressive strength of rock material.
(ii) Rock Quality Designation (RQD)
(iii) Spacing of discontinuities.
(iv) Conditions of discontinuities.
(v) Ground water conditions.
(vi) Orientation of discontinuities.

a. Rock quality index (RQD): RQD is defined as the percentage of intact core pieces longer than 100mm (4inches) in the total length of core (Deere et al. 1967). The core should be at least NX size (54.7mm or 2.15 inches in diameter) and should be drilled with a double-tube core barrel.

RQD = (Σ Length of core pieces > 10cm length/ Total length of core run) X 100%

Palmström (1982) suggested that, when no core is available but discontinuity traces are visible in surface exposures or exploration adits, the RQD might be estimated from the number of discontinuities per unit volume. The suggested relationship for clay-free rock masses is:

RQD = 115-3.3 J_v

Where J_v is the sum of the number of joints per unit length for all joint (discontinuity) sets known as the volumetric joint count.

b. Rock Tunneling Quality Index (Q): The numerical value of the Barton's 1974 Tunneling Quality Index (Q) varies on a logarithmic scale from 0.001 to a maximum of 1,000 and is defined by:

$Q = RQD/J_n \times J_r/J_a \times J_w/SRF$

Where RQD is the Rock Quality Designation, J_n is the joint set number, J_r is the joint roughness number, J_a is the joint alteration number, J_w is the joint water reduction factor and SRF is the stress reduction factor.

1.4.3 Estimation of tunnel supports

The principal objective in the design of underground excavation support is to help the rock mass to support itself (Hoek and Brown 1980). However, supports are required to be installed to stabilize the rock mass. Estimation of roof and wall supports is essential for the safety and stability of underground cavern and tunnels.

a. Excavation support ratio and Equivalent dimension: In relating the value of the index Q to the stability and support requirements of underground excavations, Barton et al. (1974) defined an additional parameter which they called the *Equivalent Dimension*, D_e, of the excavation. This dimension is obtained by dividing the span diameter or wall height of the excavation by a quantity called the *Excavation Support Ratio, ESR*. Hence:

D_e = Excavation span, diameter or height (m)/ Excavation Support Ratio *ESR*

The value of *ESR* is related to the intended use of the excavation and to the degree of security, which is demanded of the support system, installed to maintain the stability of the excavation. Barton et. al. (1974) suggested ESR value 1.0 for the Power stations, major road and railway tunnels, civil defense chambers, portal intersections. Thus in the present study of underground powerhouse ESR 1 has been used.

b. Unsupported span: The maximum unsupported span can be estimated from:

Maximum span (unsupported) = 2 ESR $Q^{0.4}$

c. Support pressure: Roof and wall support pressures are estimated as detailed below:

Permanent support pressure P_{roof} (P_v) can be estimated from the equation (Grimstad and Barton 1993):

P_{roof} (P_v) = $(2/3\ J_r) \times (J_n)^{1/2} \times (Q)^{-1/3}$

Estimation of short-term Roof Support Pressure (pvi) after Barton et al. (1980):
pvi= pv/1.7

Where, pv (i.e. p_v) is ultimate roof support pressure.

Wall Rock Mass Quality (Qh): The ultimate wall rock mass quality has been estimated by multiplying Q with the wall factor (W). For different range of Q, different values of wall factor has been suggested by Barton et al. (1975) as given below:

For	$Q > 10$	$W = 5.0$
	$0.1 < Q < 10$	$W = 2.5$
	$Q < 0.1$	$W = 1.0$

Estimation of Ultimate Wall Support Pressure (ph): The ultimate wall support pressure (ph) is estimated by the following equation:

ph = $(2/3\ Jr) \times (Jn)^{1/2} \times (Qh)^{-1/3}$

Estimation of short-term wall support pressure (phi) can be done by following equation:

phi=ph/1.7

d. Estimation of shotcrete capacity ps: Shotcrete capacity can be determined by following equation:

ps= t σ_c/R

Where t = thickness of shotcrete, σ_c = compressive strength of shotcrete and R = radius of curvature of shotcrete layer.

e. Estimation of the bolt capacity pb: The load pb sustainable by a rock bolt is as below:

pb= Bc/a

Where

Bc = yield capacity of the bolt and

a = area of influence of bolt

The bolt yield load Bc can be given by estimated by following equation:

Bc= Sb/ Ab

Where

Sb = yield stress of bolt material and

Ab = cross sectional area of bolt

f. Estimation of rock bolt and cable length: Barton et al. (1980) provided additional information on rock bolt length, maximum unsupported spans and roof support pressures to supplement the earlier support recommendations. The length L of rock bolts and cables can be estimated from the excavation width (span) B and height H for roof and sidewall and the excavation Support Ratio ESR as below:

(i) Arch roof support

Bolt length

L =(2+ 0.15B) / ESR i.e. for ESR=1 for the powerhouse

L =(2+ 0.15B)

Cable length

L = 0.4 X B

(ii) Side wall support

Bolt length

L = (2+ 0.15H)/ ESR i.e. for ESR=1 for the powerhouse

L = 2+ 0.15H

Cable length

L= 0.35 X H

1.4.4 Use of the rock mass classification in the estimation of in-situ deformation modulus

The in-situ deformation modulus (E_m) of a rock mass is an important parameter in the numerical analysis and in the assessment of deformation around underground openings and in the dam foundations. Deer's RQD approach is now seldom used for estimating in-situ deformation modulus Em (Deere and Deere, 1988). Several workers have attempted to estimate its value based on the analysis of a number of case histories (many of which involved dam foundations) and rock mass classifications and developed following relationships:

i. Bieniawski's (1978):

$$E_m = 2\ RMR - 100$$

ii. Serafim and Pereira (1983):

$$E_m = 10^{(RMR-10)/40}$$

iii. Barton et al. (1980):

$$E_m = 25\ Log_{10}\ Q$$

The Serafim and Pereira (1983) equation provides a reasonable fit for all the observations plotted for wider range of RMR value (Hoek et. al. 1977) (Fig. 5).

1.5 Salient features of the Narmada and Karjan dams

The Narmada dam (21°50'; 73°45') under construction is a multi-purpose concrete gravity dam creating a terminal reservoir on the Narmada River. The main dam is 1227m long and 162m high from the deepest foundation level (129m from the river bed level). Other important structures at Narmada project site are 1200 MW underground powerhouse and 250MW surface powerhouse and four rock fill dams (Table 16 and 17). The Karjan dam (21°49'; 73°32') constructed mainly for irrigation purpose is a masonry-cum-concrete gravity dam located on the left bank of Narmada River at about 25km downstream of Narmada dam across its tributary a Karjan River (Fig. 1 and Plate 1). The dam is 911m long and 100m high from deepest foundation level. The Karjan dam constitutes the first major step in the development of the Narmada Basin in Gujarat State.

	RMR
I. Narmada river channel fault	11
II. Karjan River channel fault	48

Underground powerhouse	RMR	Q
1. Jointed Basalt	60	9.16
2. Jointed inclined dolerite dyke	63	10
3. Vertical dolerite dyke	45	1.5
4. Dolerite sill 4a	30	0.6
4b	40	1.25
5. Shear zone	35	1.25

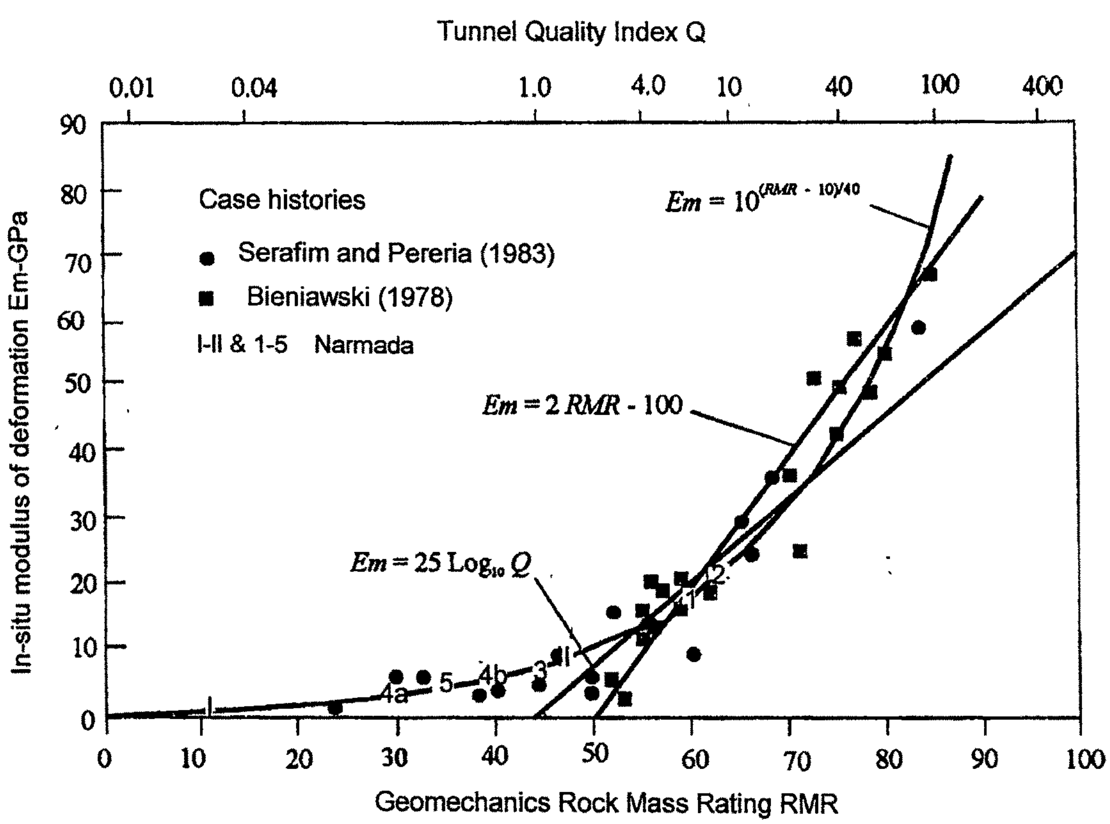


Fig. 5: Prediction of in-situ deformation modulus Em from rock mass classification (After Hoek et. al. 1997)

Table 16: Salient features of Narmada and Karjan dams

	Narmada dam	Karjan dam	
A. Main Dam			
Type	Concrete gravity	Masonry cum concrete gravity	
Total length	1270m	911m	
Crest level of spillway (R.L.)	121.92m	101.23m	
Full reservoir level (FRL)	138.68m	115.25m	
High Flood level	140.21m	116.0m	
Minimum draw down level (MDL)	110.64m	78.0m	
Top of dam	146.50m	119.70m	
Maximum height of the dam from the deepest foundation level	163m	100m	
B. Spillway			
Type	Gated spillway	Gated spillway	
Shape of crest	Ogee	Ogee	
Number, size and type of gates	7Nos. (Size 18.30x18.30m-Auxillary spillway), 23Nos. (18.30x16.76m)	9Nos. (Size 15.55x14.20m)	
Energy dissipation device	Sloping-cum-horizontal jump type stilling basin for main spillway and split-level chutes terminating into ski-jump buckets for auxiliary spillway	Stilling basin with horizontal apron and fillip at the end	
C. Rock fill dam	4nos., total length 2567m, maximum height above deepest foundation level 55.60m, M.W.L. 99.10m, F.S.L. 95.10m, L.W.L. 83.80m and storage at F.R.L. 63.39MCM	-	
D. Canal System	Main	Left bank Main Canal	Right bank main canal
Type	Lined contour	Lined contour	Lined contour
Length	460km	51km	126.6km
Gross command area	34.286 lakh ha	60350ha	19435ha
Cultivable command	21.190 lakh ha	42510ha	13690ha
Annual irrigation	17.920 lakh ha	58664ha	18892ha

Table 17: Sardar Sarovar (Narmada) Project Power Installation

	River bed (underground) powerhouse	Canal head powerhouse
Number of unit	6	5
Rated capacity each unit	200MW	50MW
Installed capacity	1200MW	250MW
Types of Turbine	Francis vertical (Reversible)	Kaplan (Conventional surface)