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Geotechnical Problems of Water Resources Development in India
The Role of Instrumentation

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Dr. V.M.Sharma graduated in Civil Engineering from IIT Kharagpur in the Year 1961. He joined the Central Water and Power Commission and started working in its 'Design and Research' Wing dealing with the Design of Concrete Dams. He took Post Graduate Diploma and Master's degree from the University of Roorkee in the Water Resources Development. Subsequently he took his Ph.D in Rock Mechanics from I I T Delhi in the Year 1985, working on the behaviour of tunnels and prediction of closures and rock loads. He worked as a UNDP Fellow for one year at the University of Alberta Canada, with Prof. Morgenstern. From Central Water Commission he was deputed to Idukki Project on deputation as Executive Engineer Quality Control for nearly two years.

On his return to Delhi he was posted at the Central Soil and Materials Research Station, where he took charge of the Rock Mechanics Division. He rose to the level of Director CSMRS and worked as its Head of the Department for about six years before taking voluntary retirement in the year 1995. He, then, started a new Consultancy Division called Advanced Technology and Engineering Services at AIMIL and worked as its Chief Consultant. Presently he is one of the Directors of AIMIL Ltd.

Dr. Sharma is a Donor Fellow of the Indian Geotechnical Society. He has remained its Honorary Secretary for a few terms and the President of the Societyfor one term. He is a Fellow of the Indian National Academy of Engineering. He is the President of the Indian Society of Rock Mechanics and Tunnelling Technology. He has published several papers and edited or co-edited about ten books dealing with geotechnical problems relating to Water Resources Development. He was the General Reporter for the Session dealing with Underground Structures at the International Soil Mechanics and Geotechnical Engineering Conference at Alexandria, Egypt in Oct. 2009. He is designated as a Key note Speaker at the Asian Regional Conference in Hong Kong in May 2011 and will speak on Rock Engineering. He is on the Editorial Board of several Technical Journals.

Dr. Sharma worked in Industry, keeping in touch with the Academics.

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V.M. Sharma[2]

"Let not a single drop of water that falls on the land go into the sea without serving the people."

-Parakrama Bahu,
12th Century Ruler of Serendip
(Now Sri Lanka)

Abstract: Development and management of water resources is essential for any country to ensure its existence. We are fortunate to have enough of this precious resources to fulfill all our needs. However, there is a vast disparity when it comes to its distribution - both location wise and time wise. As a result, there are floods in the monsoons and enormous amount of water goes down the drain. At the same time there is shortage of water and electricity in dry months. It is, therefore, all the more important that we develop and manage our water resources well. Geo-technical problems are inevitable in any water resources development project. Starting from the investigations or lack of it, there are questions relating to its adequacy, quality of drilling, sampling and testing both in the field and in the laboratory. The problems are mainly in the areas of construction materials, dam foundations, slope stability, underground excavations and those relating to dynamic situations. There is a wide gap between the practice and the state of the art and this gap needs to be bridged. Enough expertise is available within the country which has to be mobilized. Academy and industry must work together towards the common goal of achieving excellence and leading the world, for we and only we have the opportunity and the resource to achieve it.

Introduction

The arrival of summer in India is still marked by shortage of water and electricity. The situation in some parts of the country is worse. Every year there is drought affecting millions of people in one part of the country or the other. Every summer, there is a large scale migration of livestock, as water tanks, lakes and wells run dry. Generally the situation calls for emergency measures and large amounts of money are spent on ad-hoc short-term measures.

It is estimated that about 400 million hectare meter of rain precipitates over the Indian land mass and there is additional 20 million hectare meter of river inflow into the country from the neighbouring countries. Given a geographical area of 329 million hectares, this quantum of water would theoretically imply that if spread evenly would submerge the entire national land surface uniformly to a depth of approximately 1.28 meters.

Notionally this is a lot of water. But much of this runs into sea, largely as monsoon flood, while a great deal of soil moisture is evaporated or evapo-transpirated by plants, trees and forests. A residual amount flows out of the country into Bangladesh, Myanmar and Pakistan.

Floods on the other hand also cause a lot of problems and damage. Traditionally the great civilizations have flourished on the banks of rivers and *Ganga, Yamuna, Brahmaputra* and *Godavari* are no exception. At the same time some of these civilizations have suffered a lot and even perished due to floods. With the rapid urbanization, the need for water has become more varied.

More water is required for producing more food to meet the demands of the ever-growing population, besides other agricultural produce which need diversified water requirements. Industrialization, municipal uses, ecological demands to dilute pollutant inflows, development of fisheries, inland navigation and recreational uses also demand availability of water and these demands are ever on the increase. Unless we develop a chain of reservoirs - both underground and over ground - to impound a large part of the monsoon that goes waste at present and adopt other supplementary techniques such as afforestation, watershed management, soil conservation and rain water harvesting etc., we will lead ourselves into a catastrophic situation.

Let us look at some of the facts and figures and related problems in this regard. The most reliable information in this connection can be had from the Ministry of Water Resources. The website of the Ministry

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starts with underlining the importance of development and management in an integrated manner. The same philosophy has been reflected in the National Water Plan prepared in the year 2002. It states,

"India is endowed with a rich and vast diversity of natural resources, water being one of them. Its development and management plays a vital role in agriculture production. Integrated water management is vital for poverty reduction, environmental sustenance and sustainable economic development." National Water Policy (2002) envisages that the water resources of the country should be developed and managed in an integrated manner.

'Water is food and fire is the eater of the food. Fire is established in water and Water is established in fire' -Taittiriya Upanishad 3.8

General Facts

The crisis about water resources development and management arises because most of the water is not available for use and secondly it is characterized by its highly uneven spatial distribution. Accordingly, the importance of water has been recognized and greater emphasis is being laid on its economic use and better management.

The water resources have two facets. The dynamic resource, measured as flow is more relevant for most of developmental needs. The static or fixed nature of the reserve, involving the quantity of water, the length of area of the water bodies is also relevant for some activities like pisciculture, navigation etc. Both these aspects are discussed below.

Irrigation World

Analyzing the country-wise geographical area, arable land and irrigated area in the World, it is found that among Asian countries, India has the largest arable land, which is close to 39 per cent of Asia's arable land. Only United States of America has more arable land than India.

Water in Indian Constitution

India is a union of States. The constitutional provisions in respect of allocation of responsibilities between the State and Centre fall into three categories: The Union List, the State List and the Concurrent List. As most of the rivers in the country are inter-State, the regulation and development of waters of these rivers, is a source of inter-State differences and disputes. In the Constitution, water is a matter included in the State List. This entry is subject to the provisions made in the Union List.

Irrigation

The irrigation projects are classified into three categories viz. major, medium and minor. A broad assessment of the area that can be ultimately brought under irrigation, both by surface and ground water, showed that the ultimate potential of irrigation is about139 m.ha.

Hydro-Electric Power

India has a vast potential for hydro-power generation, particularly in the northern and north-eastern region. As per an estimate of Central Electricity Authority, the potential in the country is assessed as 84,000 MW at 60 per cent load factor, which is equivalent to about 450 billion units of annual energy generation. The basin wise distribution is as given in Table 1 below.

At the time of independence, out of total installed capacity of 1362 MW, hydro-power generation capacity stood at 508 MW. The capacity has since been raised to about 13,000 MW. In addition 6,000 MW is available from projects under construction. A potential of about 3,000 MW is contemplated from projects already cleared. The total potential harnessed/under harnessing would thus be about 22,000 MW which is nearly one-fourth of the estimated potential.

Table 1 Basin Wise Distribution

Sl.No.	*Basin*	*Potential at 60% load factor [MW]*
1	Indus Basin	20,000
2	Brahmaputra Basin	35,000
3	Ganga Basin	11,000
4	Central India Basin	3,000
5	West Flowing River System	6,000
6	East Flowing River System	9,000
	Total	84,000

Domestic Water Supply

The National Water Policy has assigned the highest priority for drinking water supply needs followed by irrigation, hydro-power, navigation industrial and other uses. The drinking water requirements of most of the mega cities/cities in India are met from reservoirs of irrigation/multi-purpose schemes existing in near by areas and even by long distance transfer. Delhi getting drinking water from Tehri Dam and Chennai city from Krishna Water through Telugu Ganga Project are typical examples.

Navigation

Total navigable length of inland water-ways in the country is 15,783 km of which maximum stretch lies in the state of Uttar Pradesh followed by West Bengal, Andhra Pradesh, Assam, Kerala and Bihar successively. Amongst the river system, the Ganga has the largest navigable length followed by the Godavari, the Brahmaputra and the rivers of West Bengal. Waterways are having the unique advantage of accessibility to interior places. Besides, they provide cheaper means of transport with far less pollution and communicational obstacles. The waterways traffic movement has gone up progressively from 0.11 m.t. in 1980-81 to 0.33 m.t. in 1994-95.

The development of inland water transport is of crucial importance from the point of energy conservation as well. The ten waterways identified for consideration for being declared as national waterways are namely:

1. The Ganga – Bhagirath - hoogli
2. The Brahmaputra
3. The Mandavi, Zuari river and the Cumbarjua Canal in Goa
4. The Mahanadi
5. The Godavari
6. The Narmada
7. The Sunderbans Area
8. The Krishna
9. The Tapi, and
10. The West Coast Canal

The Ganga – Bhagirath-Hoogli and Brahmaputra have already been declared as National Waterways. Farakka Navigation Lock has been opened for transport, thus allowing transport for upstream reaches of Ganga with Calcutta. With network of national waterways the carriage and cargo in this sector in the 10 river systems is expected to increase by 35 m.t. per year. The consumptive use of water for navigation is not substantial as the wastage is only at the point of terminal storage projects.

Industrial Use

A basic necessity of industrial development is adequate availability of water. The Second Irrigation Commission in their report of 1972 recommended a provision of 50 b.cu.m. for industrial purpose for the country as a whole. However, a recent assessment indicates that requirement for industrial use during 2000 AD will be about 30 b.cu.m. while it will rise to 120 b.cu.m by 2025 AD.

Inland Pisciculture

As a consequence of water resources development works, apart from the major objectives there has been development in various other sectors as well. Among them, development in inland fish production occupies a prominent place. During 1950-51, total inland fish production stood at 0.22 m.t. which by 1994-95 has gone up to 2.08 m.t. India has now the distinction of being the seventh largest producer of fish in the world and second largest producer of inland fish after China. Amongst the States, West Bengal is the highest producer followed by Andhra Pradesh and Bihar. These three States put together produce about 50 per cent of total inland fish production in the country, while West Bengal along accounts for about one third of the production.

National Water Policy (2002)

The ten page document is covering the following topics:

1) Need for a National Water Policy
2) Information System
3) Water Resources planning
4) Institutional mechanism
5) Water Allocation Priorities
6) Project planning
7) Ground Water development
8) Drinking Water
9) Irrigation
10) Resettlement and Rehabilitation
11) Financial and Physical Sustainability
12) Participatory Approach to Water Resources Management
13) Private Sector Participation
14) Water Quality
15) Water Zoning
16) Conservation of Water
17) Flood Control and Management
18) Land Erosion by Sea or River
19) Drought Prone Area Development
20) Monitoring of Projects
21) Performance Improvement
22) Maintenance and Modernization
23) Safety of Structures
24) Science and Technology
25) Training
26) Conclusion

Hydro-electric Power Development

While hydropower is one of the cheapest, and cleanest source of power and therefore generally the most desirable, the share of hydropower in the overall power generation has been decreasing over the years. There are several reasons for this. Even now there is about 80 percent of the hydro-electric power potential which needs to be tapped.

According to hydraulic characteristics, there are two types of hydro-power developments – storage and run of the river. Where storage capacity is available to regulate the river flow, power plants are generally located adjacent to the dam. In run of the river plants, there is some sort of a water diverting structure, a conduit to carry water to the turbines and a housing for the equipment of the power house. Trash racks and gates are placed at the head of the water intake structure; and a forebay is provided to effect regulation.

The basic difference between the two types is with regard to storage only. Whereas the storage projects can cater for daily and seasonal fluctuations of water in the river, the run - of - the - river plants normally do not provide pondage to take care of more than 24 hour's fluctuations.

Some of these reservoirs are multi-purpose. Where irrigation is involved, power can be developed before water is used for irrigation. Power may be developed utilizing the head at the diversion structure, drops in canals or from the drops from high canals to distributaries at the lower level.

The amount of power that can be generated from a steady discharge Q with head H is app. given by

$$P = (Q*H)/12 \qquad (1)$$

where

P is the power generated in Kilowatts,

Q is the discharge in cusecs, and

H is the head available in feet.

For example, a drop of 12 feet in canal carrying 10,000 cusecs of water can be exploited to generate 10 MWs of power.

Plans for Development

Hydro-electric power projects need heavy financial investment, but once commissioned they have very low running costs, as the basic 'fuel' is free. India is fortunate in having the Himalayas, the highest mountain range in the world, which can be utilized for power generation and other consumptive uses of water.

In the past various factors such as the dearth of adequately investigated projects, environmental concerns, resettlement and rehabilitation issues, land acquisition problems, regulatory issues, long clearance and approval procedures, power evacuation problems, the dearth of good contractors, and in some cases, inter-state issues and law and order problems have contributed to the slow pace of hydropower development. There have been large time and cost overruns in case of some projects due to geological surprises, resettlement and rehabilitation issues, etc. However, considering the large potential and the intrinsic characteristics of hydropower in promoting the country's energy security and flexibility in system operation, the Government is keen to accelerate hydropower development.

Most of the above concerns are being addressed through a number of legislative and policy initiatives at the central and state levels. As discussed in detail in the report, these include preparation of a shelf of well-investigated projects and streamlining of statutory clearances and approvals, establishment of independent regulatory commissions, provision for long-term financing for projects, increased flexibility in sale of power, etc. In May 2003, the Prime Minister of India launched a 50,000 megawatt (MW) hydro initiative. Under this scheme, detailed project reports (DPRs) are being prepared for 73 schemes, which have an indicative first year tariff below Rs 2.50.

This would provide a shelf of fairly well investigated low tariff projects to prospective developers. Risk perceptions in taking up the projects and the possibilities of time and cost overruns are also expected to get minimized. Of these schemes (total about 32,000 MW), 70 are located in the Brahmaputra, Indus and Ganga basins in the north and north-eastern part of the country.

The Government has formulated a number of measures to address the issues related to watershed management of upstream and downstream, coordinating the activities of the state government and the generating companies, the techno-economic clearance and the optimal development of the river or its tributaries consistent with other requirements.

The Ministry of Environment and Forestry would look into the environmental impacts and social/community development aspects associated with the projects and the developers would be required to deposit adequate funds for compensatory afforestation, catchment area treatment plan, wildlife management plans, biodiversity conservation plans, etc.

Private sector participation has been low in the hydropower sector although the sector was opened up in1991 since the investors looked at it as a higher risk proposition compared to thermal projects. As indicated above, the Government has initiated a number of policy measures to address such concerns.

India needs to mobilize large finances for implementation of its power program. While the Government has substantially stepped up its budgetary allocations to the hydro sector, support from international agencies and the private sector is also needed. India has been cooperating with Bhutan and Nepal in hydropower development for over a decade. There are prospects of further enhancement for the benefit of all the countries and in the larger interest of energy security in the region. Some prospects of hydropower cooperation with other neighbouring countries are also indicated.

Geotechnical Problems of Water Resources Development

A large number of water resources development projects are located in the Himalayan Region. Before we proceed with the actual problems and their solutions, let us look at the formation of the Himalayas itself and the problems arising out of it.

The Himalayas

The plate tectonic theory postulates mobility of thick crustal plates. Inter-plate collision results in release of energy causing earthquakes and mountain formation at plate boundaries. The collision plate boundaries can be identified by high concentration of seismic energy release reflecting the local disturbances resulting from crustal destruction and mountain formation. Such boundaries are also associated with zones of high temperature and pressure.

The Himalayas form the North-Western boundary of the Indo-Australian plate. This is a continent to continent collision boundary, the delineation of which runs along the axis of the Himalayas. The trailing margin forms the Nabas mountains along the boundary of the Eastern States of India and Myanmar. In the West, a trailing edge turns southwards to form the Hindu Kush mountains of Afghanistan. The highest concentration of seismic energy release occurs at these syntaxial bends.

This continental collision margin has produced some of the highest mountain ranges and deepest valleys on any of the continental plate boundaries. The mountain also acts as a rain divide, the rainfall decreasing as one moves to the west from a maximum in excess of 3000 mm (in the foothills of Nepal, Bhutan and Darjeeling), some 80 percent of which is confined to the four monsoon months and is sometimes of cyclonic intensity. Snowfall tends to conserve some of the rain; which otherwise runs off this steep terrain resulting in recurrent flooding on reaching the plains. Erosion can be considerable.

The Sun Kosi in Nepal, west of Darjeeling has been recorded as discharging silt at an average rate of 3100 cubic meters per square kilometers. The ganges is said to discharge the equivalent of 900,000 tonnes of silt daily into the Bay of Bengal.

Such is the general make-up of an area which has a potential abundance of hydro-electric power by virtue of the available water and enormous drops from which the water can be fed into turbines.

The distribution of seismic energy release, the thrust faults, and the contorted and fracturous nature of many rock sequences is compatible with the concept of the Himalayas being formed by an active continent to continent crustal collision.

As a result of the fracturing and failure, which is still active, the once horizontally bedded rocks forming the crust prior to collision are observed to be distorted and fractured. Major thrust faults contain glide plains along which the continent movement has occurred, permitting rocks of different ages to slide past one other over considerable distances, thus accommodating the continued collision. Such thrust faults are known at several locations, striking along a series of ever rising mountain peak ranges, and are associated with many faults and shear zones. The main Boundary Fault extends along the foot hills in an almost unbroken sequence from the north –eastern states of Meghalaya and Mizoram to Kashmir.

Geotechnical Problems Associated with the Himalayas

In addition to such major failure planes and associated folding and faulting, the rocks can be extensively fractured or jointed, due to the excessive forces which produce the mountain building. In certain brittle rocks such as dolomites at Salal project (J&K), this fracturing can be penetrative.

The in-situ stresses remaining locked within this mountain system can be expected to be very variable in magnitude and direction over an extended site. The attitude relate on a macro scale to the thrust fault systems. A simple vertical and horizontal distribution of the in-situ stresses may not be generally applicable.

As these mountains are forced up to increasing heights and steeper angles, the valley sides fail by toppling rock falls and rock slides provoked by the action of rain, snow and ice. At all stages the situation reflects a delicate balance between the tendency for mountains to be forced up, as a result of displacement of the ground by failure, and the continued degradation by landslides, sustained by removal of the resulting debris by the river systems. Some of the in-situ slopes can be considered to exist at factors of safety of 1 or less. The rocks, too, are very varied across the Himalayas. Starting with the youngest deposits along the foothills, which are sandstones, mudstones and conglomerates of the Siwaliks, one passes to the ancient Archean crystalline rocks of the iner Himalayas. Entering the mountains from the plains the sequence of rocks exposed is not continuous. Certain intermediate strata lie hidden beneath over-riding older rocks.

Type of Geotechnical Problems Faced by Indian Projects

The problems of the water resources development projects in India can broadly be divided in the following categories:

Foundation Problems

The foundation problems particularly for barrages and concrete dams have caused cost and time over-runs on several projects. The problems can be further classified into the following sub-categories.

Large Alluvium Deposits

Due to the easily erodible strata, the rivers have been changing courses and it is not surprising to find buried channels or huge deposits of gravelly bouldery material brought up by the river, below the foundation level. It is sometimes difficult to remove the pervious material to get to the rock strata and therefore elaborate arrangements are required to make the foundations and the nearby strata impermeable. Deep grout curtains or positive cut-off diaphragms are generally recommended. A look at the cross section of Baira dam site provide the example (Figure 1).

The size of the barrages in the Himalayan region have also been increasing ever since. This involves very deep excavation and stabilization of slopes. Problems associated with de-watering are always there. A typical example is that of the barrage at Sikkim.

Typical Design and Photographs.

Design Methodology and Stage Stability studies required.

Weak Zones

Shear zones or fault zones in the foundations are something very typical of the Himalayan region. Right from the Bhakra dam, where a major foundation problem was posed by the steeply dipping shear zones and claystone bands creating problems of settlement, there is hardly any dam where shear zones have not been encountered. Some typical examples are discussed in the following sections.

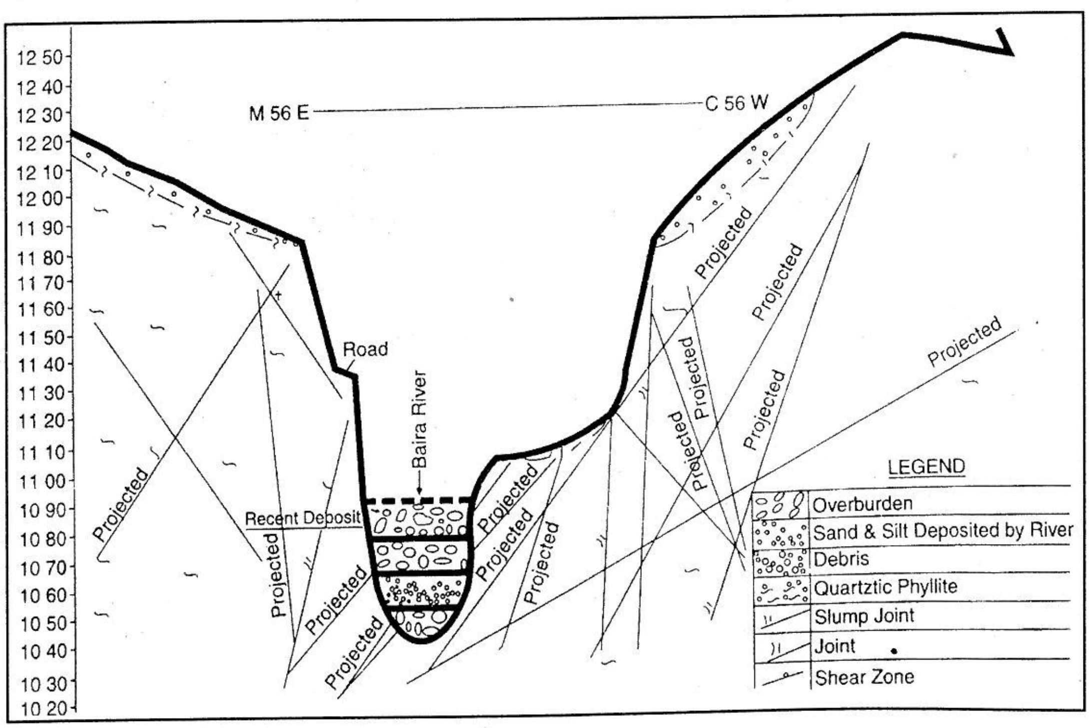


Fig. 1 Typical Section of a Valley along the Dam Axis in the Himalayan Region

Bhakra Dam

One of the highest dams of the world at the time of its construction, Bhakra dam is 226 m high straight gravity dam. The dam is located on thick sandstone bands inter-bedded with bands of siltstone and claystone. The beds dip at an angle of 70 deg. In the downstream direction and strike slightly skew to the axis of the dam. Major foundation problems were posed by the following adverse geological features in the foundation:

> steeply dipping shear zones and claystone bands posing problems of shear and settlement.

> Gentle downstream dipping cross shear zones in the abutments which posed problems of shear, settlement and seepage by-passing the key section in the abutment.

> Highly fractured and jointed nature of the foundation which posed problems of uplift pressures.

As a remedial measure, the load was transferred to the adjoining competent sandstone members by replacing the intervening weak zones of claystone and sheared sandstone with concrete down to adequate depth based on formula used for Shasta dam. The heel claystone was removed to a depth of 42 m below the river bed by mining methods and back-filled with concrete. Further, a concrete strut of adequate thickness spanning the clay band was provided for transferring the load to the member exposed upstream of 'heel' claystone. The 'heel' claystone exposed on the valley slopes in the reservoir was provided with concrete cover to confine the member. The axial shear zone was removed to a depth of 1.5 to 2 times the width and back filled with concrete

Plug tunnels of 2 m X 2 m in size and up to 15 m to 30 m depth following the cross shear zone were provided into the hill to confine the crushed material and act as shear keys and increase the path of percolation. Such drifts were provided at the axis, toe of the dam and at intermediate spacing of about 22 m apart. In addition, the shear zones in between the consecutive plug tunnels were washed, grouted upto a depth of 22 m and the surface trace of these was further excavated and provided with dental fill of the concrete.

Salal Dam

At Salal dam site, the Chenab river flows around a ridge called Dhyangarh loop and a 100 m high rock fill dam across the upstream limb has been constructed.

The spillway and intake structure for the power house are concrete gravity structures and are located on a ridge between the two limbs of the river (Figure 2). The

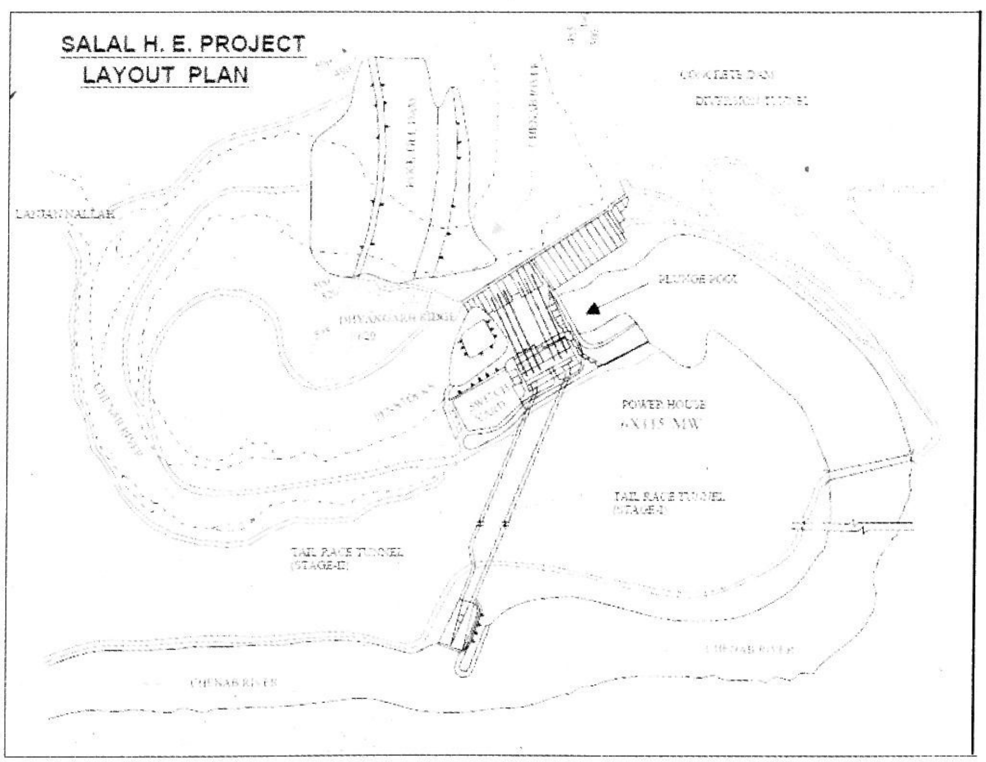
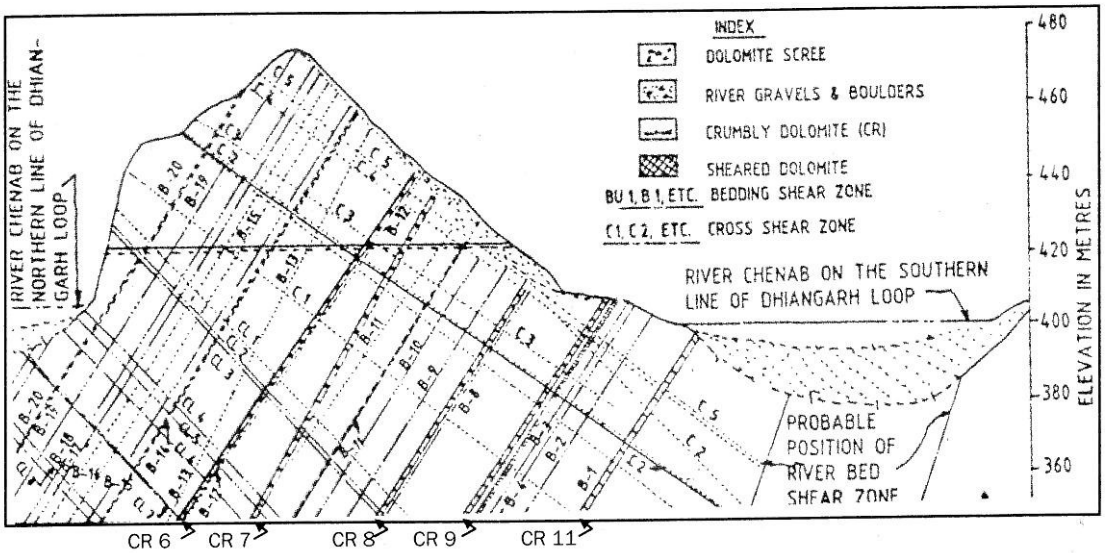


Fig. 2 Layout of Salal Hydroelectric Project



Cross Section of the Ridge

Fig. 3 Section of the Ridge Showing Downstream Dipping Shear Seams

ridge is made up of jointed dolomites. It was realized during investigations that whereas most of the bedding shear zones in the dolomite ridge were dipping upstream, a set of cross shear planes were dipping downstream creating the problem of sliding (Figure 3).

The geologists at site had also projected a wide shear zone in the river bed both in the upstream and downstream limbs of the loop. It has been the experience that the shear zones in the Himalayan region are sometimes wide, but the properties of material within the zone varies. The entire shear zone material is seldom 'tooth paste like'. Whereas some of the shear zones contain gougey material, others contain only sheared rock pieces. Majority of shear zones contain a mixture of crushed rock and gougey material.

It was difficult to establish the properties of shear zone in the river bed. This would need excavation of a drift through the river bed which would take considerable time. The design of the concrete dam could not be finalized till some of these issues were resolved. It was, therefore, decided to base the designs on indirect tests conducted on the shear zone material made in a drift through the extension of the zone in the left abutment. The in-situ static modulus was found to be moderate (0.5 GPa). Seismic wave velocity measured across the river from bank to bank gave high velocity indicating either the narrowness of the width of the zone or its not so weak characteristics. Large scale in-situ shear tests in drifts indicated an angle of internal friction along cross shear seams of the order of 41 degrees. Examination of the sheared surface in these test indicated that the failure was taking place by cutting across of rock in appreciable portion due to asperities.

These investigations gave the confidence that the dam could be built founding on the last day-lighting seam and the concrete strut could be provided across the shear zone if and when met with after excavating the bed downstream as shown in the Figure. Finite Element analysis carried out indicated that the factors of safety across the assumed planes of failure could be brought to the acceptable values and the dam design was finalized along these lines. The provision of concrete around the penstocks cutting across the valley and a compacted backfill buttress beam towards the abutment were the additional measures provided (Figure 4). The time required for these studies and the climate of uncertainty created during the period caused considerable delay.

Some of the Projects Left Abandoned

Maneri Bhali Stage II - Lakhwar Dam

Lakhwar dam and underground power house were under construction on river Yamuna before the project was abandoned. Diversion tunnels and stripping of dam foundations had been completed. Approach adits to the underground powerhouse had been excavated and the excavation of the power house was under progress. The dam was slated to be a concrete gravity dam of 204 m height above the deepest foundation level. At one stage the proposal to construct an arch gravity dam was also under consideration.

The dam was located on a 300 m wide band of trap rock which as per the geologists, was present there as the result of a freak accident of nature. Underlying the trap, however, were thinly foliated slates at a depth of about 30 - 40 m below the toe of the dam. The contact of slates and trap is dipping at an angle of about 45 degrees in the upstream direction and there exists a shear zone 1.5 m thickness containing gouge material at the contact. The slates and the gouge material were the source of weakness in the foundation material.

Proposals were considered as to whether the weak zone could be excavated and back filled with concrete, or the alignment of the dam shifted upstream in the centre giving it curvature and designing it as an arch gravity structure.

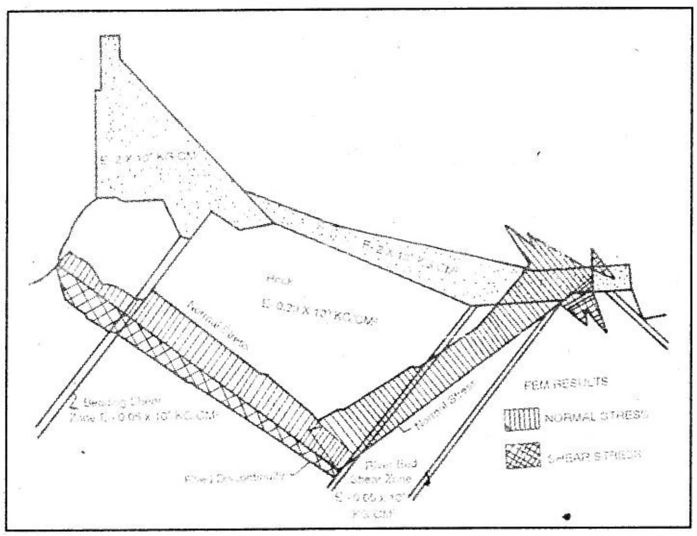


Fig. 4 Provision of Struts in the Foundation of Salal Dam

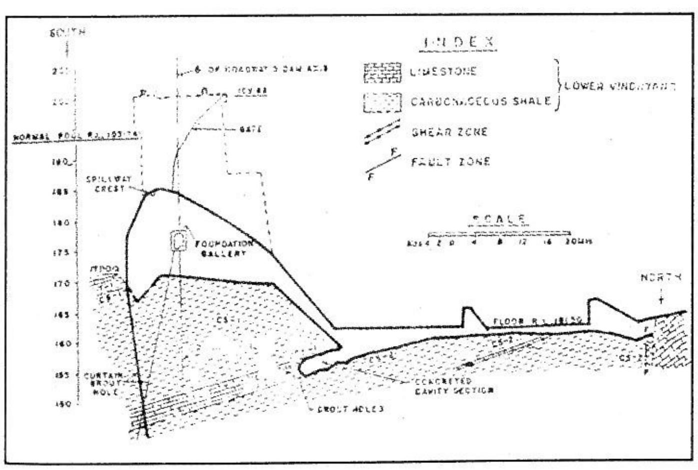


Fig. 5 Cross Section of Obra Dam

Cavernous Limestone

A 23 m high earth and rock fill dam was constructed across the river Rihand to generate 99 Mw of power for peaking purposes and to provide cooling water to Obra Thermal Power Station (Figure 5). The foundation rock at and around the dam site comprised inter-bedded sequence of limestone and carbonaceous shale. There were problems related to high permeability of cavernous limestone in the foundation and to that of sandy overburden overlying the dam section on the river bed portion. Also there was problem of stability of spillway portion founded on shale underlain by cavernous limestone. The problem of permeability was solved by providing a set of concrete diaphragms and grout curtains. One of the grout curtains was about 700 m long for which 500 holes had to be drilled and 2,35,000 bags of cement of 50 kg. each were consumed.

For the stability of the spillway, the altered portion of the limestone was completely removed, its cavernous part underlying the spillway was partly mined and partly back-filled with concrete to provide support to the shallow cover of the overlying shale member. Beyond the concrete plug, the limestone was grouted up to the limit of cavernous zone.

Foundations with Sliding Problems

Sometimes the dam foundations comprise of horizontally stratified formations which are weak causing the dams to slide unless some strengthening measures are taken. Two examples are presented, one in which no such measures were taken and the dam decided to walk off immediately on first filing. The second case shows how this could be done by taking corrective measures.

Tigra Dam

Tigra dam is located at a place close to Gwalior in Madhya Pradesh and was constructed for the first time in the year 1917 for supplying water to the town of Gwalior.

The dam was built as a gravity dam made of brick-work with on a sandstone foundations without any contraction joints. The designer did not take the uplift forces into account for this approximately 25 m high dam. The dam or the 'wall' was 1341 m long, and the artificial lake contained 124 millions m water. Well known engineer Mokshagundam Visvesvarayya was also involved in the construction and its subsequent rebuilding in the year 1929.

During the first filling of the artificial lake, the concrete dam broke. The dam failed in sliding and moved downstream before breaking apart. Some parts of the dam can still be seen standing in the downstream area (Plate1). It was one of the largest disasters claiming more than 1000 lives.

The causes of failure were thoroughly investigated by the British and American experts besides Indian engineers. Uplift was considered as the main cause of failure. Construction of the dam resumed and completed in the year 1929, but continued to have the problems till now.

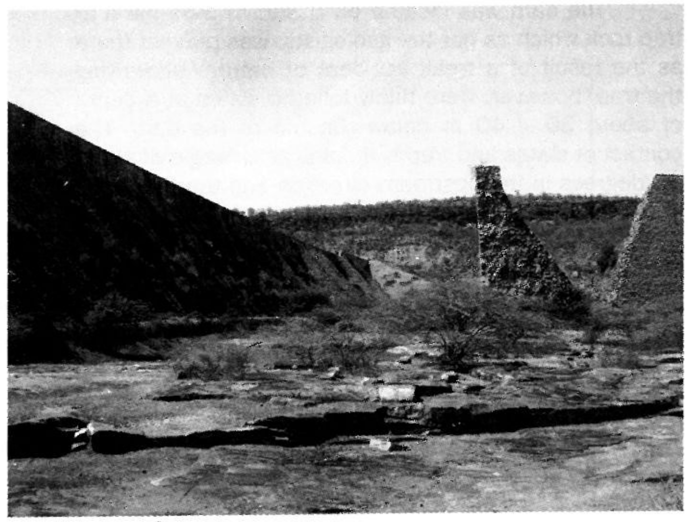


Plate 1 View of the Existing Tigra Dam with the Parts of old Failed Dam

Omkareshwar Dam

Built across river Narmada the Omkareshwar dam consists of 28 concrete blocks designated in the direction from left to right bank as blocks 1 to 28. Blocks 5 to 26 are spillway blocks with 5 m wide pier at the centre of the blocks. As the foundation rocks comprised of quartzites and siltstones/shale layers with some clay filled bedding seams a number of field shear tests were conducted. Based on the test results the following values of shear strength parameters were taken for the calculations.

Shear Parameters

Rock to Rock parameters

Cohesion 0.079 MPa

Friction Angle 37 degrees

Rock to Concrete Interface

Cohesion 0.55 MPa

Friction Angle 49 degrees

Because of the poor rock concrete interface values it was decided to provide shear keys of 8.5 meters depth on the downstream side. Factors of safety provided in Table 2 were obtained with and without the shear keys taking the passive resistance of rock into account.

Numerical Model

Just to ensure that the studies carried out manually were on the right track, numerical models of the dam with and without the shear key were prepared and studied for both seismic and static conditions. The finite element models without and with shear key are shown in Figure 6 and Figure 7 respectively.

Table 2 Factors of Safety with and without Shear Key for Omkareshwar Dam

Sl. No.	Loading Combinations	Factor of Safety			
		No Shear Key		Shear Key	
		rock to concrete	rock to rock	Rock to concrete	rock to rock
1.	B - Reservoir Full	2.20	0.96	2.75	1.50
2.	D - Reservoir at Full Discharge Gates Open	1.52	0.55	2.09	1.12
3.	E - Reservoir full with earthquake (design based, dbe)	1.68	0.71	2.17	1.20
4.	Reservoir full with earthquake (max. credible, mce)	1.77	0.64	2.44	1.31
5.	Reservoir full- earthquake (dbe) & discharge 10000 cumecs	1.62	0.68	2.12	1.18

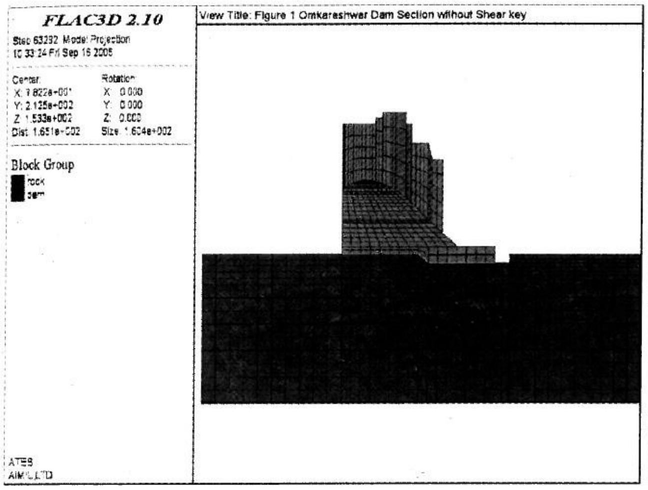


Fig. 6 Section of Omkareshwar Dam without Shear Key

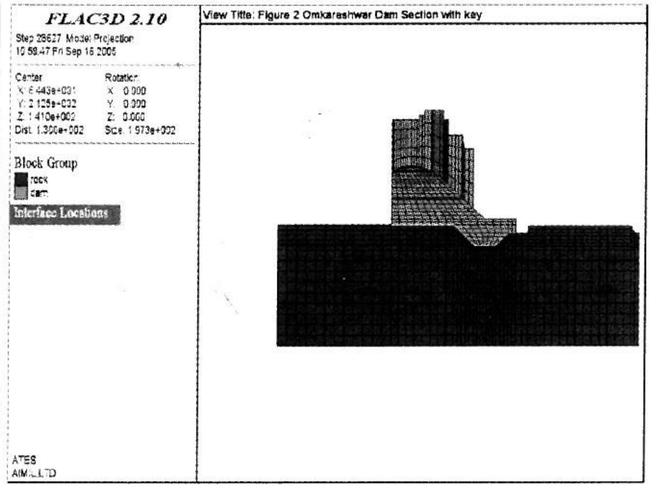


Fig. 7 Section of Omkareshwar Dam with Shear Key

Figures 8 and 9 show the stresses and the shear displacement along rock concrete interface respectively in the section with the shear key. Figure 10 shows the results of the field shear test conducted at site.

It was seen that whereas the dam section without the shear key was unsafe against sliding. Fortunately the data from a field shear test conducted on weak layers was available and it included the displacements in the horizontal direction as well. One of the problems that commonly arises in numerical modeling is the choice of parameters kn and ks, the normal and shear stiffnesses. A model of the shear block test was prepared and run with various combination of values of kn and ks. The horizontal shear displacement received with the models was compared with those actually obtained in the field and the values of kn and ks giving the closest fit were chosen for the model of the dam.

The maximum compressive stresses, the tensile stresses and the shear displacements for the different cases were found as given in Table 3

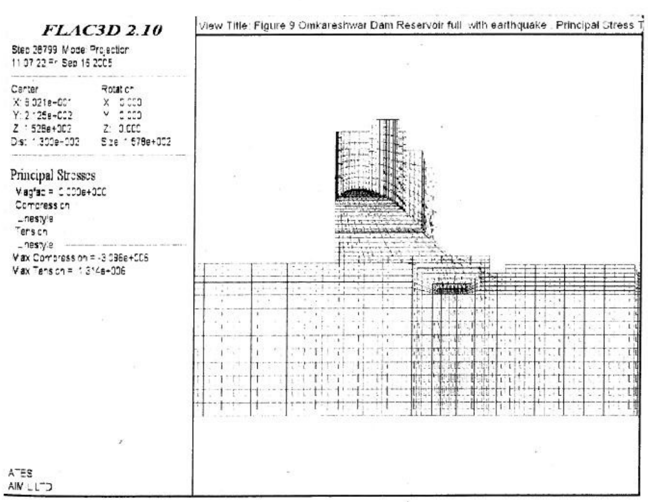


Fig. 8 Typical results of Stress Analysis and Shear Displacements

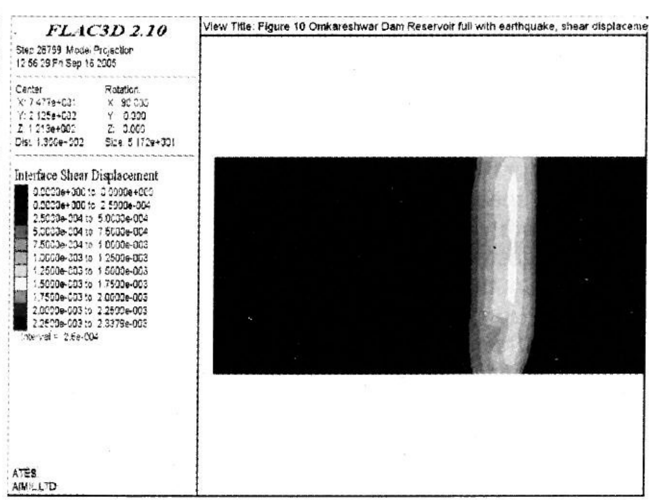


Fig. 9 Shear Displacement along the Rock Concrete Interface

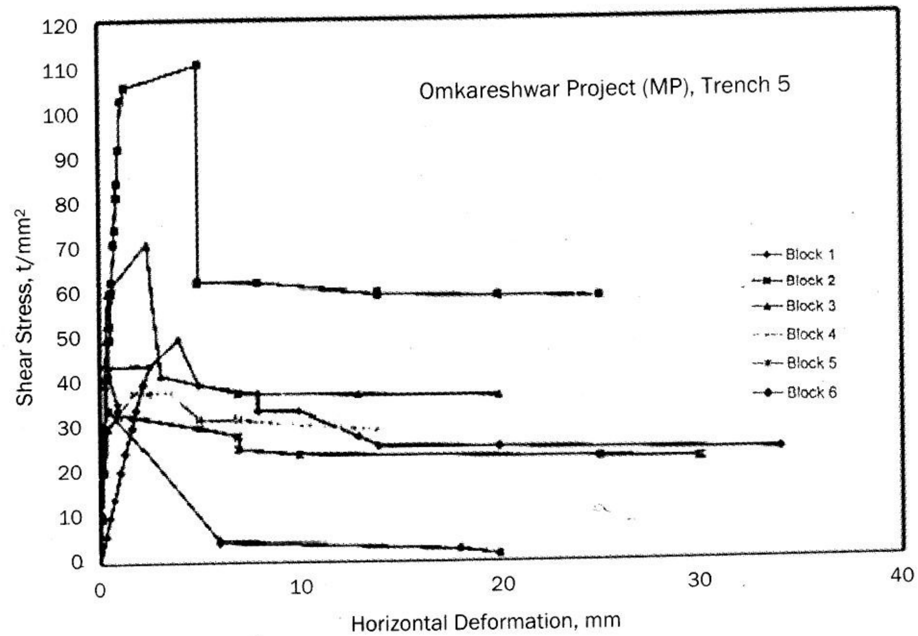


Fig. 10 Results of in-Situ Shear Tests Conducted on Dam Foundations

Table 3 Maximum Compressive Stresses, Tensile Stresses and Shear Displacements for Omkareshwar Dam

Sl. No.	Loading Combinations	Maximum Compressive Stress	Maximum Tensile Stress	Maximum Shear displacement
1.	Full Reservoir, No Earthquake, No key	2.3 MPa	1.2 MPa	1.10 MPa
2.	Full Reservoir No Earthquake With Key	2.38 MPa	1.17 MPa	0.74 MPa
3.	Full Reservoir With Earthquake No Key	3.012 MPa	1.53 MPa	5.14 MPa
5.	Full Reservoir With Earthquake With key	3.096 MPa	1.31 MPa	2.33 MPa

Dam Foundations with Shear Zones

It is difficult to imagine a dam foundation in India without a shear zone. The presence of shear zones become more important in the case of concrete dams. It was suggested at the time of construction of Shasta dam in USA that the weak zone material should be excavated up to a depth and then filled back with concrete. These recommendations were based on studies carried out using a Begg's deform meter and a photo-elastic bench and thus were based on the theory of elasticity. The empirical formula developed on the basis of theses studies was popularly known as the Shasta formula and was slightly modified in the case of Bhakra dam to get a new name 'Bhakra formula'. According to Shasta formula, the depth up to which the weak zone material should be excavated and filled back is given by,

$d = 0.002\ b\ H + 5$ for $H > 150$ ft.

$d = 0.3\ b + 5$ for $H < 150$ ft.

where d = depth of excavation of weak zone

b = width of weak zone

H = Height of the dam

For clay gouge $d > 0.1\ H$

Bhakra Dam

Bhakra dam was the highest dam in India when it was constructed. It is a 234 m high concrete gravity dam and it had several weak zones in the foundations when the foundations were opened up. Figure 11 shows the section of the dam.

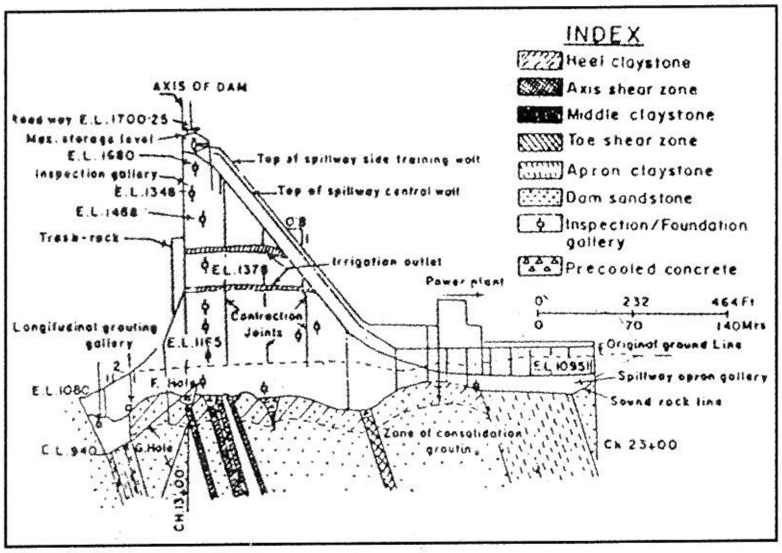


Fig. 11 Cross section of Bhakra Dam in Punjab

Indirasagar Dam

Indirasagar dam across river Narmada is a 92 m high concrete dam which is 653 m long. It is located about 10 km away from the Punasa village in Khandwa district. It is slightly curved in plan and houses 8 units of 125 Mw each to make up a total of 1000 Mw.

There was a shear zone in the blocks 14 and 15 of the dam foundation

Numerical studies were done to see the validity of Shasta formula and see if the three dimensional analysis would make any difference. The moduus of deformation of the shear zone was considered as 1/100th, 1/10th and 1/5th of the modulus of deformation of the parent rock.

It was observed that when the modulus values of the shear zone are low, the rock on the sides of the shear zone take the load, that is, very little load is allowed to come on the weaker material. The stress pattern shows some sort of arching taking place. As the modulus values of the shear zone begin to go up, the load on the shear zone also starts to go up. The trend continues and the maximum sharing is seen when the modulus value increases to a value 1/5th of the modulus value of the rock. Figures 12 to 15 show the results.

The figures show that as the modulus value of the shear zone starts to increase, the shear zone also starts to participate and sharing the load. Studies were also carried out to see if

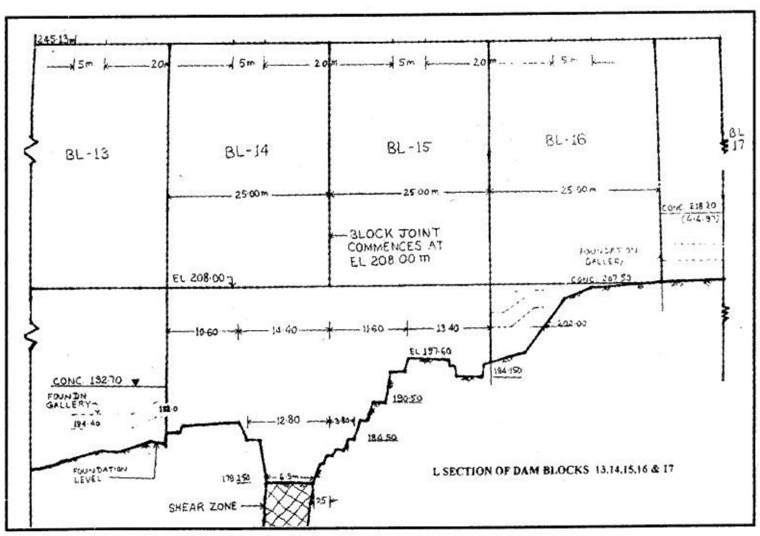


Fig. 12 Showing the shear zone in the block 14 and 15

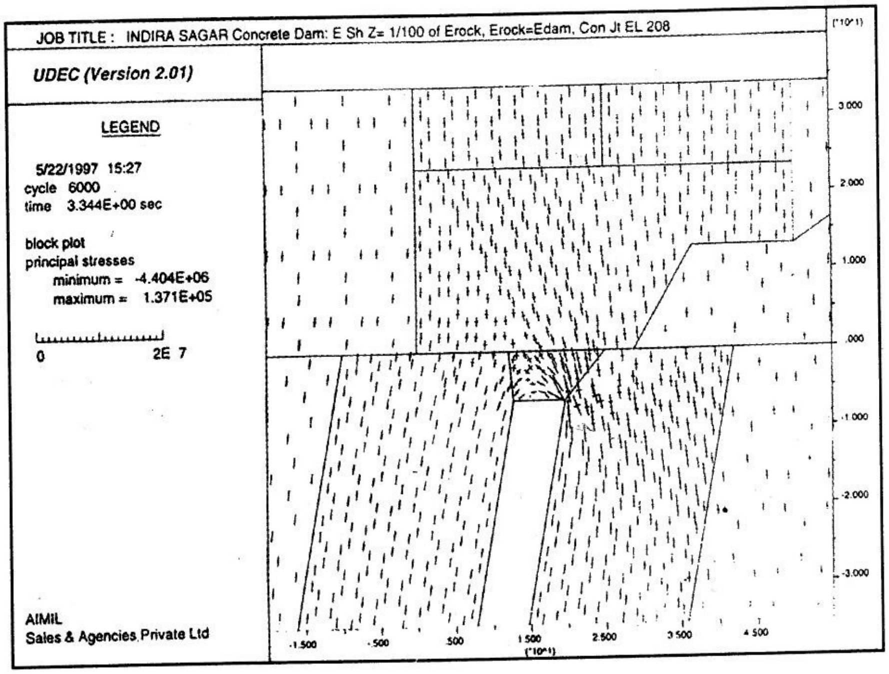


Fig. 13 Stress Distribution when
The Modulus Value of the Shear Zone is 1/100th of the Rock

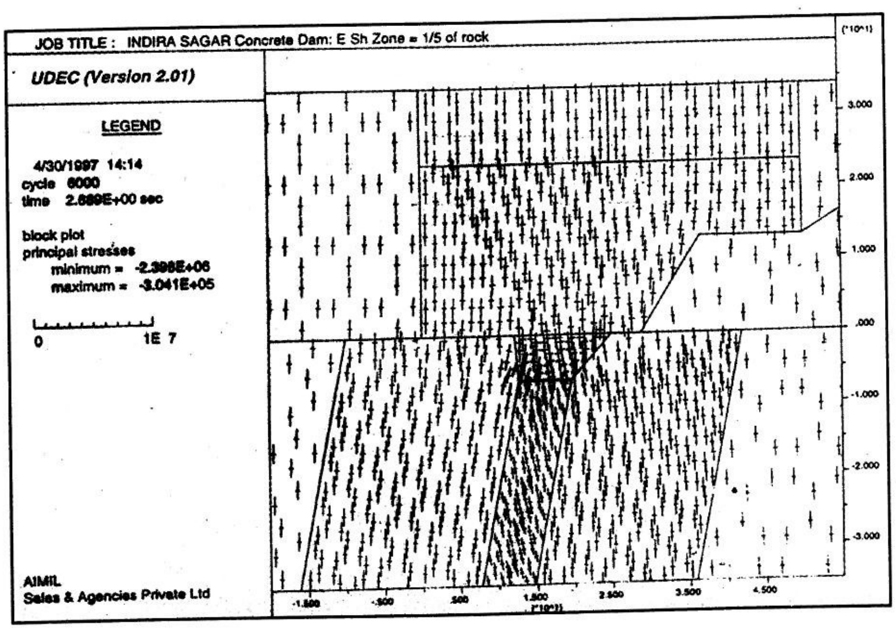


Fig. 14 Stress Distribution when the Shear Zone Modulus is 1/10th of the Rock

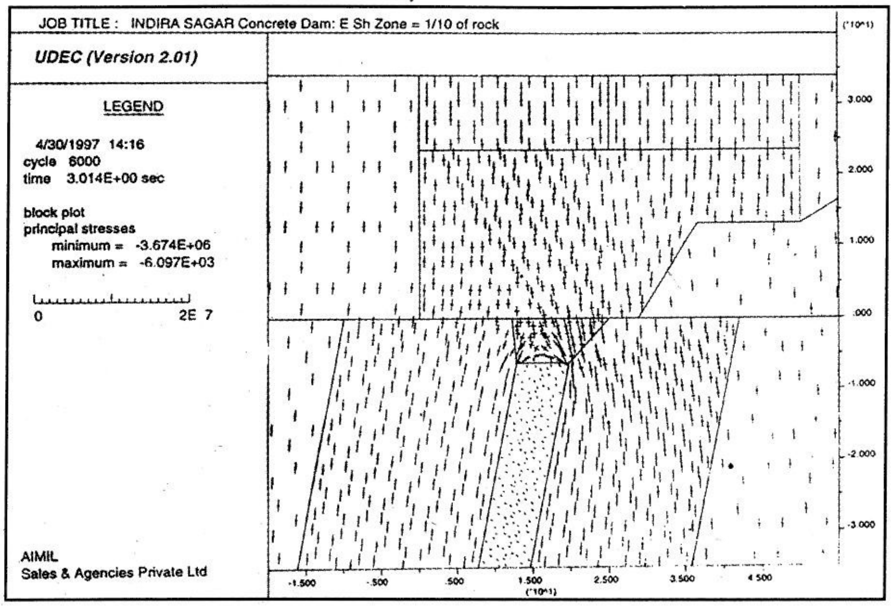


Fig. 15 Stress Distribution when
The Modulus of the Shear Zone is 1/5th of the Modulus of Rock

there was any advantage gained if the depth of the excavation and thus the depth of the shear key was increased. It was seen that the increase in depth beyond what was calculated on the basis of Shasta formula was not required.

A three dimensional analysis of the shear zone was also carried out taking the skew-ness of its orientation into account. It was seen that since the orientation was essentially perpendicular to the axis, the three dimensional analysis also did not make much difference. Plate 2 presents an areal view of the Indirasagar Dam.

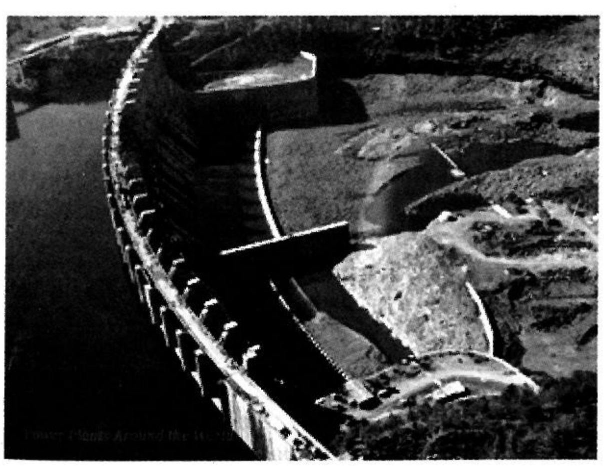


Plate 3 An Aerial view of the Indirasagar Dam

Baglihar Dam

Baglihar Hydro Electric Power Project is a run of the river power project on river Chenab in the district Doda. The project was investigated in the Seventees, but formally proposed in 1992, approved in 1996 and the construction began in 1999. The first phase of the project was completed in 2004.

The project is known more for the controversies between India and Pakistan, rater than the technical problems it faced. One of the problems it faced was concerning shear zone it had in the river bed which was very wide. Shasta formula is essentially meant for shear zones which are small in width compared to the size of the blocks of the dam. Normally the block width is about 15 meters and if the width of the shear zone is less than a meter or so, the Shasta formula is applicable. But the width of the shear zone at Baglihar was about 30 meters as originally conceived. It turned out a little lesser, on excavation, that is about 24 meters. Even 24 meters was a lot compared to the block width of 17 meters.

To solve this problem, it was decided to join the three blocks - two adjacent and the one in the center coming over the shear zone, and not have any contraction joint up to a particular level. The joints were provided only after that such that the stress distribution and the arching effect was complete by then. In a way it was a combination of the shear key and the pedestal which were used for stress distribution, above which the conventional dam was sitting. The studies were carried out using a numerical model in which several parameters and properties were changed to see their effect on the final results. A view of the dam is given in Plate 3.

Subansiri Dam Project

The Subansiri Lower HE project (8 x 250 MW), situated on river Subansiri in Arunachal Pradesh/Assam, owned by NHPC is presently under construction. As a component of the project a concrete gravity dam is being constructed. The height of the dam is 133 m. The dam spillway section is catered by 9 blocks (S1-S9) for which radial gates are proposed for the spilling arrangement. The anticipated maximum flood is 37,500 cumecs. The dam comprises of a total of 16 individual blocks of 19.5m width. The maximum height of the concrete gravity type dam is approx. 133m.

The base width of maximum overflow section is 160m approx. Each spillway block with crest level at EL 150m consists of 11.5m x 14.7m Sluice Type opening followed by Energy Dissipation structure, which is Ski-jump with Performance Plunge Pool.

As the permeability of the sandstone formations in the foundation were of higher permeability than normally acceptable, it was decided to provide a 50 m deep positive cut-off running from one abutment to the other by making a diaphragm wall. A D shaped gallery of 7m x 7 m has been provided for this purpose which will be plugged after the construction of the diaphragm wall is over. An adit of the same height will provide access to this gallery. It was apprehended that the junction of the adit and gallery may develop stress concentration such that it may exceed the allowable limits.

It was decided to carry out a three dimensional (3-D) stress analysis of a critical block of the gravity dam, identified as spillway block S4, which envisages the intersection of the access gallery and cut off wall gallery. The primary objective was to study the stability at and around the intersection of the galleries, subjected to various loading conditions which are expected to occur during the construction and operation of the dam. Also the stability of dam has to be studied in case of galleries plugged, when the dam is constructed up to an elevation of EL 170.0m as well as the when the construction of the structure is over.

As a result of this construction process, several intersections of galleries are created in the respective blocks. One such intersection is occurring in the maximum overflow block S4 at the floor elevation, which has been identified as the critical block subjected to extreme loading condition. These galleries have to be plugged with plum concrete after the construction of cut-off wall. However, the construction of the dam structure without plugging the galleries is a matter of concern, as the size of galleries is large enough. Hence the stress

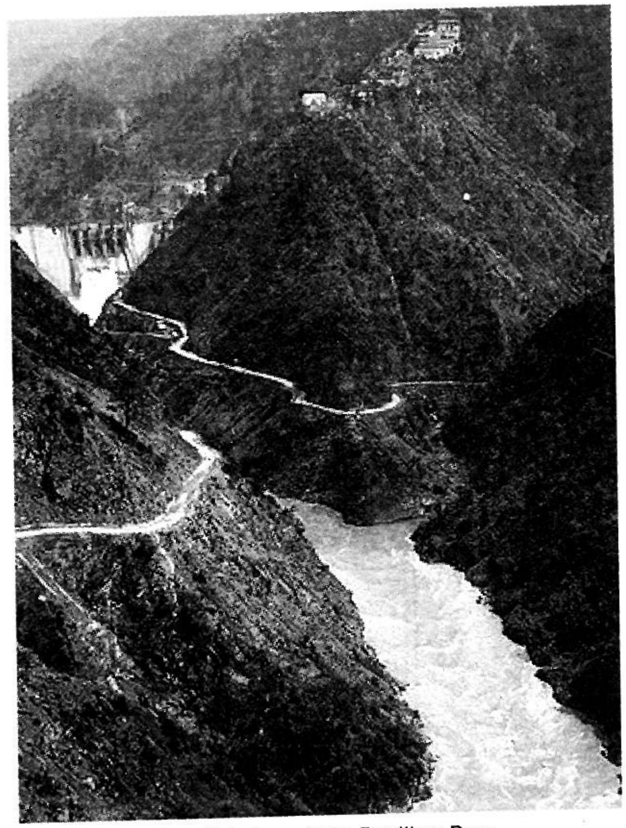


Plate 3 A view of the Baglihar Dam Across Chenab

concentration in the structure has to be checked under the impact of various load combinations suggested by the code.

As the concrete gravity dam is a massive structure with most of its body lies under compression, except sometimes a slight tension on the upstream dam face and at some other isolated locations depending upon the geometry of the structure. The stresses at any horizontal plane can be estimated through conventional analytical approaches (Limit Equilibrium Approach), under appropriate loads applicable to the dam.

The galleries of small sizes do not have much effect on the overall stability of the gravity dam. However, larger galleries when kept close enough to any of the face of dam are always a matter of concern. This can adversely affect the stability of dam by creating highly stressed zones in vicinity of the dam face resulting in the cracks through which seepage can occurs.

Since the spillway dam block S4 was identified as the critical block it was studied in detail using 3-D Continuum Analysis, modeling the cut-off wall gallery, the access gallery and their intersection along with few small galleries.

In the analysis, the dam block & its foundation was modelled using FLAC 3D, simulating the material model behaviour when subjected to the extreme loading combinations as applied in manual calculations. The stress distribution at critical locations and specifically around the galleries intersection zone was studied.

Also the dam was analyzed for its stability and stresses, when it reached the height of EL 170.0 m, along with the galleries in unplugged and plugged state. Further, the behaviour of complete dam structure with the plugged cut-off wall galleries was also studied, under different loading conditions.

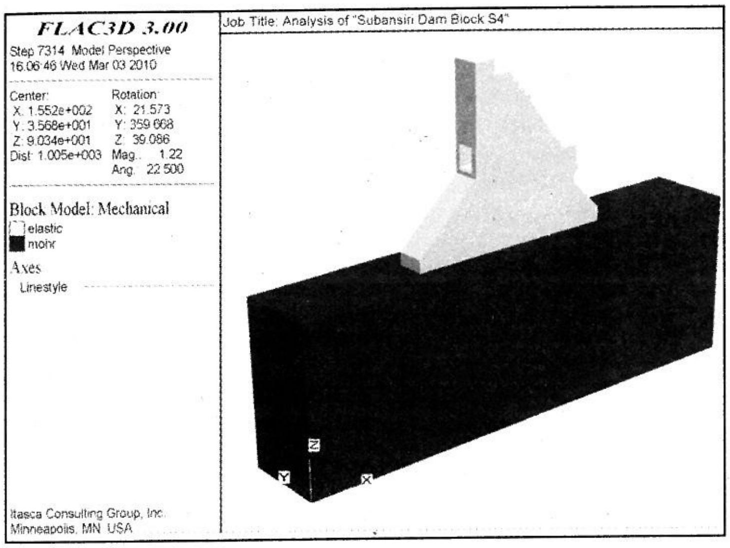


Fig. 16 A Section of the Spillway of Subansiri Project

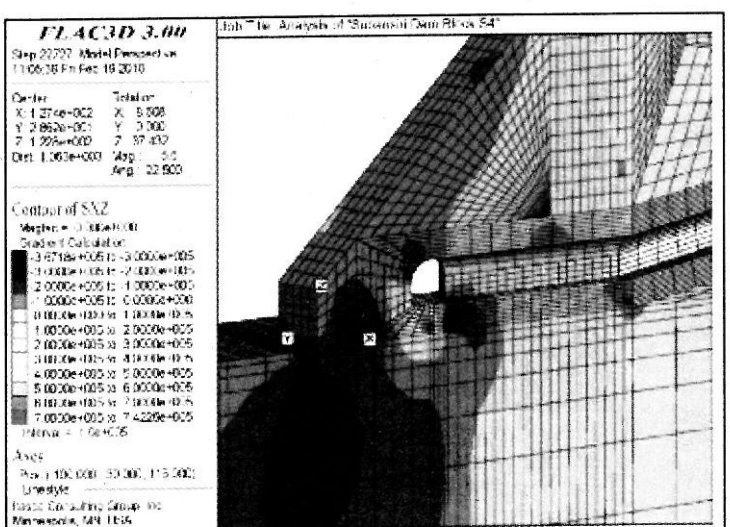


Fig. 17 Section of the Dam though the 7 m x 7 m Diaphragm Gallery and the Adit showing Stress Concentration at Different Locations

The continuum analysis of the structure was performed using FLAC 3D software. FLAC 3D is a powerful finite difference based analytical tool, capable of modeling complex geometrical shapes with typical material responses. The material is analyzed assuming it to be a continuum, i.e. homogeneous and isotropic and subjected to a boundary condition and various load combinations.

The zones under maximum stress concentration, different strength failure state, displacement vectors, principal stress tensors and various displacement and stress history plots could be visualized along any reference plane.

The stress contours around each gallery, along and across its respective axes could also be visualized to determine the stress magnitude at any desired locations. On the basis of these studies the stability of the structure was assessed. Figures 16 and 17 illustrate the results of the analyses.

Slope Stability

The steep mountains rising to great heights made up of comparatively weaker material cause problems of slope stability. As soon as it is disturbed, whether due to road construction or any other activity, it starts a perpetual process of road blockages due to rock falls and landslides. Several landslides are triggered with the onset of monsoons. Reservoir rims are potential source of failure when the reservoirs are impounded or suddenly depleted.

A number of roads are required to be constructed on water resources development projects for the haulage of materials and equipment. Some of these roads are temporary, but some are permanent. Some of the existing roads come under submergence, and alternate roads are required to be constructed in lieu of the roads getting submerged. All these roads have problems relating to stability as they are either in cutting or in filling – either way there is a problem. Plate 4 shows the failing slopes above the diversion tunnel intake and the road cuts in the slope at Tehri Dam Project.

A conventional way of stabilizing these roads has been to construct small toe walls or retaining walls. In my opinion, nothing could be more un-economical and therefore wasteful expenditure than these measures, which are taken on an ad hoc basis. Sometimes rock bolting with wire mesh and shotcreting is also done, but in a way that leaves much to be desired. The geotechnical investigations, testing both in the laboratory and in the field is rarely

Plate 4 A View of the Slopes above the Diversion Tunnels of Tehri Project

carried out. No analysis is carried out in most of the cases, with the result that the stabilization measures fail much earlier than they are supposed to. Examples of slope stability problems and measures taken on some of the water resources development projects are briefly described below.

Tehri Dam Project

Slopes in the vicinity of diversion tunnels and the dam site of Tehri Project have been treated extensively. In fact this is seen almost on all projects in this region that a huge effort and a large amount of finance is required before the slopes around the dam site can be made safe and stable.

The treatment of the slopes generally comprise of deep anchors combined with shallow anchors or fully grouted rock bolts with wire mesh and shotcrete. Drainage forms an important component of the total stabilizing scheme. Slightly upwardly inclined holes are drilled for this purpose and perforated pipes with filter packed in a geosynthetic sack are inserted into those pipes.

Instrumentation forms an integral part of the scheme to give an advance indication of movement as well as any stress relaxation on the anchors taking place. Slope movement in the horizontal direction is picked up by inclinometers which are sometimes automatic 'in-place', and read manually at other times.

An aerial photograph of the terrain shows the softness of the strata of the lower Himalayas and the erosion caused by the falling rains. As soon as the rivers hit the plains and slopes flatter, the silt and debris starts to deposit and the rivers meander. Plate 5 shows one such aerial picture

The movement of water level in the reservoir also causes the slopes to fail, particularly if the depletion is fast, not allowing the pore pressures to dissipate. Plate 6 shows one slope in the Tehri reservoir which has failed. Plate 7 shows the elaborate stabilization measures taken at Tehri Project.

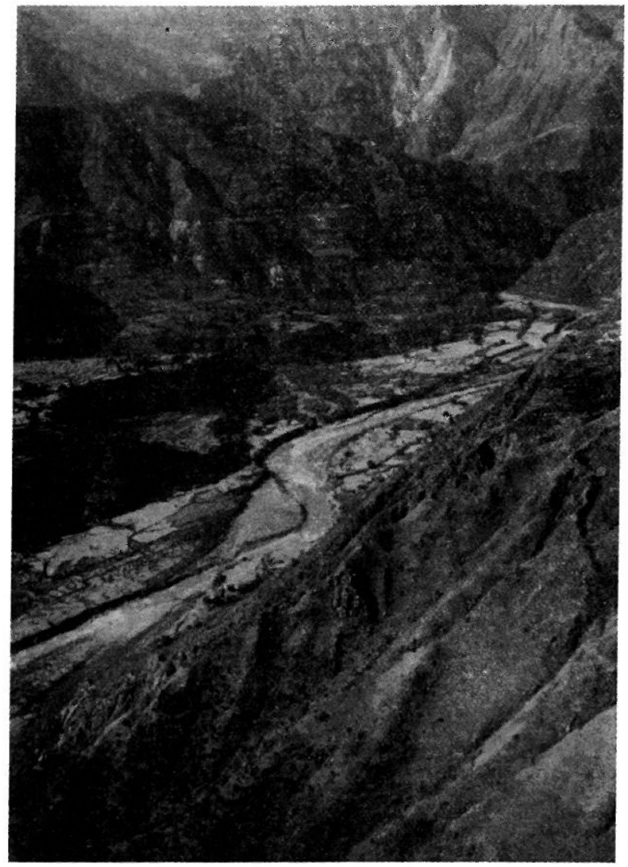


Plate 5 Soft Formations of the Lower Himalayas and the Meandering Rivers

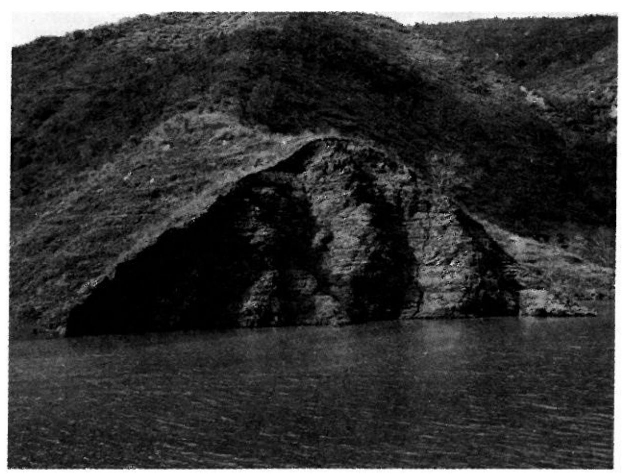


Plate 6 Failing Slopes in the Reservoir of Tehri Dam

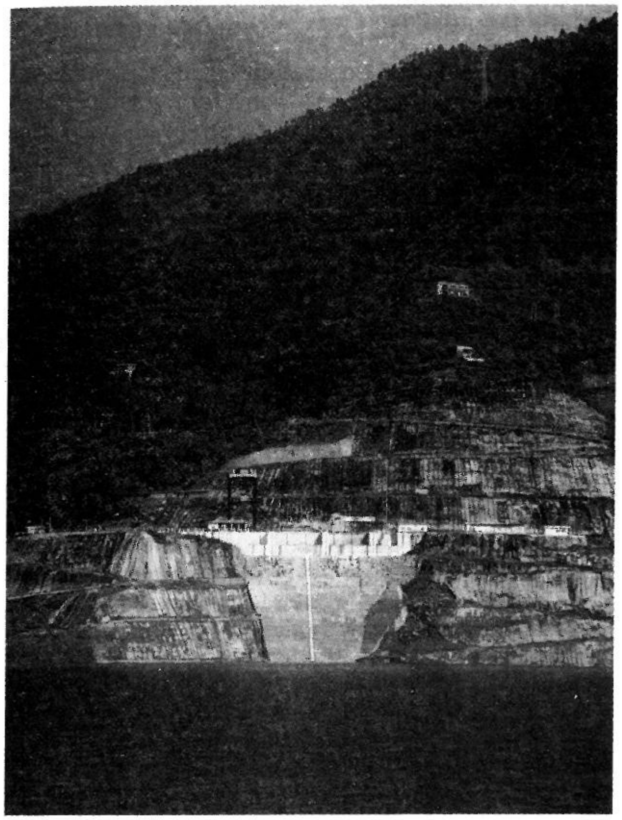


Plate 7 Extensive Slope Treatment on Tehri Abutments

Similar treatment has been tried at Lakhwar dam site and the power house of Giri Bata Project in Himachal Pradesh.

At Baira Siul Project the creeping hill slope consisting of the phyllite rocks above the powerhouse remained unstable for a long time. One of the reasons why these hills behind the penstocks become unstable is that there is a head race tunnel culminating in a surge shaft before the penstocks take off. Tunnel linings crack invariably and the hill starts getting saturated. Sometimes he hill creeps and tends to push everything towards the river. Any treatment carried out on the surface is of little value. What is really required is a very elaborate drainage arrangement which could even take the shape of a tunnel with radial holes filled up with pea gravel to enlarge their zone of influence.

A large scale landslide occurred on the right bank hill slope just downstream of the proposed axis of the baira dam. The landslide filled up the river by about 10 m along its length upstream of the dam axis. In fact this is a common occurrence on several projects and the reasons are different. One common reason is cutting the toe of the slopes dipping towards the river, for widening the road or some other similar reason. This happened at Nathpa Jhakri Project where a huge slide blocked the river and threatened the functioning of a Sanjay Vidyut Pariyojana immediately upstream of it.

Koteshwar Dam

Koteshwar dam is being constructed just downstream of the Tehri dam and would not only generate power from its own head but will also act as a part of the Pumped Storage Project. It is about 100 m high concrete gravity dam which is nearing completion. It will generate 4 x 100 Mw power from its own head and if the reservoir is used as a Pumped Storage it would be able to generate another 1000 Mw of power.

The dam site has faced problems of slope stability on the banks as well as in the power house pit. Plate 8 shows the extensive treatment of the slopes which has been carried out. Following instrumentation has been suggested to keep an eye on any untoward movement so that timely action could be taken to control it.

1. Piezometers for measuring pore water pressure
2. Inclinometer to measure the movement in horizontal direction
3. Multiple Borehole Extensometers to see if any movement is taking place along the slopes
4. Load Cells for measuring the load on the anchor
5. Surface Targets to se if there is any movement on the surface

If the remedial measures are not worked out properly, failures are likely to occur. Plate 9 shows an ad hoc arrangement done and the resulting failure.

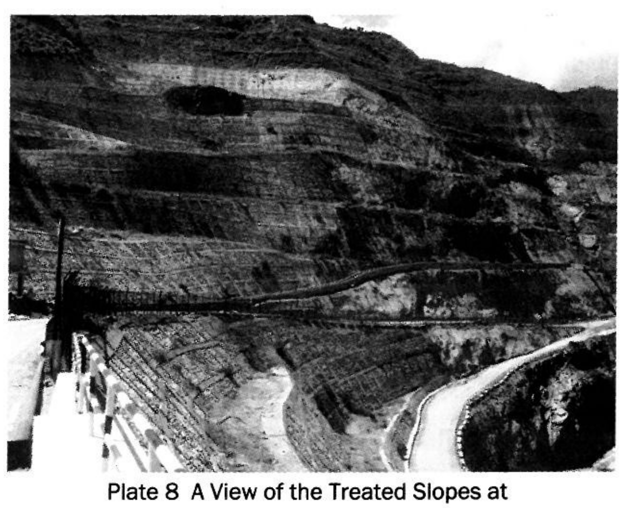


Plate 8 A View of the Treated Slopes at Koteshwar Dam Site

Plate 9 A View of the ad hoc Stabilization of Slopes on River Bank and Subsequent Failure

River Blockade due to Landslide

Maneri Bhali Hydro-Electric Project Stage I harnesses the power potential of water flowing down the river Bhagirathi between Maneri and Uttarkashi. The project works include construction of 8.63 km long, 4.75 m diameter concrete lined tunnel, a concrete dam an intake cum sedimentation chamber and a surface powerhouse.

For the construction of the main diversion dam, intake structure and the sedimentation chamber etc., a diversion tunnel of 7.0 m diameter and 139 m length was constructed on the right bank of river Bhagirathi.

Cloudburst and Landslide

A sudden cloudburst took place one evening at an upstream place about 30 km away from the dam site called Kanodia Khud. A huge land slide followed the rains. Landslide mass contained uprooted trees along with the debris of soil and rock. This created a blockage in the river. The normal flow of the river at that time of the year was substantial, but following the blockage, the flow was reduced to a mere trickle. When the engineers at site noticed that the flow in the river had considerably reduced, they investigated and found that there was very little water coming from upstream. Whatever little water was coming in the river was from another Nala called Maneri Gad. The district authorities, the contractors, the public works department and other concerned authorities were also informed about the blockage.

Due to the general alert sounded by the district authorities, irrigation and hydel departments and the contractors took necessary precautionary measures to save the men and machinery from the floods which were imminent.

Precautions Taken

The Hindustan Construction Company which was constructing most of the civil works for the project, took precautions to remove their construction plant and machinery from the vicinity of the river bed. Work was stopped and people staying near the river banks were evacuated. The Department also took care to remove equipment which was meant for installation in the power house. Even the spiral cum sped ring weighing as much as 35 tonnes was removed to a safer place. The diesel power house which was to supply the construction power was blocked all around by sand bags to prevent the water from entering.

Flash Flood and the Damage

Efforts were being made in the mean time to blast the blockage to breach the dam created by the landslide. As a result thereof, the dam breached the next day late in the afternoon. What was not anticipated at that time was the fury with which the water will run down. As the dam broke, the water gushed out creating a monstrous wave front of about 5 to 6 meters height.

As the wave front moved downstream, it carried with it the uprooted trees and debris along with it, smashing everything that came its way. Roads on either side near Gangnani village were washed out. Five to six road bridges were washed out. The diversion tunnel was day-lighted for a length of about 40 m. large cavities had been created below the sill beam of the gate and also on both sides of the tunnel inlet. The flood waters found its way into the tunnel through thee cavities below the intake structure.

Since prior warning had been given and most of the people were prepared, no loss of life took place, even though the entire left bank of the river was washed out. The spillway glacis had already been constructed. Its face on the upstream as well as the downstream side was damaged due to the boulders brought by the river hitting the surface. Boulders as big as 1.5 m x 1.5 m to 2.5 m x 2.5 m were seen rolling over the glacis and passing through the diversion tunnel. There was not much damage to the intake structure of the sedimentation chamber as the trees blocked the passage of boulders etc.. At the powerhouse site the penstock pieces were shifted downstream. Slushy material filed all the block-outs left for construction, creating a mess.

Second Wave

However, the breach of the dam was not total and quite unexpectedly a second flood wave came down two nights later. Since most of the people in the downstream area had seen the flood two days earlier, they had become complacent thinking that the worst was over. This second flood wave came as a surprise and caused heavy loss of life.

Repairing the Damage

Following the damage to the diversion tunnel due to the flash flood, it was decided to create a barrier in front of the diversion tunnel by dumping crates filled with boulders, stones and sand bags. This had to be done before the arrival of the next monsoon flood. It was, however not found very effective and as this operation was going on, it was seen that a new cavity was formed on the right side of the inlet portal of the diversion tunnel, cutting off the road. The main cavity also started to widen after the monsoon set in and further damage to he road took place. It was then decided to divert the river over the spillway section. A diversion channel and a coffer dam was accordingly constructed. The bulk head gate from the tunnel was removed by cutting piece by piece, and the diversion tunnel was plugged by a concrete portions of the diversion tunnel which had day-lighted were converted into a compact backfill section with a concrete core wall. Rock fill material of the order of 54.50 million cubic meter was required for this purpose.

Penstock foundation movement due to the hill creep has been experienced on several projects. One of them is the Loktak Project in Manipur.

Loktak Project

Penstock slopes at Loktak Project, particularly around anchors 11 and 12 caused severe problems. Though rock was available at the powerhouse and at anchor 12, the rock levels were practically horizontal after a height of 12 m above powerhouse level. The overburden consisted of brownish clayey slope wash material for about 5m to 6m depth underlain by weathered rock material. At the time of laying the penstock pipes, the clayey material got saturated and when excavated for cuts required for the penstocks and the powerhouse, it triggered large slides on the right side of the abutment. A number of remedial measures were taken up. The slope was being continuously monitored when it was active.

The water conductor system of Loktak project had 2.31 km long open channel, 1.1 km cut and cover section, 6.6 km long head race tunnel, a 60 m high 9.15 m diameter surge shaft, about 0.3 km long steel lined tunnel and 1.3 km long penstock for each of the three units.

Problems were faced at the time of the excavation of the open channel due to sensitivity and the organic nature of the soil. In some reaches, extensive cracks developed longitudinally and there was heaving of channel bed by 1.5 to 2 meters. For the cut and cover section, bulking of struts was observed, and the construction technique had to be modified.

Soil Nailing Technique – Indigenous Improvisation

Instead of constructing a toe wall or making a retaining wall, it is always a good idea if the strength of the ground could be increased so that it is able to support itself. Soil nails or rock bolts do the same thing. Sometimes it is difficult to find a suitable material which can be used as soil nail particularly if the material is clayey as it would not take grout readily.

An attempt was made to improvise and make a slope stable in one of the projects in the Himalayas. Since the experiment was very successful, it was repeated at another project which was also in the Himalayan region. The technique is briefly described below.

Holes of 100 mm are drilled first and if required a casing pipe is used to keep the borehole intact. Once the borehole has reached its required depth, an anchor coupled with a perforated galvanized iron pipe of about 40 mm is inserted in the casing pipe. Thereafter, a mixture of sand and small size gravel is mixed and poured inside the pipe. If possible, the material which is placed in the annular space between the galvanized pipe and the casing pipe is compacted and simultaneously The casing pipe is pulled up to that extent. This process is continued till the entire casing pipe has been withdrawn and the sand gavel mixture filled up in the entire hole.

Neat cement is then poured into the galvanized iron pipe which is perforated. The cement flows out and mixes with the sand gravel mixture and ultimately forms a reinforced concrete pile which acts as a soil nail. Of course to know the stiffness characteristics, pull out tests in the field are carried out for use in the designs.

A number of software are now available in the market which have taken away some pain of the designer. It is possible to work out the loads and deformations of nails and the surface at any point required and matched with the instrumentation data.

Tehri Dam Project

The Tehri Dam is a rock and earth-fill embankment on the river Bhagirathi near old Tehri township in Uttarakhand. It is the primary dam of the Tehri Hydroelectric complex built by THDC India Ltd. Completed in the year 2006, the Tehri Dam withholds a reservoir of 2.6 billion cubic meters for irrigation, municipal water supply and the generation of 1000 MW of hydroelectricity along with an additional 1000 MW of pumped storage hydroelectricity.

After an initial visualization in the year 1949, a preliminary investigation for the Tehri Dam Project was completed in 1961 and its design was completed in

1972 with a 600 MW capacity power plant based on the study. Construction began in 1978 after feasibility studies but was delayed because of financial, environmental and social impacts. In 1986, technical and financial assistance was provided by the USSR but this was interrupted due to political instability. Government was forced to take control of the project and at first it was placed under the direction of the Irrigation Department of Uttar Pradesh. In 1988, the Tehri Hydro Development Corporation (THDC Ltd.) was formed to manage the dam and it was decided that 75% of the funding would be provided by the central government and the balance 25% by the state government. Uttar Pradesh would finance the entire irrigation portion of the project. In 1990, the project was reconsidered and the design changed to its current multi-purpose form, including a 2400 MW power production capacity. Construction on the Tehri Dam was complete in 2006 while the second part of the project, the Koteshwar Dam, is about to be completed. Work on the Pumped Storage part of the Project is about to begin.

The Tehri Dam along with the downstream Koteshwar Dam and the Tehri pumped storage hydroelectricity power plant will afford a power generation capacity of 2400 MW, provision of irrigation to an area of 270,000 hectares, irrigation stabilization to an area of 600,000 hectares, and a supply of 270 million gallons of drinking water per day to the industrialized areas of Delhi, Uttar Pradesh and Uttarakhand.

Tehri is the highest dam of our country. The salient features of the dam are given Table 4.

Table 4 Salient Features of Tehri Dam

Location	Downstream of the confluence of rivers Bhagirathi & Bhilangana in Uttarkhand
Type of dam	Earth & rock-fill embankment
Length	575 mtr
Height	260.5 mtr
Base width	1,128 mtr
Crest width	20 mtr
Type of spillway	Gated chute spillway & four tunnel spillway
Discharge capacity of spillway	15540 cubic mtr per second
Construction began	1978
Opening date	2006
Type of core	Inclined core

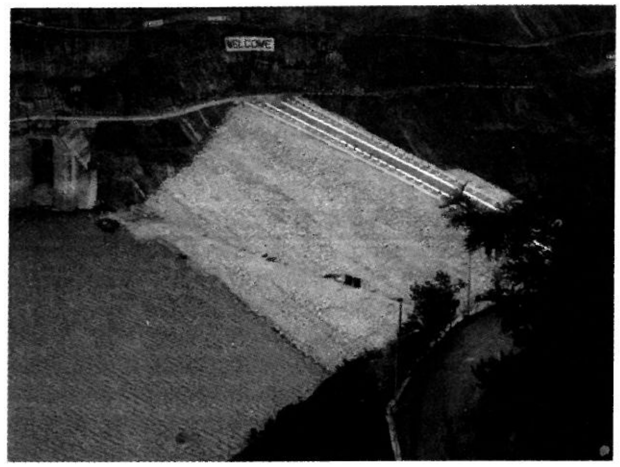


Plate 10 A View of the Earth and Rock Fill Embankment at Tehri Project

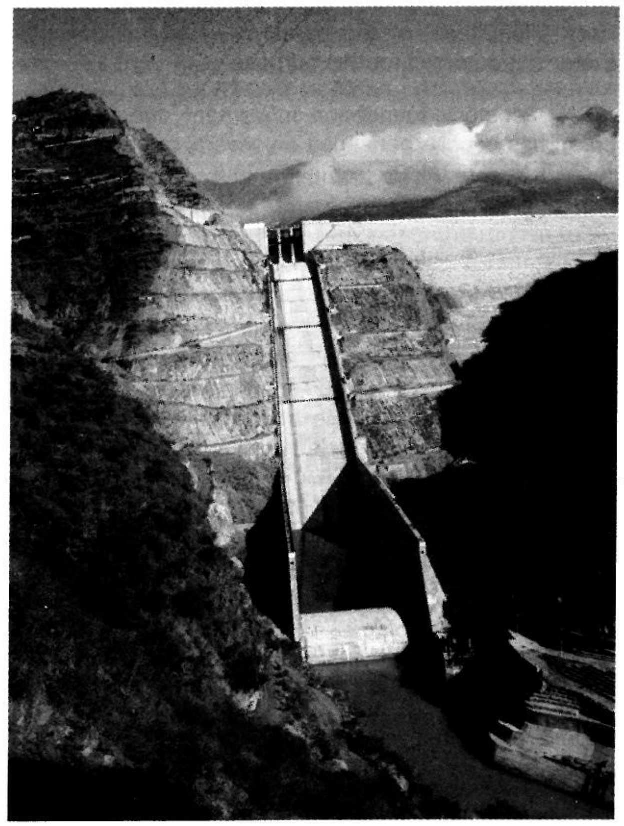


Plate 11 A View of the Chute Spillway at Tehri Dam Project

Plate 10 shows a view of the earth and rock fill embankment. Plate 11 shows the chute spillway from the downstream side. The picture also shows the arrangement for the dissipation of energy. Plate 12 shows the glory hole type of intake for the tunnel spillway.

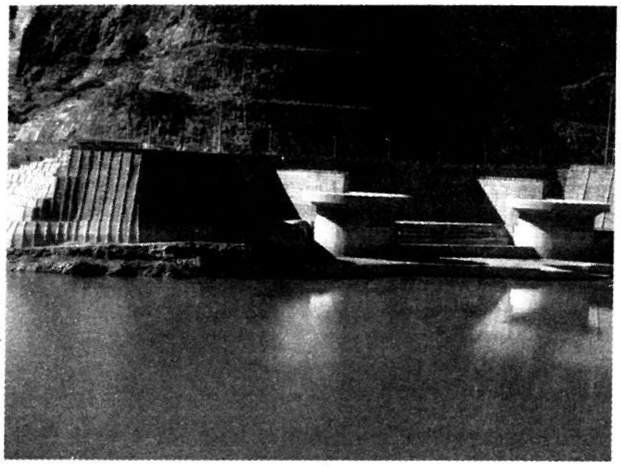


Plate 12 A View of the Intake Structure for The Tunnel Spillway

Some of the design features including its evolution in stages, and the concerns with regard to its seismic design are discussed below. This part of the write up is largely based on a paper written by B. L. Jatana who was involved with the design and construction of the dam for a very long time. His contribution is gratefully acknowledged.

Investigations and Studies for Project Formulation

Starting in 1960s, over a period of thirty years by 1990s, extensive surveys and investigations relating to topography, hydrology, geology, seismology, geo-technical, construction materials, hydraulic model studies were carried for dam and appurtenant works, first for project formulation and later on for the preparation of Detail Project Report, further for the preparation of detailed designs. Most of the national agencies like the Geological Survey of India, Indian Meteorological Department, Department of Earthquake Engineering, University of Roorkee, Roorkee, U.P. Irrigation Research Institute, Central Soil and Materials Research Station (CSMRS) were involved in various investigations for the project.

Alternate design options for the dam structure & appurtenant works were studied at the stage of project formulation. At the stage of firming of design, special investigations and design studies relating to project were undertaken with the assistance of Soviet Consultants during 1988-90. The Project engineers were advised by a Board of Consultants (BOC) constituted in 1970, comprising of experts including two well known foreign experts. Mr. Barry Cook of U.S.A. and, Dr. Mueller of Austria, who were members of the Board during initial 2 to 3 years; BOC rendered advice on project layout and finalization of initial designs for the project till 1984. In 1984-85, a Composite Team of engineers was deputed by the Governments of Uttar Pradesh and India, comprising of senior engineers responsible for design of Tehri Dam project in Irrigation Design Organization, Roorkee and senior engineers from Central Water Commission to have firsthand knowledge about the State-of-the-Art with regard to high dams and about actual performance of high dams built in France, Canada, U.S.A., Colombia, & Mexico. The team had extensive discussions with the leading Consulting Engineer Firms in these countries and exchanged views with them about specific technical issues relating to the design of Tehri dam and appurtenant works.

Geology of Dam Site

The rock formations at the dam site are phyllites of Chandpur series. The Phyllites in Tehri gorge are, in general, banded in appearance, the bands being constituted of variable proportions of argillaceous and arenaceous materials. Tehri phyllites have been classified as phyllitic quartzites, thinly bedded (PQT), phyllitic quartzites massive (PQM), quartzitic phyllites (QP) and sheared phyllites (SP). PQM and PQT are the most competent rocks and have been geomechanically grouped into grade-I phyllites, while QP and SP have been considered grade-II and grade-III in decreasing order of strength. The geometry, orientation, frequency and interplay of diagonal and longitudinal shear zones have considerably affected the geomechanical behaviour of the rock mass and provided a scope of dividing the area into different tectonic blocks. Tehri dam is seated on one such single tectonic block.

Seismicity

The Tehri dam project area is seismically active and falls in zone- IV of seismic zoning map of India. The Tehri region falls between two main regional tectonic features of Himalayas-Main Boundary Fault (MBF) on the south-western side and Main Central Thrust (MCT) on the north-eastern side. Besides these, there are some other tectonic features in the vicinity of dam site, the important among them being Srinagar Thrust, which has a strike continuation of over 100 Km and lies at a distance of 6 Km towards north from the dam site. The evaluation of data of past earthquakes has revealed that epicentres of about 80 earthquakes fell within radius of 320 km from the Tehri dam site, the nearest epicentre was located 35-40 km north-east of the project site. Most of these earthquakes have a magnitude of 5 to 7 (on Richter scale).

Dam Foundation

Tehri dam gorge has phyllitic rocks which have been classified into four categories: Phyllites Grade I constitute about 45% of the total rocks in the gorge; Grade II Phyllites constitute about 25% of total rock. Four major joint sets were observed in the Tehri gorge. In the early stages of investigations by GSI, based on geomorphic expressions and intensive sub-surface explorations by drilling at Tehri dam site, a N-S trending major tectonic dislocation was interpreted along the river Bhagirathi. An initial proposal of a concrete dam was discarded on this ground.

Subsequent explorations done failed to prove the existence of such dislocation. In 1987, for exploration under the river bed, a drift (E-W) was driven at a depth of 29 m (elev. 574.0 m) below the river bed for a length of 64 m, through an approach shaft located on the right bank, close to dam axis. Detailed examination of geological features in the drift revealed the absence of any major dislocation or shear zone along the river. Also hydro-geological conditions in the drift, further indicated the tightness of planes of discontinuities in the rock.

Construction Materials

Based on surveys, several borrow areas were identified for obtaining various materials for the dam fill. For clay core, borrow area terraces at Koti, Sirai, Dobra and Uppu were investigated. Koti borrow area was selected as the main source. Material at Koti terrace was an alluvial deposit of mixture of medium Plasticity clay, silt, sand and small quantity of gravel up to 80 mm size. For pervious dam shells, source was identified as gravel terrace at old and new Dobata areas. These deposits comprised of sand, gravels and boulders, upto 600mm size, with varying deposits of silt. No natural pervious deposits, satisfying the requirements of filters were available around project area. The material for filter material was, therefore, proposed to be obtained by processing of material excavated from Dobata borrow area. To improve the seismic stability of dam the material for riprap was specified as angular blasted rock fragments, instead of oversize boulders and excavated phyllitic rock. After detailed investigations and surveys to locate suitable source a quartzitic rock quarry, 25 km away from dam site along Bhilangana river at Asena was identified, for meeting most of the requirement for riprap; some requirement being met by breaking oversize boulders.

Evolution of Design Concepts for Tehri Dam

Initial Design: On the basis of investigations and design studies carried out in early 1970s, an Earth and Rockfill dam, with an inclined clay core was considered appropriate for Tehri dam site, from considerations of topography, geology, seismicity, availability of construction materials, safety and project economy. Based on conventional static and pseudo- static stability analyses, a 260.5 m (height as per deepest foundation level expected at that time), a zoned dam section with crest width of 20 m and upstream slope of 2.5(H):1.0(V) and downstream slope of 2.0(H):1.0(V) was adopted; the dam zones were envisaged as under:

Core: Mixture of medium plasticity clay, silt, sand and gravel and boulders up to 150mm size. Core with an upstream slope of 0.6 (H):1.0 (V) and downstream slope of 0.3(H):1.0 (V) giving a horizontal core width of 0.3 (H).

Upstream shell: Well graded terrace gravels comprising of sand gravels & boulders with silt content not exceeding 6%; maximum size up to 600mm; shell zone above MDDL(elev. 740.0) of the same specification as above, but silt free.

Downstream Shell: Well graded terrace gravels comprising of sand gravels and boulders with maximum size up to 600 mm.

Fine Filter: Well graded sand, with 5% silt.

Coarse Filter Including horizontal drain: Well graded mixture of coarse sand and gravels up to 80 mm size.

Upstream & Downstream Rip Rap: 10 m thick (normal to slope) zone of oversize boulders and phyllitic rocks with sizes up to over 600 mm; not more than 25-30% material smaller than 300 mm with minimum size restricted to 150 mm. Besides the two-layer filter provided down stream of core there was a provision for a transition zone, of fine filter gradation, between upstream shell zone and upstream face of core. A second layer of coarse material (corresponding to coarse filter gradation) was provided in between first layer and upstream shell from MDDL upwards up to crest level. The downstream filter zone, after its contact with foundation was continued all along the foundation horizontally up to dam toe to function as a drain.

The heel portion of upstream shell was designed to serve as upstream coffer dam, with provision of inclined clay core-u/s slope 1.55(H): 1.0(V) and d/s slope 1.25(H) : 1.0 (V), ultimately to be integrated with the main dam.

Changes in Design in 1980s

By mid 1980s, after further detailed investigation of borrow areas for dam fill materials and further design studies for the dam, including dynamic analysis, finalization of scheme for foundation treatment, and scheme for river diversion, changes were effected in the dam design. The changes proposed/ being considered were,

> Scheme for river diversion was finalized for a diversion discharge of 1 in 1000 year frequency.

> Scheme for foundation treatment and seepage control was finalized and curtain grouting was proposed to be carried out through an underground gallery below the river bed and inclined underground galleries along the dam abutments.

> Design criteria for filter was changed. A more conservative criteria based on concept of 'Perfect

Filter' (Vaughan's Concept) was proposed to be adopted.

> The material from Koti terrace comprised of top 6 to 8 m layers of medium plasticity silty clay (CL), followed by sandy gravels, down to bed rock. Tests had established its erosion resistance and it was capable of being compacted to high density at optimum moisture content (omc) in its natural state. One option was to use the material as such; other option considered was to blend the material with gravelly sand, to have a material, which besides having low permeability, would have low compressibility and high shear strength and would have lower probability of hydraulic fracturing. Project designers were having both these options under consideration.

One major issue which posed a very difficult problem with regard to construction planning was how to build upstream coffer dam in one working season (in time span of about eight months). Construction involved major activities of excavation of dam foundation, its treatment, consolidation grouting of dam core seat and raising 81 meter high (above river bed level) upstream coffer dam involving a fill volume of about 2.2 million cubic meters. To reduce the quantum of work for construction of upstream coffer dam, one option seriously considered was to reduce the coffer dam height, catering to diversion discharge of 1 in 100 year frequency, instead of 1 in 1000 year frequency.

Further Refinements in Dam Design

During 1987-88, as a result of 1986 Indo-Soviet Agreement on Economic assistance and Technical cooperation for construction of Tehri Hydro Power Complex, the design concepts for Tehri dam & Spillway structures were re-appraised with the Soviet Consultant (M/s Hydroproject Institute, Moscow).

After intensive interaction with the Consultant, the dam design was further fine tuned and firmed up. Following important changes were made in the design, as a result of the re-appraisal,

1. To reduce the quantum of work to be done during a single working season, it was decided to spread the construction to two working seasons.
2. For curtain grouting, a system of multi-tiered horizontal grouting galleries at interval of 60m, approached from downstream was adopted in place of going through a vertical shaft.
3. The consolidation grouting was also specified to be done from three tiers of horizontal underground galleries running parallel to dam axis, having cross galleries extending across the core width for consolidation grouting; upstream half of these galleries were to be plugged, after execution of work. Compared to normal practice of grouting from open surface, grouting through underground galleries increased almost five times, the drilling requirement. Yet this arrangement was opted for, as it enabled the dam fill to be placed independently of grouting operation, thus reducing the construction time; it enabled higher grouting pressures to be used near the surface, when carried out after partial fill placement.
4. It was decided to have natural material available from Koti terrace, blended with sand & gravel, for placement in the dam core as per the international practice.
5. Optimization of Core Geometry: Optimisation studies were carried out to determine the most optimal geometry of core, which would lead neither to formation of plastic zone under dynamic load nor to excessive transfer of stress from core to shell leading to possibility to hydraulic fracture. Geometry of core meeting this requirement was evolved and adopted.
6. Design of Filter: After considering the concept of perfect filters and carrying out laboratory tests on the line of Vaughan's method, and taking Sherard's recommendations into account, the gradation of filter, as finally adopted for Tehri dam had D 15=0.3 mm. This gradation was checked in laboratory under high head gradients and found satisfactory. The filter material was obtained by processing of excavated material from Dobata borrow area through screening/ crushing, so as to produce non- plastic, free draining and well graded material.

Test Embankments

In 1988, test embankments were constructed to determine the compaction and moisture conditioning requirements for dam fill materials proposed to be placed and to specify the type of compaction equipment required. In case of material for core, the requirement for blending course material, to achieve the core material gradation, recommended by the Consultant, was also determined. It was established that material of desired gradation is obtained by blending the overlying fine material at Koti terrace with coarse lower layer, in the volumetric ratio of 1.5 : 1.

Aseismic Design

Being located in the region of high seismicity, evolving a safe design, capable of withstanding strong motion during large earthquakes, has been a prime consideration, for the dam design since the very beginning. In the preliminary design studies, stability analysis for dam section for seismic loading was carried out by pseudo-static method, adopting a seismic design co-efficient of 0.12 g, based on recommendation of a

National Standing Committee on this subject. This was to be followed up with a rigorous dynamic analysis.

Dynamic Analysis by the Department of Earthquake Engineering

The detailed dynamic stability analysis was carried out by the Department of Earthquake Engineering (D.E.E.), University of Roorkee. This was done in four steps viz.

a) Establishing the seismicity of project area.

b) Determination of dynamic properties of dam fill materials.

c) Liquefaction studies for dam fill materials.

d) Carrying out dynamic analysis.

Inputs about seismic tectonic set-up at Tehri were provided by Geological Survey of India (GSI). Considering four sources for generating earthquakes around Tehri, MCT, MBT, HFT & Srinagar Thrust, estimates for effective peak ground acceleration for various source zones were made. In absence of record for strong motion for earthquake in the region, response spectra were selected, considering expected range of predominant frequencies and dynamic amplification for various levels of dumping for all the source zones and an envelope of this was adopted as response spectra for design. Artificial accelerogram was generated to match the slope of this spectra. For dynamic analysis, the value of dynamic shear modulus, at various shear strain levels, in absence of suitable laboratory equipment, were determined by carrying out field tests in the borrow areas. To assess the liquefaction potential of shell and core material due to excessive pore pressure build up during occurrence of estimated Maximum Credible Earthquake (MCE), studies were conducted with the help of a shake table. These studies confirmed that there would be no significant loss of strength for either of the material under M.C.E. condition. Dynamic stability analysis was carried out using the following approaches.

a) Plastic Displacement Analysis.

b) Linear gravity turn-on two dimensional Stress Analysis by Finite Element Method, both for static and dynamic conditions.

The plastic displacement for the dam was worked out for the two scenarios-

a) Dam behaving as rigid structure, and

b) Dam behaving as non-rigid structure.

The permanent crest settlement, worked out was found to be much less than the provision made for this purpose in the total free board provided for the dam. Dynamic Stress analysis indicated dam section to be safe from seismic consideration. Seismic risk at Tehri dam was also evaluated by the probabilistic approach, taking into consideration the data of earthquake occurrence from 1917 onwards. The analysis indicated that effective peak ground acceleration for 100 years service life would be 0.07g, 0.08g and 0.23g with an excedence probability of 0.67, 0.50 and 0.25 respectively i.e. there exists 67%, 50% and 25% chance that actual P.G.A. will exceed 0.07 g 0.08 g and 0.23 g in 100 years.

Dynamic Analysis by Soviet Consultants

In view of opposition of Environmental Groups to taking- up of Tehri project and concerns about the safety of dam during the occurrence of large earthquake in the Tehri region, Soviet Consultant (Hydroproject Institute, Moscow) were asked to review the seismic stability of Tehri dam based on State-of-the-Art methodologies/ techniques. Fresh studies were carried by the Soviet Consultant comprising,

1. Site specific assessment of seismicity.
2. Dynamic testing of material on large size dynamic triaxial testing machine, and
3. Sequential, non-linear elasto- plastic dynamic analysis of dam.

For site-specific assessment of seismicity, all existing available data about seismo tectonics of Himalayas, was studied and collated; information base was further strengthened by the study of satellite imageries of the region. Consultant carried out paleo-seismological assessments by field studies. Instrumental seismic microzoning studies were carried out to study the variation of seismic parameters of seismic effect along the height of Tehri canyon. Consultant's assessment was that estimate of Tehri dam area seismicity was optimum and conservative.

For the dynamic analysis, as actually carried out, accelerograms with PGA of 0.5 and 0.4 g at the base of dam, ignoring canyon effect. A State-of-the-Art, non-linear sequential (static as well as dynamic) stress analysis by Finite Difference method using Elasto Plastic model was carried out for the adopted dam section. This analysis takes into account the stress dependent properties of the fill material, the schedule for raising the dam, the schedule of reservoir filling, the consolidation of clay core, development of construction pore pressures and the effect of the permeability of various components.

Grouting and Drainage Galleries

A network of underground galleries- gallery below river bed with cross drifts, three tiers of galleries at elev. 640.0 m, 700.0 m and 760.0 m, along both left and right dam abutments have been provided for curtain grouting and for consolidation grouting of dam core seat below elev. 700 cm. The access to these galleries is through the approach cum drainage galleries on either bank from downstream. These approach galleries are also to function as drainage galleries for release of any seepage occurring through dam abutments, after building up of the reservoir.

Inspection Galleries in the Body of Dam

A novel feature of dam design is the provision of two Inspection galleries in the body of dam one in the dam core at elev. 725.0 m along center line of core, with a finished diameter of 2.2 m and other at e/n. 838.0 m, aligned on top of core- 2.5 x 2.4 m (finished internal dimensions), with open base, resting directly on top of dam core. The provision of such galleries was suggested by Soviet Consultant; Similar Inspection galleries, in the dam cores have been provided in Nurek and Charvak dams in U.S.S.R. (now in Tajikstan & Uzbekishtan respectively) and Aswan High dam in Egypt. 2 to 3 tiers of Inspection galleries were also planned to be provided in 335 m high Rogun Dam, U.S.S.R. (now Tajikstan) which was under construction in 1990 (work on which stands abondoned, since the break-up of U.S.S.R. in 1991). Provision of a gallery inside the dam core for Inspection was considered by many Dam designers, outside the Soviet Union, a rather daring practice in high embankment dams. THDC Design Group on detailed consideration, after inspection and seeing the performance of such galleries provided in the core of Nurek dam (300 m high), have incorporated these galleries in Tehri dam core to identifying possible transverse cracks, especially in the reaches close to abutments, at the dam top, due to tensile stresses induced by differential settlement. Studies done for Tehri dam had indicated that manifestation of tensile stresses would be more pronounced in about a fifth of the dam length at top near the abutments. While provision has been made in the design and construction for taking care of any such manifestation, visual observation through this gallery would make definite assessment about the occurrence or otherwise of the cracking, possible.

Dam Instrumentation

To monitor the structural behaviour of dam during construction and during operation, a well planned instrumentation network has provided in the dam to have a continuous assessment about its safe performance. About 350 nos. of instruments have been installed in the dam body and in its foundation, with arrangement for automatic data acquisition and retrieval, for measurement of pore pressures, horizontal and vertical stresses, deformations and settlements in the body of dam and at its surface, seepage through dam body and abutments. Details of various instruments and typical data obtained are briefly discussed below.

Instrumented Sections: It was decided that the instruments shall be put in the designated cross sections named as B-7, B-9, B-11, B-15, B-18, valley section etc.. A typical section of one of such cross sections is shown in Figure 18.

Hydraulic Piezometers: A total of 48 twin tube type hydraulic piezometers were istalled in the body of the dam. The numbers installed at sections B-7, B-11 and B-15 were 15,23 and 10 respectively. It is commonly considered that the difference between the

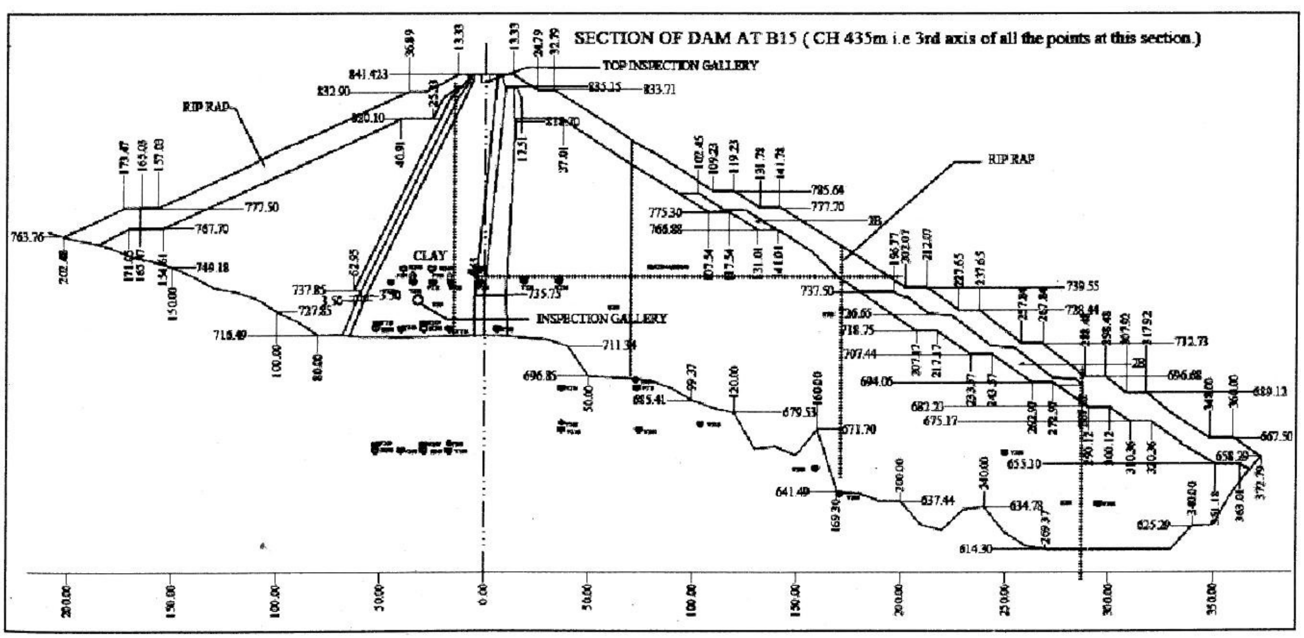


Fig. 18 A typical Section of the Dam showing location of Instruments

tip of the piezometer and the tubings or the read out should not exceed 5 meters. These instruments have to be read in the old ways and are not connected with the data loggers.

Vibrating Wire Piezometers: A total of 126 piezometers have been installed in the body of the dam. Their number at locations B-9, B-11, B-15, B-18 and the valley section are 39,57,31,01 and 28 respectively.

Vibrating Wire Thermometers: A total of 12 thermometers have been installed, out of which five are at section B-7, three at section B-11, three at section B-18 and one at section B-18.

Earth Pressure Cells: A total of 96 earth pressure Cells have been installed in the dam, out of which 27 are located at section B-7, 48 at section B-15 and 12 at the valley section.

Inclinometers: A total of 13 inclinometers have been provided in the dam, three at section B-7, one at section N-9, three at section B-13 and four at section B-15. The settlement is also measured with the same instruments as it has magnetic settlement points also incorporated inside.

Surface Settlement Points: A number of settlement points have been provided on the dam crest and on the upstream and downstream faces.

Seepage: A number of gadgets have been provided to measure the seepage through the body of the dam at different locations.

In addition to the above-mentioned instruments, Horizontal Movement Gauges and an ASPIRE system were also provided in the dam.

Typical Results of Instruments: Some plots of the data obtained from the instruments with respect to the reservoir level is plotted and shown in the following Figures 19 to 23.

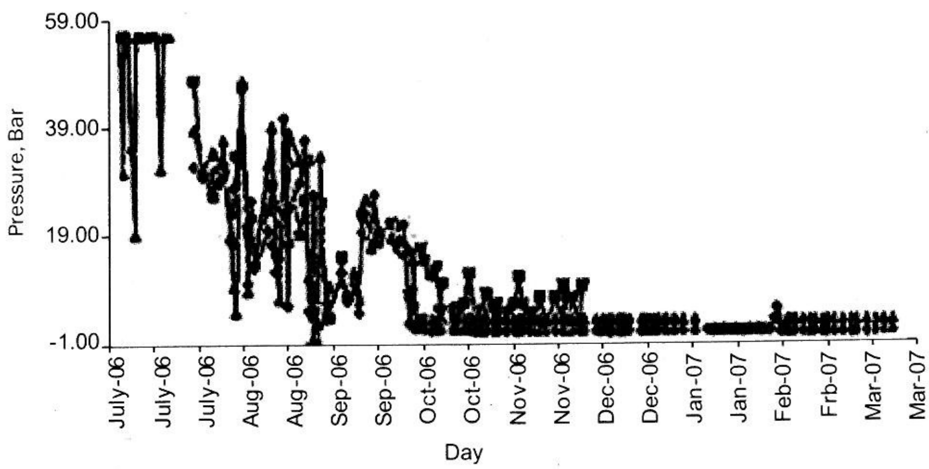


Fig. 19 Plot of Earth Pressure Cell Recordings at Upstream 20.5m at EL 770m (Readings from P 404-H, P 404-I & P 404-V)

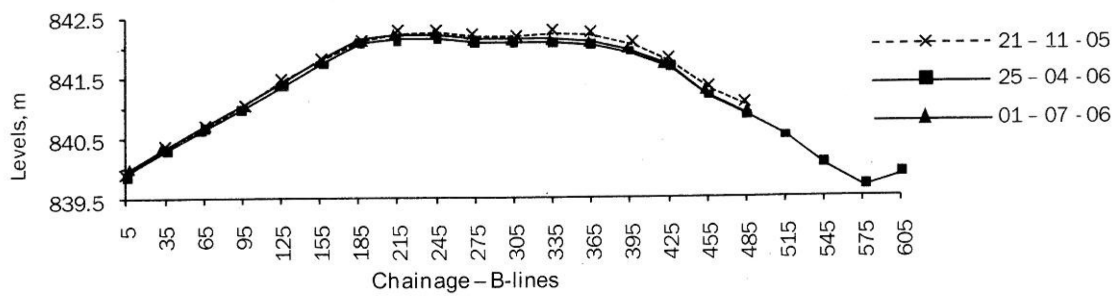


Fig. 20 A Typical Plot of Settlements at the Top of the Dam with Chainage

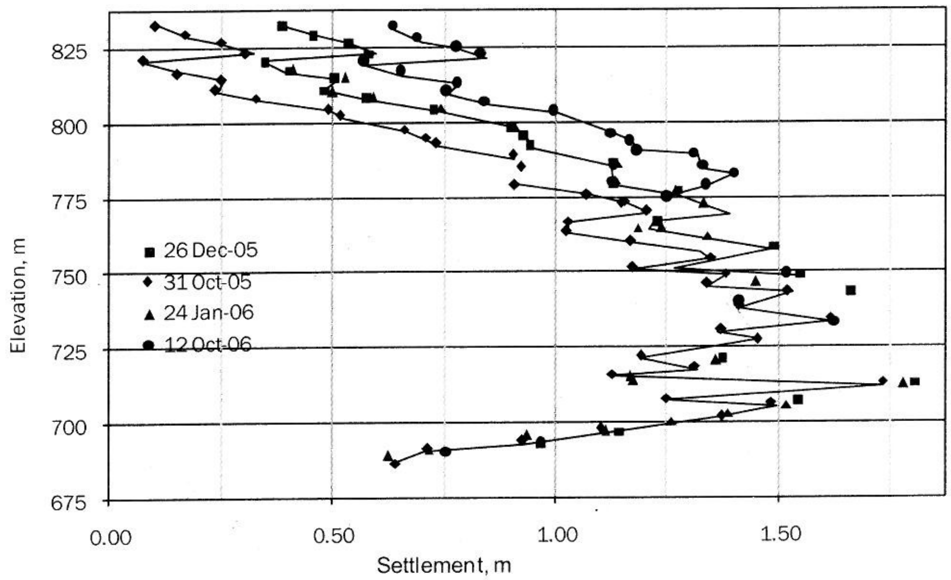


Fig. 21 Typical Plot of Settlement versus Elevation of the Dam
(as obtained from Inclinometer S-101 in the Core, U/S 19.8m of B-7)

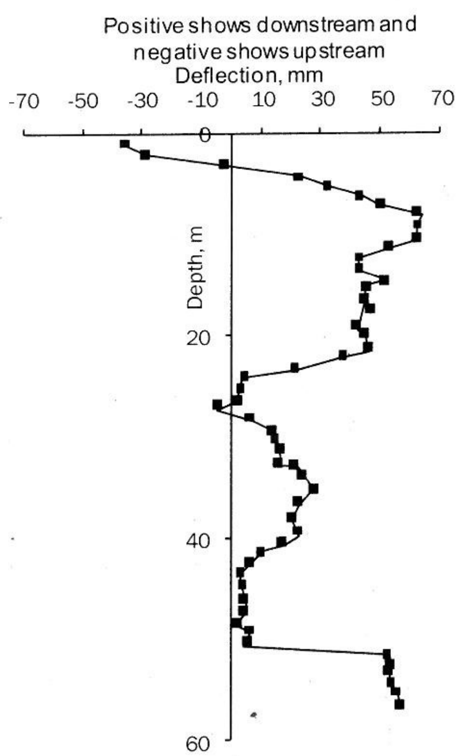


Fig. 21 Deflection of Core in a
Direction Perpendicular to Axis
(From S-101 at B-7,RD 194.8m & U/S 19.80m)

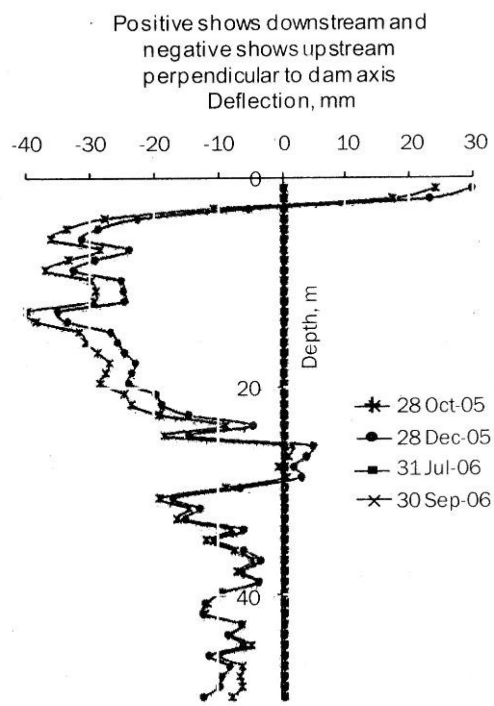


Fig. 22 Typical Inclinometer Data Plotted w.r.t Time
(During 28 Oct 2005 to 30 Sept 2006)

Underground Excavations

Underground excavations are an essential part of most of the hydro power project in India. Till some time back, most of these tunnels and underground power houses were designed using Terzaghi's Rock Load Theory postulated in 1946 and in fact, some of the design offices use it even now. There is nothing wrong in using Terzaghi's method, provided the designers understand its basis and its limitations.

Rock defects and loads on tunnel supports" by Karl Terzaghi was a landmark paper in tunneling literature and, for many years, it provided the basis for the design of tunnels, particularly those constructed in North America. There are still many valuable lessons to be learned from this work as it was based on the experimental work done by Terzaghi. He was fully aware of its limitations when he said that the supports worked out on the basis of present methods were on the higher side and the tendency was to put more supports than required due to lack of our understanding, and any attempt to reduce them should be welcome.

The "tunnel supports" discussed by Terzaghi were primarily steel sets and these were designed to support the "rock load" due to the weight of the broken ground resulting from the excavation of the tunnel. This concept is illustrated in Figure 24 and Terzaghi developed a set of guidelines for estimating the rock load for different geological conditions.

Really speaking, supporting a tunnel which has just been excavated is an interaction analysis.

Terzaghi (1925) had already published one of the first solutions for the elasto-plastic stress distributions around a cylindrical underground opening but he did not apply his calculations to the design of tunnel support systems. Between 1938, when Fenner's paper was published, and 1983, when Brown's paper came out, there were several other papers describing alternate solutions for the rock-support interaction analysis. Brown et al (1983) reviewed these solutions and they also published their own analysis. There have been several additional rock support interaction solutions published since 1983.

Hoek (2001) stated in his Terzaghi Memorial Lecture that the behavior of the tunnel face was not considered in either the Terzaghi "rock load" design method or the rock-support interaction analysis. This omission was not important for relatively shallow, small tunnels since it was usually possible to devise some practical means for supporting the face if this proved to be necessary. However, as both the size of tunnels and their depth below surface increased, the stability of the face became a serious issue. This is illustrated in Figure 25 that shows the plastic extrusion of a tunnel face as determined by means of an axi-symmetric finite element model. Lunardi (2000) has suggested that understanding and controlling the behaviour of the "core" ahead of the advancing tunnel face is the secret to successful tunnelling in squeezing ground conditions.

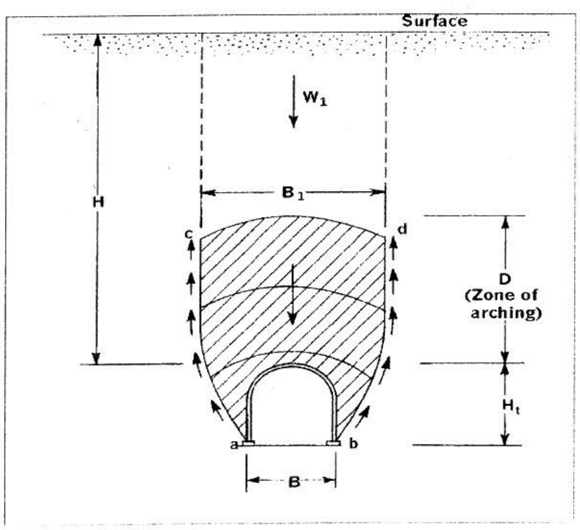


Fig. 24 Terzaghi's Ground Arch Concept (Reproduced from "Rock Defects and Loads on Tunnel Supports" Published in 1946)

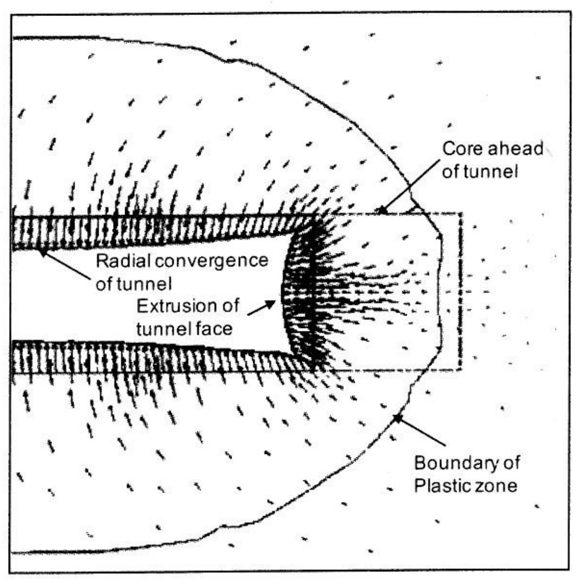


Fig. 25 Section through an Axi-symmetric Finite Element Model showing Extrusion of the Tunnel Face due to Failure of the Core ahead of the Tunnel

Singh and Goel (1999) rightly gave the following comments on the use of Terzaghi's Rock Load Theory for designs:

> It provides reasonable support pressure estimates for small tunnels with diameter up to 6 meters.

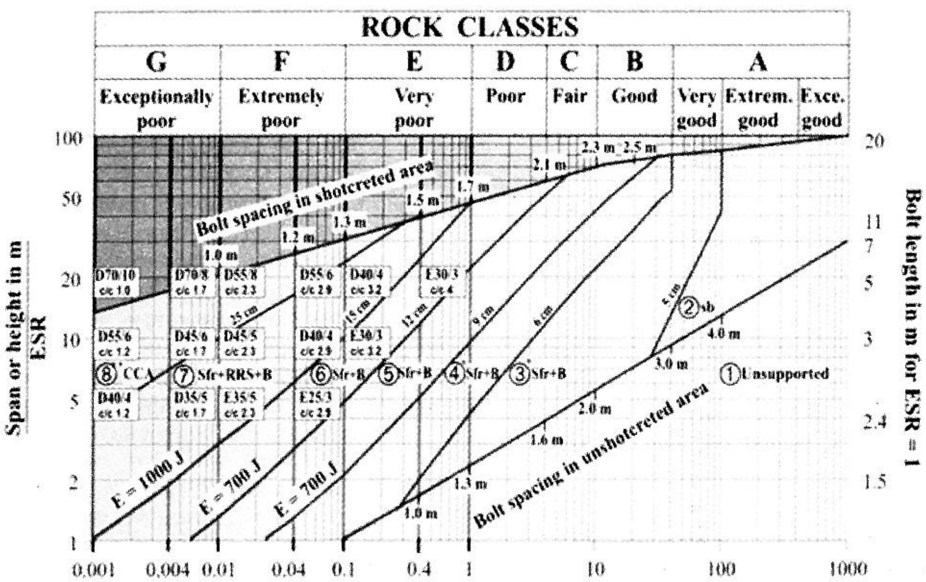
> It gives over-estimates for large tunnels with diameter above 6 meters.

> The estimated support pressure has a wide range for squeezing and swelling rock conditions for a meaningful application.

Then came the Rock Mass Classifications given by Barton (1974) and Bieniawski (1973), popularly known as the Q Index and RMS system. Both these systems came almost together and became instant hits with the engineers mainly because the engineering geologists till then used to talk in qualitative terms. For the first time, an attempt was made to quantify various factors affecting the effort required in tunneling through the given media. It also made it possible to compare one rock mass with the other and transfer the experience of one site to the other.

What it also did, unfortunately, was that it was stretched to a degree and several correlations were developed by authors and other researchers which was, probably, not its original intention. Bieniawski's Classification System was revised at least half a dozen times. Barton had correlated the Rock Mass Quality Q with the supports provided in the tunnels which he had studied. He proposed that the supports required for the new tunnel may also be designed using the similar supports. He also used a factor called SRF or the Stress Reduction Factor, which was not very clearly defined. The other factors Jn, Jr, Ja, Jw and RQD were more clearly defined and therefore easily estimated.

Barton suggested a support system with respect to the rock mass quality as shown in the Figure 26.



Rock mass quality $Q = \frac{RQD}{Jn} \times \frac{Jr}{Ja} \times \frac{Jw}{SRF}$

REINFORCEMENT CATEGORIES

1) Unsupported
2) Spot bolting, **sb**
3) Systematic bolting, (and unreinforced shotcrete, 5-6 cm), **B(+S)**
4) Fibre reinforced shotcrete and bolting, 6-9 cm, **Sfr+B**
5) Fibre reinforced shotcrete and bolting, 9-12 cm, **Sfr+B**
6) Fibre reinforced shotcrete and bolting, 12-15 cm, **Sfr+B**
7) Fibre reinforced shotcrete > 15 cm + reinforced ribs of shotcrete and bolting, **Sfr+RRS+B**
8) Cast concrete lining, **CCA** or **Sfr+RRS+B**

E) Energy absorption in fibre reinforced shotcrete at 25 mm bending during plate testing

D45/6 c/c 1.7 = RRS with 6 reinforcement bars in double layer in 45 cm thick ribs with centre to centre (c/c) spacing 1.7 m. Each box corresponds to Q-values on the left hand side of the box. (See text for explanation)

*) Up to 10 cm in large spans
) Or **Sfr+RRS+B

Fig. 26 Barton's Correlation of Q (Rock Mass Quality) and the Support System Required

Interaction Analysis

In preference to correlations, Hoek and Brown have been advocating the use of the interaction analysis which appears to be the right way to analyze underground openings and find out the support pressure and the corresponding convergence. It is possible to work out and draw the ground reaction curve and the support characteristics manually if some assumptions are made. If the tunnel is taken as circular and the pressure around is taken as hydrostatic, and the zones around the tunnel are taken as plastic, strain softening and elastic respectively, it gives results which are quite comprehensible.

Hoek and Brown have used Bieniawski's Classification system as the base for their rock mass strength criterion. They have also been modifying their originally proposed system almost regularly. The final system is based on GSI (Geological Strength Index) and uses RMR to work out the indices qualifying the final strength of the rock mass.

Since Barton's system was evolved using the case studies of several tunnels, it has found favour with the designers of tunnels in our country. They have not only used it for preliminary designs but have actually used it for the final designs as well. However, there are two major problems with the use of this system for any design of permanent support system.

> Since Barton's database was not from instrumented tunnels, we do not know whether the supports provided in those tunnels were adequate. If the supports were less than required, it would have showed up. If the supports were more than the required there is no way of finding out, as there was no instrumentation provided. Quite obviously the supports provided in those tunnels were more than required. If the data base is faulty, that is, provides more support than is required, any design based on such correlation would end up suggesting supports which are more than those required. When other researchers carried out instrumentation of a few tunnels and compared the actual loads with those predicted by Barton's method, they found that the actual supports required was much less than the one predicted by Barton.

> The interaction analysis shows that the rock loads and the tunnel convergence not only depend on the relative stiffnesses of the ground and the support system, but also the time of the placement of supports. In other words the final load coming on the supports depends on the initial deformation allowed to take place before the support system is placed in position.

> Since Barton's method does not take any other factor into account other than the Rock Mass Quality, in particular the characteristics of the support system and the timing of its placement, it misses an important aspect of the interaction.

Importance of Stress Measurements and Failure of Kopili Tunnel

At Kopili Hydro electric project, the tunnel between the surge shaft and the penstock was about 600 meters in length. Out of this, about 60 m was provided with steel liner and its preceding 70 m length was provided with reinforced concrete liner. The remaining length of the tunnel up to the surge shaft was lined with plain concrete. The slope of the tunnel in this reach was 1 in 50. When the intake gate was opened for the pressure - testing of the tunnel, the reinforced concrete portion of the tunnel cracked open and a stream of water gushed out of the sloping hill mass, bringing soil debris down the hill. The details of the Project are briefly given below.

Located on the river Koili in the norh Cachar District of Assam, the Kopili Hydro Electric Project was the maiden venture of NEEPCO when it came into existence in 1976.

The first stage of the project had two dams and dyke systems for creating two reservoirs, one on the Kopili River and the other on the Umrong, a tributary of Kopili. Water from the Kopili reservoir is utilized in the Khandong power station through a 2759 meter long tunnel to generate 50 (2x25 MW) of power. The tail water from this power station is led to the Umrong reservoir. The water from Umrong reservoir is taken through a 5473 meter long tunnel to the Kopili power station to generate 100 MW (2x50 MW) of power.

The plant was extended for setting up two additional units of 50 MW each. The second stage of the Project involved generation of additional 25 MW of power. Thus the total capacity of the project at present is 275 MW. However, the project suffered a major accident and a set back in the early stage.

The tunnel was designed to withstand an internal water pressure of 1.4 MPa. Part of length of the tunnel between the surge shaft and the penstock was provided with the steel liner and part with reinforced concrete. The lengths of the steel lined and RCC portions were 60 m and 70 m respectively. The remaining Length of the tunnel was lined with plain concrete. The slope of the tunnel in the reach was 1 in 50. When the tunnel was being pressure tested for commissioning the project, the reinforced concrete portion of the tunnel cracked open and a stream of water gushed out the sloping hill mass; bringing down soil debris along the hill. It was reported later that the maximum pressure inside the tunnel at the time of failure was recorded as 0.7 MPa at the valve house gauge located close to the tunnel exit portal.

Causes of Failure

A committee of experts was constituted to investigate the causes leading to failure. During their inspection, it was found that the RCC lining had cracked at the crown extensively along its entire length of 70m and about 30 m in the adjoining plain concrete lining. A sub vertical joint in the rock could be seen through the crack in the concrete lining. The joint had been opened up by the water which escaped from the tunnel flushing the joint of the infilling materials. Though the reinforcements themselves were found to be intact, they were separated from the concrete at many places. The lapping joints of the reinforcements were all along in the crown, and these joints had failed. Behind the crack lining, the rock mass was observed to be extensively jointed.

When the tunnel was being designed, the practice followed was to locate the tunnel below the ground as far as possible at a depth equal to the total static head of water over the center line of tunnel. Wherever sufficient cover was not available, steel liner or RCC liner was provided. These liners were designed depending upon the competency of rock mass in sharing the hydraulic pressure. If leakage of water from the tunnel was expected to endanger the rock mass, steel liner was preferred to RCC liner. Based on the above concept, it was decided to provide a steel liner in the last 300 m length of the tunnel. However, due to certain practical difficulties of construction, a review of the design was made to reduce the length of steel liner. Though in-situ stress measurements were made using flat jack tests at several locations in this tunnel, the. Tests could not be carried out at certain critical locations. Based on the general observation of the rock mass, the length of steel -lining was reduced to 60m and an RCC lining of 70m length was provided.

The failure of this tunnel has demonstrated the need for precise measurement of in-situ stresses and its impact on the design. The criteria of sufficient rock cover may be suitable for preliminary designs but for the final design, the detailed geological features, rock mass deformability, in-situ stresses, rock permeability and ground water conditions should be given appropriate considerations.

Where the minimum principal stress is not the vertical stress, and where deformable rock or shear zones exist, positioning a tunnel to meet only vertical cover criteria may not be adequate. Knowledge of complete stress field and of the rock modulus is necessary. Finally, in the design of the lining for a pressure tunnel, the following factors should be considered.

> Head loss
> Leakage of water
> Long-term stability

Head loss through a conduit is principally a function of wall roughness, the diameter and the velocity. As a result, hydraulic equivalence can be obtained between large diameter unlined tunnels versus smaller lined tunnels of greater hydraulic efficiency. With small tunnels, 2 to 3 m in diameter, there is a greater need for a smooth lining to maintain acceptable head losses. However, as the diameter is increased, the wall roughness has less effect on head loss, and equivalence is achieved with small diametral changes.

Excessive leakage of water can occur from pressure tunnels in two ways. Firstly, by hydraulic fracturing and secondly if the rock is pervious and the internal pressure exceeds the external ground water pressure. Different types of linings can be considered as impervious and RCC lining or reinforced shotcrete lining can be considered to be pervious as they have local pervious zones due to placement imperfections, or shrinkage cracks during curing. It is a common misconception that concrete or shotcrete is impervious. They are also easily cracked under internal pressure where deformable rock zones exist. If the permeability of the rock mass is high, grouting will be necessary. Controlled pressure grouting in different stages will be useful where grout loss has to be controlled and where high pressures are necessary.

The designed thickness of RCC lining was 30 cm. Good amount of skill is called for concreting the lining, as it is to be carried out in the restricted space behind the shutter. The workability of the concrete could be increased without reducing strength by adding plasticisers. Concrete may have to be pumped using concrete pumps. After shrinkage of concrete, the gap between concrete and rock surface should be grouted. In the failed portion of the tunnel, it was learnt from the records that the grout holes at the crown had taken substantial quantities of grout whereas the side holes had taken very little grout. This raised a doubt about the integrity of the concrete at the crown. However, a series of non-destructive tests carried out on the concrete lining did not indicate any apparent bad quality.

Use of Numerical Analyses

Methods available for working out the ground reaction curve and the support characteristic curve are simple if certain assumptions are made. However, the Numerical modeling has advanced to such an extent that it can take any shape of the opening and any stress regime into account. It has also become very popular because of the user friendliness of commercially available packages and as a result, has reached most of the design offices. It can also take the actual sequence of excavation and supporting into account. This is of particular relevance for underground powerhouses. Cases of Tehri Underground Powerhouse and Monastiraki Metro Station at Athens could be cited as examples.

Tehri Underground Powerhouse

The complexities of design of the underground powerhouse at Tehri were compounded by the fact that either the basic data required for the design was not known or known with several question marks. The geological strata through which the excavations were to be made were not known precisely So also the material characteristics - not only the properties of various rock masses but also the characteristics of the interfaces, and the in situ stresses, ground water conditions, etc.

Due to complexities described above it was not possible to design and construct the underground excavations in one go. The design had to be modified as the excavation progressed. The basic philosophy has to be - *design as you excavate.* The design methods used in the past were based on thumb rules or empirical formulations.

Underground excavations tend to close under the in situ stresses of the surrounding rock mass and, depending on the ground characteristics, this may lead to either reduction of the excavated area or collapse, either of which is not desirable. Therefore, a basic requirement for the success of the excavation of an underground opening is to monitor its deformation during excavation and to control it before it reaches the limits of instability.

Modern methods of underground excavations are based on the principle of controlled deformation rather than no deformation or too much deformation. This is achieved by establishing a detailed monitoring system which not only records the deformations during excavations, but also indicates the need for additional supports if the convergence exceeds certain limits. Incidentally it also tells us about the adequacy of the support system being deployed for the particular reach which depends upon several factors such as in situ stresses, rock mass characteristics, method of excavation etc.

Monitoring of deformations, support pressures and other parameters like pore water pressure, subsidence etc., is crucial for deciding about the optimum support system and provide an economical solution for stable underground excavations.

The design process involved continuous evaluation of rock mass parameters, detailed 2D and 3D numerical modelling and monitoring program. Hoek and Brown failure criterion was used to estimate the rock mass strength parameters for the model analysis and the model results validated with the field measurements. FLAC 2D (Itasca 1995a) was used for design of pattern support system. Shear zones and major shear planes were modelled in 3D discontinuum analysis with 3DEC (Itasca, 1995b). Instrumentation scheme was planned and implemented to calibrate the model. This case study demonstrates the role of advanced numerical modelling technique in the analysis and design of the caverns.

Power House Complex

The power house complex consists of two main parallel caverns namely the Machine Hall (MH) and the Transformer Hall (TH) located about 370 m below the surface. The MH cavern is 188 m long, 22m wide and 47 m high. There are four turbine pits 16 m deep on the floor of the machine hall. The TH cavern is 161 m long, 18.5 m wide and 36 m high located upstream of the MH with a 41.75 m rock pillar between them. In addition there are other excavations such as pressure tunnels, draft tubes, bus duct tunnels and adits joining the main caverns and drainage galleries.

The Tehri Hydropower Project is located in the Lesser Himalayas. The rock formation in the power house area is mainly massive to thinly bedded Phyllitic Quartzite with caverns oriented perpendicular to foliation strike direction. There are four major joint sets and several shear planes with varying thickness (Navani, 1996). There is a shear zone of about 2 - 5 m thick, dipping along the foliation intersecting the caverns. The 3D model of excavation and geological features are shown in Figure 27.

Numerical Model

The input parameters for the model were estimated based on the methodology proposed by Hoek and Brown (1977). With the progress of excavation, rock mass properties were updated and support design was modified. The final rock mass properties used for the analysis are given in the Table 5 which were based on the Geological Strength Index = 56 (obtained from visual inspection and geological mapping), and Unconfined

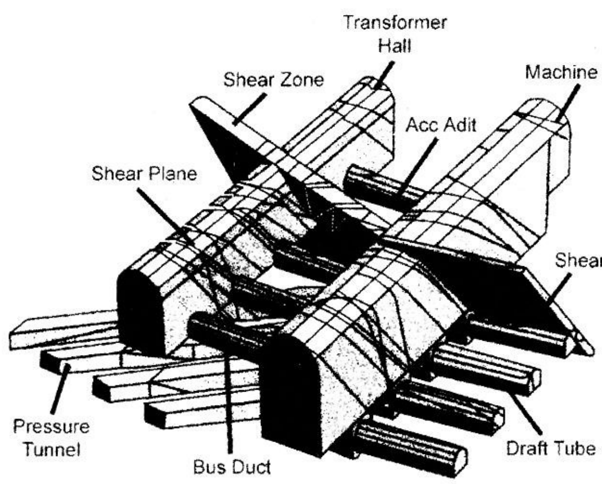


Fig. 27 3DEC Model of Excavation and Geological Features (Tehri Dam)

Table 5 Roc Mass Parameters (Tehri Dam)

	E	c	ϕ	$\sigma 1$
Rock mass	8.66 GPa	2.75 MPa	33°	0.22 MPa
Shear zone	4.00 GPa	1.50 MPa	30°	0.10 MPa
Shear plane		1.00 MPa	28°	

Compressive Strength = 60 MPa and m[1] (Hoek Brown Parameter) = 10 (determined from laboratory triaxial tests).

The in situ stresses were measured at the crown by hydro fracturing test. It was established that the horizontal to vertical stress ratio along the caverns was 0.526 and across the caverns was 0.314, while the vertical stress was equivalent to the overburden rock mass.

The power house complex with MH and TH was analyzed in 2D to establish pattern support design. In this analysis, the construction sequence of the excavation and support installation was modelled considering the rock mass to be continuum with Mohr Coulomb non linear material behaviour. The effect of rock bolt was considered in the support interaction analysis neglecting the shotcrete. The length and spacing of rock bolts were optimized by a series of analyses, which also involved updating of rock mass properties during construction. The results of the analysis showed that the failure zone on the roof was about 3 m for both MH and TH, and on the walls it extended to about 12 - 13 meters for MH and about ten meters for TH at the mid-height. The displacement on the roof and mid-height of MH was about 2.5 cm and 2.25 cm and that of TH was it was 0.9 cm and 1.5 cm. The final support design consisted of pre-tensioned rock bolts: on the roof 25 mm dia, 6 m and 10 m long for MH and 6 m and 8 m long for the TH, on walls 32 mm dia of varying length from 9 m to 15 m for MH and of varying length 8 m to 13 m for TH. The axial forces developed in the bolts were within the bolt capacity. The excavation sequence, deformed boundary, failure region, and rock bolts with axial forces are shown in Figure 28.

The power house complex was analyzed in 3D with bus duct, draft tube and pressure tunnels joining the MH and TH caverns. The geologic features mainly the folded shear zone, and about 19 shear planes were explicitly modelled as shown in Figure 27. Discontinuum analysis was performed to understand the interaction of multiple excavations with geological features and identify the regions requiring enhanced support.

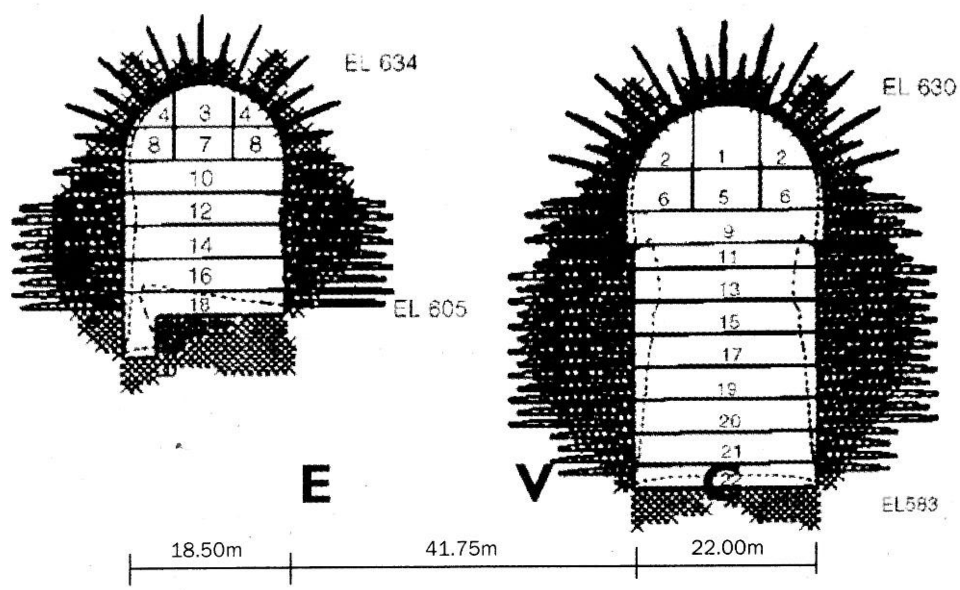


Fig. 28 Failure Region and Rock Bolts with Axial Forces

Model Calibration

Instrumentation was planned and installed at appropriate locations and time during excavation so that maximum information could be obtained for the calibration of the model. Rock deformation was measured by mechanical type MPBX. The extensometers were installed on the roof of both the caverns from inside, i.e. after the excavation of the crown advanced to the desired location, however, monitoring usually becomes difficult with benching.

The MPBX on the walls were installed and monitored from the drainage galleries before the excavations reached the instrumented levels. The wall was monitored un-interrupted by the construction activity. A layout of the instrumented cavern is shown in Figure 29. Caverns were instrumented at three sections.

The wall instruments were installed in a phased manner when excavation level of MH was El 609 and TH at El 618 and the exact sequence of excavation and installation of the MPBX was modelled using FLAC. The comparison of the calculated and measured deformation from the cavern wall into the rock mass for MPBX 3 and MPBX 5 is shown in Figures 30 and 31. It can be seen that the results are in close agreement indicating proper validation of estimated rock mass parameters and support design.

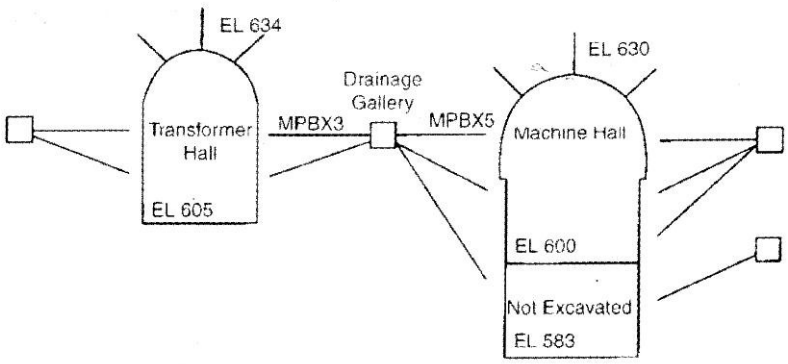


Fig. 29 Layout of Instrumentation

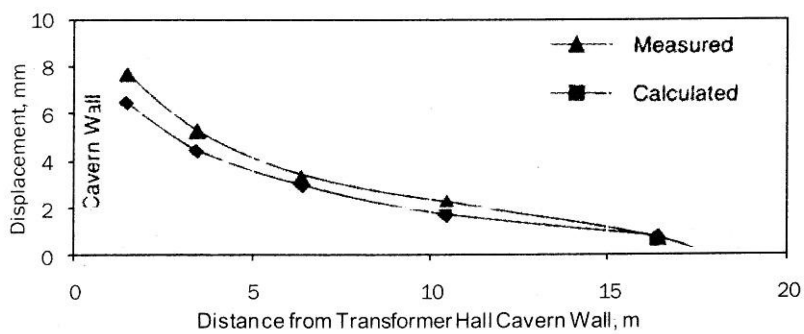


Fig. 30 Calculated and Measured Data-MPBX 3

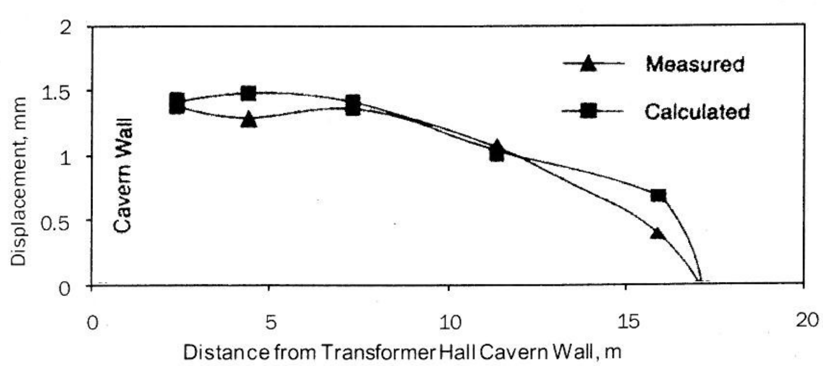


Fig. 31 Calculated and Measured Data-MPBX 5

Metro Station at Monastiraki, Athens

Athens is a country full of archaeological monuments. When the Olympic games were to be held in Athens, there was a need to improve the existing infrastructure. It was decided that the existing metro line would be extended and a station would come up at Monastiraki. Since the settlement at the ground surface was of prime concern, it was decided that a vertical shaft would be driven first and supported heavily with a circular slab so that the deformations are minimized.

On reaching the crown level of the station cavern, a horizontal drift was driven along the length of the cavern. Then a number of circular micro -tunnels made out of steel pipes filled back with concrete were driven to form an arch like structure. By this process the entire top was covered from one spring level to another. The idea was that this arch like structure would take the vertical load and when the excavation is carried out in the middle of the cavern, the vertical deformations would minimize.

However, even when this excavation of shaft, drift and the micro-tunnels was being done, there was some deformation. These deformations were measured from time to time at different stages of excavation. The problem referred to us was to create a three dimensional model of the ground along with the shaft, drift and micro tunnels and see how much deformation we get. By suitable manipulation of the parameters, the deformations had to be matched with those actually measured at different stages. Once the model is calibrate and validated, then the anticipated deformations were to be worked out for the entire excavation. Also the locations of various other openings had to be finalized on the basis of ground subsidence.

Figures 32 and 33 show the model and a typical comparison between calculated and observed results.

However, it is essential that some measurement data should be made available so that the models can be verified and calibrated.

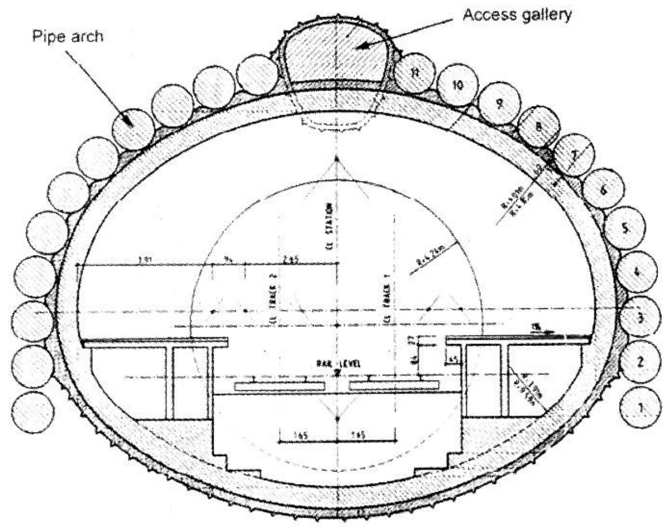


Fig. 32 Improvised Pipe Arch above Tunnel Crest at Monastiraki Metro Station

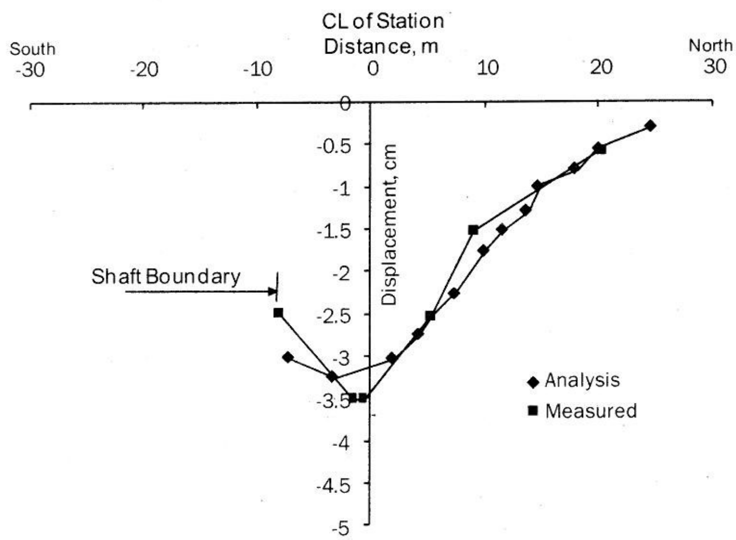


Fig. 33 Typical Comparison of Observed versus Calculated Settlements

Squeezing Ground Conditions

Some of the projects, particularly in the Himalayan region have experienced squeezing ground conditions. The stresses in the tunneling media are high on account of high overburden which get further enhanced because of the stress concentration due to excavation and the strength of the material is poor to very poor leading to its failure. The natural tendency, therefore is to close or squeeze. To make the matters worse, the supports provided are generally the steel sets with pre-cast concrete laggings back-filled with lean concrete. The movement of the ground is so much that it very quickly catches up with the gaps available and start showing up in the form of twisting and turning of steel supports. Some photographs of the buckled and twisted steel supports are shown as examples (Plates 13 and 14)

The cases of Giri Bata Project and Yamuna Hydel Scheme may be cited as a classical example. Both the Hydro power projects are located in the Yamuna Valley, in fact

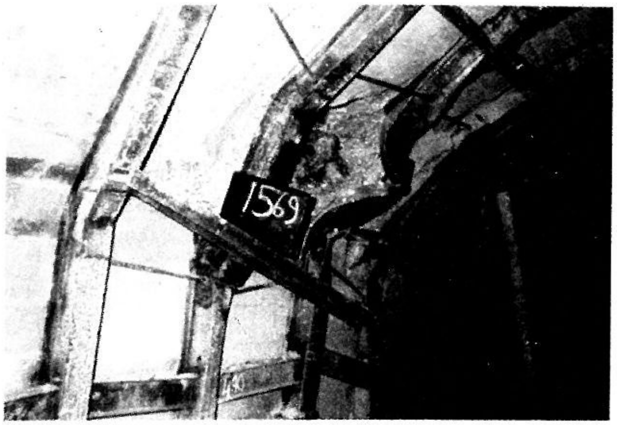


Plate 13 A View of the Buckled Supports in One of the Squeezing Tunnels

Plate 14 Another View of Tunnel through Squeezing Grounds

separated by the river Yamuna itself. Whereas the Giri Bata Project was being constructed on the river Giri, a tributary of Yamuna, by Himachal Pradesh State Electricity Board and was being designed by the Central Water Commission. The Yamuna Hydel Scheme on the other hand was being designed and constructed by the Irrigation Department of Uttar Pradesh.

The designers of the tunnels in Uttar Pradesh, wanted to stop the movement of tunnels, and deployed the heaviest steel sections available in the market with extra flanges welded to make them more stiff. They ended up with a virtually steel lined tunnel and still had some squeezed sections where the supports had to be replaced.

On the other hand, the designers of the Giri Bata Project tunnels - the Central water Commission - allowed them to proceed and did not bother about the squeezes. As a result the ground moved as much as 1.4 meters in a tunnel of 5 meter diameter, allowing too much loosening all around. When the wanted to bring the section back to its normal size there were chimney formations and all kinds of problems.

Looking back, none of he two methods were correct. As we understand now, the best way to tackle squeezing ground condition is to allow deformation to take place under controlled conditions.

Typical closure diagrams of a section of Yamuna hydroelectric project is shown in Figure 34.

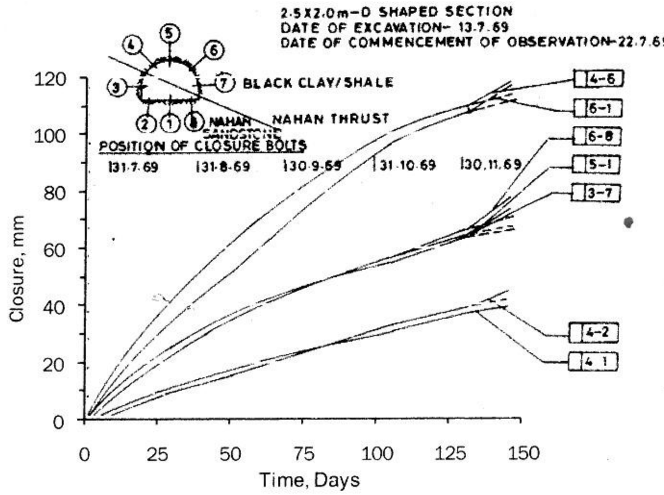


Fig. 34 Tunnel Closure Measurements of a Section of Yamuna Tunnel

The Phenomenon of Squeezing

It is not that the squeezing ground conditions have been experienced in India alone. These conditions have been met in European tunnels as well, particularly in the Alps and in tunnels through Mount Blanc. As a result a lot of other researchers have also applied themselves to find out the causes and work out the remedial measures.

Before we proceed with the actual experience of squeezing ground in India, let us look at what is meant by squeezing ground conditions and why do they occur.

According to the International Society for Rock Mechanics (ISRM) squeezing is: the time dependent large deformation which occurs around the tunnel and is essentially associated with creep caused by exceeding a limiting shear stress (Barla, 2001). This phenomenon usually takes place in water bearing weak rocks accompanied by high tunnel depth. Most of the tunnels in the Himalayan region pass through such ground conditions and are therefore susceptible to squeezing behaviour. Deformation may terminate during construction or may continue over a long period of time. The degree of squeezing often is classified to mild, moderate and high, by the conditions below,

a) Mild squeezing: closure 1-3% of tunnel diameter;

b) Moderate squeezing: closure 3-5% of tunnel diameter;

c) High squeezing: closure > 5% of tunnel diameter.

Rate of squeezing depends on the degree of over-stress. Usually the rate is high at initial stage, say, several centimetres of tunnel closure per day for the first 1-2 weeks of excavation. Closure rate reduces with time. Squeezing, unless checked, may continue for years in exceptional cases. Squeezing may occur at shallow depths in weak and poor rock masses such as mudstone and shale, particularly if they are water charged.

Rock masses of good quality may also squeeze at great depths under very high cover or under very high tectonic stresses. Sometimes, the geological features such as syncline or the bottom of the valley may also result in high stresses.

Prof Bhawani Singh and Dr. Goel have given a simplified method in which they have separated the squeezing tunnels from the non - squeezing ones based on the Rock Mass Classification and the rock cover. It presumes that the rock mass classification is an indicator of its strength which is not necessarily true and the rock cover indicates the stress at that level. Rock cover in fact indicates the stress in the vertical stress provided the ground above that tunnel is horizontal to a reasonable extent. It can also be assumed that since the rock mass is of poor quality and it will not be able to take high differential stresses, the stresses could be assumed as hydrostatic. Within these limitations they found that the division between squeezing and non-squeezing condition is by a line $H = 350\ Q^{1/3}$, where H is in meters.

Above the line, i.e., when $H > 350\ Q^{1/3}$, squeezing condition may occur. Below the line, i.e., when $H < 350\ Q^{1/3}$, the ground is of generally non-squeezing condition.

As per the authors the squeezing is represented by the Rock Mass Number (N), which is the Q-value when SRF is set to be 1. By doing so, the parameter N allows one to separate the effect of in-situ stresses from the rock mass quality. In situ stress, which is the external cause of squeezing is separated by considering the overburden depth. From the above figure, the line separating non-squeezing from squeezing condition is,

$$H = (275\ N^{1/3})\ B^{-0.1} \qquad (2)$$

Where H is the tunnel depth or overburden in meters and B is the tunnel span or diameter in meters.

Evert Hoek in his Terzaghi Memorial Lecture stated that based on field observations and measurements, Sakurai (1983) suggested that tunnel strain levels in excess of approximately 1% are associated with the onset of tunnel instability and with difficulties in providing adequate support.

Field observations by Chern et al (1998), plotted in Figure 35, confirm Sakurai's proposal. It may be noted that some tunnels which suffered strains as high as 5% did not exhibit stability problems. All the tunnels marked as having stability problems were successfully completed but the construction problems increased significantly with increasing strain levels. Hence, the 1%

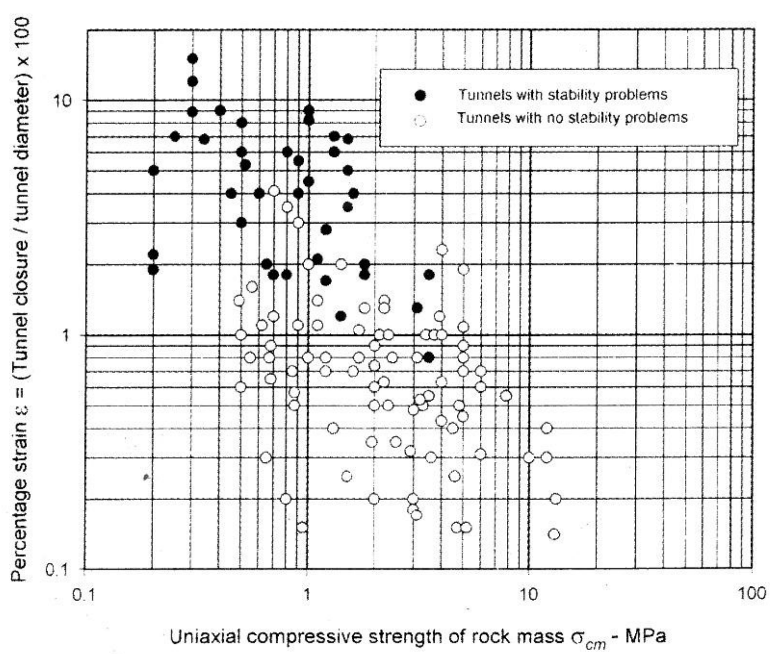


Fig. 35 Percentage Strain versus Uniaxial Compressive Strength

limit proposed by Sakurai is only an indication of increasing difficulty and it should not be assumed that sufficient support should be installed to limit the tunnel strain to 1%. In fact, in some cases, it is desirable to allow the tunnel to undergo strains of as much as 5% before activating the support.

According to Hoek, one of the problems in interpreting field observations of tunnel squeezing is that of estimating the influence of the tunnel support. It is particularly difficult when the support capacity is exceeded and where steel sets buckle, shotcrete cracks or rockbolts yield. The best that can be done under these circumstances is to plot the observations and to compare them with strain curves for a range of support pressures. This has been done in Figure 36 that shows observed closures for a number of tunnels in Venezuela, Taiwan and India.

Figure 36 shows that the observations are in reasonable agreement with the squeezing behaviour predicted by the Equation. The points marked 1, 2, 5 and 7 are for tunnels in which severe squeezing occurred and where extraordinary steps had to be taken to stabilize the tunnels.

To conclude this discussion on tunnels in squeezing grounds which is very important for tunnels in our country particularly in the Himalayan region, it can be stated that Hoek (1999) published details of an analysis that showed that the ratio of the uniaxial compressive strength σcm of the rock mass to the in situ stress po can be used as an indicator of potential tunnel squeezing problems. Following the suggestions of Sakurai (1983), an analysis was carried out to determine the relationship between $\sigma cm/ po$ and the percentage "strain" of the tunnel. The percentage strain ε is defined as 100 x the ratio of tunnel closure to tunnel diameter.

Figure 37 gives the results of a study based on closed form analytical solutions for a circular tunnel in a hydrostatic stress field published by Duncan Fama (1993) and Carranza-Torres and Fairhurst (1999). Monte Carlo simulations were carried out to determine the strain in tunnels for a wide range of conditions. It can be seen that the behaviour of all of these tunnels follows a clearly defined pattern, which is well predicted by means of the equation included in the figure.

The same equation can be plotted as shown in figure 38 for use in any converging tunnel.

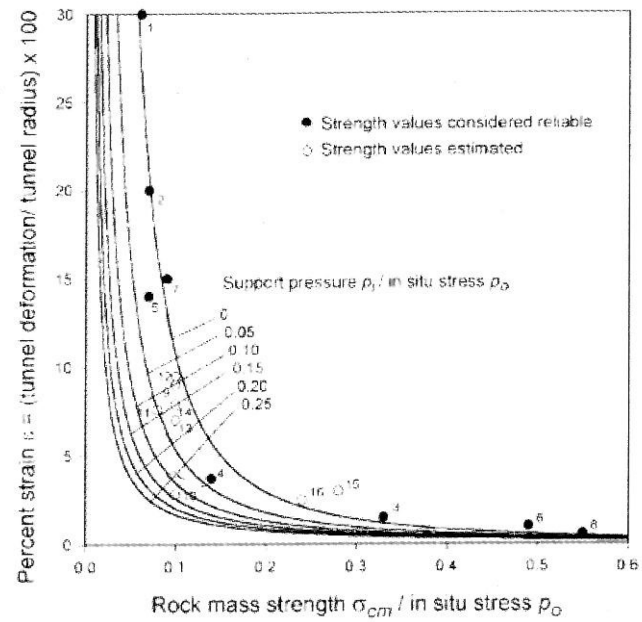


Fig. 36 Predicted and Observed Strains

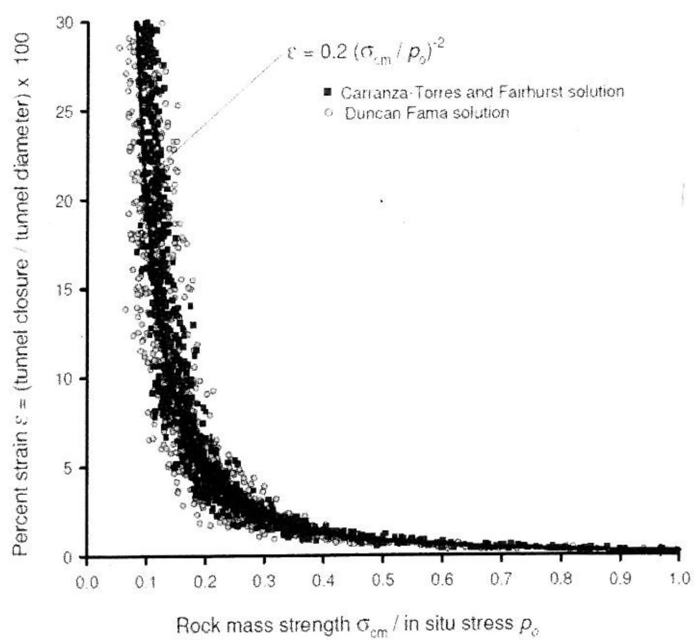


Fig. 37 Relation between Percent Strain and Rock Mass Strength/In-situ Stress

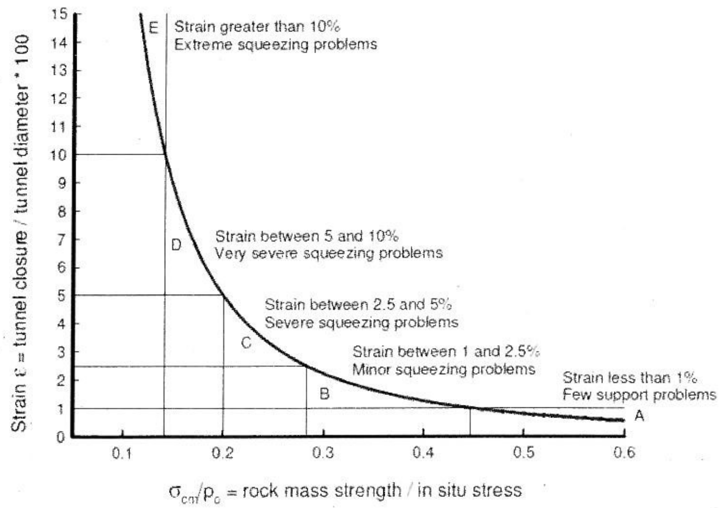


Fig. 38 A Guide Curve for Squeezing Tunnels after Hoek

Construction Materials

For constructing water resources development projects economically, it is necessary that locally available construction material should be used to a large extent. It is the task of the geotechnical engineer to ensure that enough material of acceptable quality is available in the vicinity of the construction site. Starting the construction activity without ascertaining the properties of the materials being used may lead to difficult situations and ultimately result in time and cost over-runs. Some of the problematic construction materials are briefly discussed below:

Dispersive Soils

Sawan Bhadon dam in Rajasthan provides an example. Though a small dam of 27 m height only, piping failure was observed immediately after 24 hours of first filling. The water level at the time of failure was not very high and the project authorities could plug the breach from upstream side.

It was seen that the soil used for construction was dispersive in nature and therefore prone to dispersion - piping problems. It was also observed that the compaction of the dam was non-uniform and that the filler material did not satisfy the filter criterion. The remedial measures included the use of Alum and compaction wherever required.

Unconventional Construction Materials and Innovative Solutions

Broken bricks are used as a coarse aggregate and fine aggregate for the preparation of lime concrete and lime mortar respectively and there are existing BIS standards for that. However when Gumti Project (Tripura) was faced with the problem of restraints on availability of natural stone, as stone aggregate has to be transported from Assam, over a distance of 600 kms. CSMRS suggested the use of over-burnt bricks known as 'Jhama' bricks.

The technical feasibility studies on brick bats concrete were carried out and it was finally recommended for use in the hearting of the dam. Such concrete contributed to more than 65 percent of concrete in the gravity dam. A similar example was faced at Bangla Desh where drilling showed that there was no rock available up to a depth of 200 meters. The problem was concerning he erosion of Meghna and Jamuna banks. Geosynthetic material was used to create bags and filling them up with sand for use as stones. Different sized bags were made to provide 'stones' of different sizes

Geotechnical Problems of Water Resources Development Projects and Strategy for Solutions

Geotechnical problems of water resources development projects in India are typical of a region resulting from the continental crust collision and the associated fracturing.

These problems are not un-surmountable. There is enough expertise available within the country which can take care of them. Certain organizational matters and linkages may, however, need to be strengthened or re-aligned.

Improved Quality of Geotechnical Investigations

A number of geotechnical problems in the water resources development projects have come up due to lack of or inadequate investigations. It is a common knowledge that we, in India, spend much less on investigations than others and many times end up paying much more for the ultimate cost of the project. Greater importance should be attached to the work of geological and geotechnical investigations. We should have a network of organizations at different levels which may take up this work. These levels could be Project level, State level, Regional level and National level. The agencies should have the expertise required to produce the Project Report in a professional manner, such that the projections are realistic and the surprises in cost and time over-runs during and after execution are minimum.

Obligatory Instrumentation

Instrumentation of structures and surrounding grounds should become obligatory. This work should be given to professional agencies. The data should be analyzed by competent persons and rational conclusions synthesized to advance the State of the Art.

Numerical Techniques

Numerical techniques are very powerful tools which can be used at different stages of instrumentation and analysis. The structures need to be analyzed to find out the anticipated stresses and strains so the ranges and precision of the instruments required can be specified as per the requirements of the structure.

Till some time back, the trend was that the Numerical Modelling was considered as an activity meant for intellectuals and confined to the Universities and colleges. The design offices were keeping away from it because they felt that the analysis was not possible unless a number of simplifying assumptions were made. That these assumptions were un-realistic and therefore were of little value as far as the actual designs were concerned. The scenario has changed over a period of time. The new generation of software is very powerful and can take most of the concerns of the designers into account and provide solutions. What is more they can even take such factors into account which are difficult to account for otherwise.

It is heartening to note that more and more design offices are now building up in-house expertise in the use of such software or are making use of organizations which provide such services.

For the interpretation of instrumentation data, it is necessary to have a software which can take the construction schedule, its sequence and the element of time into account and then calculate the response of different instruments provided in the structures at various locations.

Dynamic Characterization of Foundations and Construction Materials

Dynamic characterization of material properties and the foundations are required for satisfactory solutions to the problems of water resources development projects, particularly those located in the seismic areas. Earthquakes of low to high intensity can occur in the country.

Various properties required for the analysis of dams and appurtenant structures viz. dynamic moduli (elastic, shear, bulk and constrained modulus), Poisson's Ratio, Damping and attenuation and Liquefaction characteristics - cycling shear stress ratio, cyclic deformation and pore pressure response can be studied in the laboratory or in the field. The common laboratory tests are Cyclic Triaxial Test, Resonant Column Test and Cyclic Torsional Shear test. Among the field test are the SPT, the Cross hole Test and the Electric Cone Penetration Test. Pressuremeter with Cyclic loading and Cone Penetration with an Accoustic cone are also used.

Earthquake Resistant Design of Dams and Appurtenant Works

In most of the design offices, the capability for the design of dams by pseudo-static method exists. This method is recommended by Bureau of Indian Standards also. Keeping in line with the International Practice the recent trend is to carry out a rigorous dynamic analysis. It is now possible to provide the necessary inputs in terms of material characterization to the designer, and carry out a rigorous dynamic analysis of the concerned structures.

The 'so called' New Austrian Tunnelling Method

The New Austrian Tunneling Method has become very popular in the recent years. Though some of the concepts used in the NATM are not so new and that is what made Muirwod to remark "New Austrian Method of Tunneling is neither new, nor Austrian". The essence of NATM is to recognize and consider the element of time, and the relative stiffnesses of ground and the support system into account. By using instrumentation and a flexible support system, the stiffness of which can be increased and thus controlled in stages, they are able to control the rate of closure of openings on opening up.

These elements are all the more required to be incorporated in our system, where several tunnels in the Himalayan region are squeezing. Several of these problems can be tackled by a combination of blasting, rock-bolting and shotcreting.

Blasting

It is now well known that the best technique to handle squeezing ground conditions is to allow them to deform but in a controlled fashion.

Instrumentation should become an integral part of the design - construct - instrument - monitor - analyze - modify cycle. Reliable instrumentation is the key to understanding the behaviour of ground as well as the structures when subjected to loading and unloading. Instrumentation in country has remained neglected for a long time for various reasons. A National Policy and a renewed thrust in this regard is required.

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Role of Research Stations and Technical Universities

The role of Research Stations, Technical Universities and other such Institutes has to be re-defined. They must be involved more and more into the problems of national importance - such as those concerning the Water resources Development Projects. Some of the Research carried out in these institutions is of little value.

Linkages between various Institutions and Project Authorities

It is imperative that the linkages between various institutions which can play an important part in organizing and resolving the geotechnical problems of Water Resources Development in the Himalayan Region need to be re-defined. A deliberate attempt will have to be made to make them operational. Students working for their M.Tech and Ph.D programs should be encouraged to take up live geo-technical problems from on-going construction programs. Necessary facilities should be provided to them at site and design offices. To attract more and more students for taking up the practical problems, an incentive in some form may be devised.

Water Resources Development Projects and Environment

Unfortunately, the pace at which the development should have taken place has not happened. This is due to several reasons. One of them is the growing opposition to these projects. The dams which were called 'Temples of Modern India' not so long ago, are being criticized for various reasons.

A well organized campaign seems to have been launched to see that the development of the Water Resources particularly in the Himalayan region does not take place or at least gets delayed to the extent possible. The campaign includes several activities, one of them is to mislead people on technical grounds.

A hypothesis that has increasingly come to be used in defining high risk areas and predicting major earthquakes is that so called 'seismic gap theory'. The tectonic theory of earthquakes postulates that when plates move against each other, strains accumulate along their boundaries. These can be temporarily absorbed in crustal deformation and be locally manifested in landslides or through swarms of minor shocks which may be only precursors to a major rupture, that must follow to release the pent up energy as it builds up to a breaking point. Mapping these signals in time and space has provided useful clues to the future. It is however not possible to predict earthquakes with any certainty with today's state of the art.

However, if an attempt is to be made in this direction it should be made on scientific lines after looking at what all others have done in this direction, particularly the Japanese - as they get hit most - and where have they reached.

Reservoir induced seismicity

It is now accepted in several quarters that the creation of reservoirs leads to increased seismic activity around the reservoir. Detailed investigations of earthquakes in the vicinity of man-made reservoirs show that the earthquakes cannot be caused by impoundment of water as the stresses caused by the reservoir impoundment are much smaller compared to the stresses released by the earthquake. The reservoir impoundment only provides a trigger. The triggering is mainly caused by an increase in the pore water pressure. It is therefore necessary for the occurrence of a reservoir induced seismicity for the region to be stressed to a critical level before the impoundment of the lake. It also follows that due to increase in pore water pressure, the slippage occurs at a much lower level of strain and thus the possibility of a very large earthquake resulting from a large build up of strain gets greatly reduced.

It is also known that a thrust fault environment is not conducive to Reservoir induced Seismicity. Investigations at some sites have shown some reduction of seismicity consequent to reservoir impoundment as at Tarbela dam in Pakistan.

Higher dams are no less safe in a seismic region than shorter dams. Rock fill dams may at best deform, or slump but earthquake shocks will not break it. No dam, in fact, has ever failed due to earthquake, even though we have more than 650 records of earthquake of magnitude more than 5.

Role of Profssional Bodies

In the newspaper (Times of India New Delhi dated August 26th 2010) there was a headline "Activist ends 36-day fast to save Ganga". The complete news item which is self explanatory is reproduced below.

"Hardwar: Satisfied with the Union government's assurance to save Ganga and its tributaries, 78-year old environmentalist G D Aggarwal on Tuesday broke his 36-day old fast. He was offered juice and fruits by Union environment minister Jairam Ramesh and Uttarakhand CM Ramesh Pokhriyal Nishank at Matresadan Ashram in Hardwar.

The decision to end the fast was taken after Ramesh handed him over the Union government's letter scrapping all the hydropower projects on the Ganga in Uttarakhand. In the letter addressed to the former retired professor, chairman of the standing committee on group of ministers (GoM) Pranab Mukherjee assured him that all his genuine demands had been accepted. Before handing over the letter to the environmentalist, Ramesh read it out from the dais in the presence of over 500 people including social activists, seers and sadhus. After that Ramesh and Nishank offered him a glass of juice.

The GoM had last Saturday scrapped all hydropower projects including NTPC's 600 MW Loharinagpala project on Bhagirathi, a tributary of the Ganga. Aggarwal, however, said he would start taking normal diet only after these decisions were formally approved by National Ganga River Basin Authority headed by PM Manmohan Singh in its next meeting

Ramesh said he carried the letter to Aggarwal only after the permission of the PM. Ramesh said Union government had accepted all the demands of the environmentalist.

Some of the demands included scrapping of Loharinagpala project; non-resumption of production on Uttarakhand government owned Palamaneri and Bhairavghati hydro-power projects; declaration of 135 km-long Gangotri-Uttarkashi area as 'extremely ecologically sensitive zone' so that no projects could come up there.

The letter said Union Government has also agreed to include Aggarwal and five others, whose names would be recommended by the environmentalist, as non-official members of National Ganga River Basin Authority. Pranab also invited Aggarwal and five others to participate in the NGBRA's meeting to be held in the next three to four weeks."

468 Billion Litres Rainwater Wasted

There was a news item published in the Times of India, dated 27th August 2010 (Friday) with the headlines '468 Billion Litres Rainwater Wasted'

The water which flooded Delhi's streets this monsoon could have been harvested. It was stated that this monsoon with 681 mm rainfall recorded till Monday evening (23.08.2010), the total availability of water for rainwater harvesting was around 468 billion litres.

These figures were based on the calculations provided by Jyoti Sharma, Director, Forum for Organized Resource Conservation and Enhancement, and took into account built-up area, including roads, paved paths, roof tops etc., of 65% and a green area of 35%. It was assumed that in the built up area segment, each surface yielded a different amount of available water for harvesting and an average figure to give a 70% yield was assumed. From the green area a yield of 25% was expected. After accounting for evaporation, ground water percolation etc., they stated that the city could have safely harvested almost 470 billion litres of water.

It was stated in the said report that rainwater harvesting, if carried out on time, properly and by everyone, could also have saved Delhiites from a lot of misery resulting from water logging and submerged subways and underpasses.

Even if the figures given by the Organization can be challenged, the fact remains that there is a lot of potential in rain water harvesting and a lot of water which goes waste could be saved and stored for subsequent use.

The Truth about Privatisation of Hydro Power

The government regulations and the ADB report paint a very rosy picture of the privatization of hydro power scene in India. The facts are somewhat disheartening.

The August 2010 issue of *the energy business (vol. 02, issue 01)* have come out with a special report on the realities of the hydropower sector and talking about the Untapped Potential, they state that long gestation periods and lower rates of returns may be the reasons that are keeping private players away from tapping the country's large hydropower capabilities. It states,

"Power sector analysts in the country rue the fact that hydro power hasn't seen stellar growth despite India being endowed with vast hydro-potential. Let alone the government sector, even private players are yet to replicate the story of thermal power in hydro. As on June 30, 2010 the private sector's contribution to hydropower in the country is only 1,233 MWs."

The industry people say that the government does not pay the same attention to hydropower that it gives to ultra mega projects, thermal power plants, solar power and wind energy. The Government officials, on the other hand, dispute this and say that the government has taken various initiatives over the years to boost the growth of hydropower in the country. These include the mega power project policy, three-stage clearance, the 50,000 MW hydro initiative, Electricity Act 2003, Tariff Policy 2006 and Hydro policy 2008. Despite hydro projects being recognized as the most economic and clean source of power and having the obvious advantage of free fuel (water), the share of the hydro power has been continuously declining during the last three decades.

The ideal hydro-thermal mix should be in the ratio of 40:60. Hydro share has declined from 44 percent in 1970 to 25 percent in 2009. In 2004, the country's total hydro-power capacity was about 22,900 MW, which rose to 36,953 MW as on June 30, 2010 registering a modest growth of 25 percent over a period of six years. Of the total installed capacity of about 16.6 million MW, hydro power contributes merely 22 per cent with the rest being thermal.

At the end of 10th five year plan (2003-07), against a target of 14,000 MW, about 7,000 MW of hydropower capacity was added taking the total hydropower installed capacity in the country to 34,654 MW.

The government seems to have reconciled itself to the shortfall in capacity addition. For the 11th five year plan which ends in 2012, the government is expecting a capacity of 8,157 MW to be commissioned against the targeted 16,000 MW.

Currently, about14,000 MW of hydro projects are under execution in states, which have huge hydro potential. These projects may get commissioned by 2012 or in the 12th five year plan. These states are - Himachal Pradesh (2,973 Mw), Uttarakhand (1,520 Mw), Arunachal Pradesh (2,600 Mw) and Sikkim (1,799 Mw)

Full of Challenges

The hydropower sector is full of challenges despite policy initiatives brought in by the government. Hydropower projects involve various risks, complexities and unforeseen challenges. Hydro is capital intensive and requires an investment of about Rs. 5 crores per Mw compared with Rs. 3 - 4 crores per Mw for a thermal plant. It is time-intensive, offers a lower internal rate of return and has a long gestation period, while thermal plants can be put up in three to four years.

There are other practical difficulties faced by hydro power projects such as getting financial closure, which affects the project execution. According to Subhranshu Patnaik, executive director at Pricewaterhouse Coopers (PwC) hydro power is very different from thermal and every hydro power project is also different. Most hydro-projects are located in remote and inaccessible areas which pose challenges to the developer. Land acquisition is another challenge for hydro projects. At times, opposition by locals, anti-dam activists and civil societies affect hydro power projects. Environmental and forest clearances also put off the developers due to their lengthy and cumbersome process.

Besides the geological surprises can mar hydro projects any time, specially in the Himalayan ranges where the maximum potential lies and where the surprises are inevitable because of the geological complexities. Many government hydro projects in India are mired in inter-state disputes for water and power sharing, which act as an impediment to the development of hydro-potential. The example of the 450Mw Indrapuri hydroelectric power project of Jharkhand, Bihar and Uttar Pradesh can be sited as an example. The central government has been an enabler for the hydro-sector by making friendly policies, but the problem is that states are not in tune with the centre. There is increasing evidence of law and order problems and terrorist activities. Several unions do not allow the promoter and the contractor to work peacefully.

Hydropower Capacity Added Over the Years

Five Year Plan	Mw
End of 6th Plan (March 1985)	14,460.02
End of 7th Plan (March 1990)	18,307.63
End of 8th Plan (March 1997)	21,658.08
End of 9th Plan (March 2002)	26,268.76
End of 10th Plan (March 2007)	32,922.53
As on June 30, 2010	36,953.00
Target for 11th Plan (2007 - 12)	15,627.00

The Way Forward

An environment conducive to development by clearing impediments may be created. The Central government should prepare a detailed project report for each project, before putting them for bids, rather than preparing very sketchy feasibility reports. No worthwhile investigations are carried out while preparing these reports with the result that no planning for equipment, construction methodology and scheduling etc., can be done in advance. Each state has its own criteria and set of rules for bidding. Since the state governments charge upfront money for projects, they should also provide infrastructure facilities beforehand.

No additional taxes should be imposed in the post bid scenario, because these additions affect the economics and the viability of the project. Also, all clearances and in-principle approvals from the related ministries such as public works department, fisheries, forest and environment, defence, external affairs etc.. should be obtained by the government. Otherwise the developers are exposed to uncertainties and the associated cost burden and are often left in the lurch.

In the land acquisition process, the district administration should help in resolving the issues and convincing the public at large so that the delays are minimized. Hydroelectric projects involving submergence causes displacement of large populations and there too the government has to act. It should not only provide alternate accommodation and help them rehabilitate, it should convince people that their sacrifice is in a larger public interest.

The government has to act fast if it wants more private participation in this area, otherwise it might be too late.

Conclusions

We are fortunate to have been bestowed with plenty of water resource. This precious resource needs to be developed and managed. What has been done so far in the field of water resources development is very little compared to what remains to be done. Just one example of the potential of river Yamuna in Uttaranchal and Himachal Pradesh, will demonstrate our capability and what remains to be done.

The Yamuna river along with its numerous tributaries in Uttaranchal Himalaya constitute about 6000 MW of Hydel power potential hardly 10 per cent of which has been tapped so far. In a total stretch of about 62 km between Ichari in Tons valley and Khara power house in Yamuna valley through five run of the river schemes a total of 550 MW of hydropower is being generated by utilizing a gross available head of 240 m.

The Chibbro underground power house of the Yamuna Hydel scheme stage-II part-I on river Tons is the first underground power house cavern constructed in Himalaya in the Indian Subcontinent under not very favourable rock conditions. The head race tunnel of Yamuna hydel scheme stage-II Part-II also posed challenging tunnelling conditions on account of intersection of two regional thrusts namely Krol thrust (MBF) and Nahan Thrust and their intra-thrust zone traversing for about 1 km length and experience of squeezing and flowing conditions with rock closures in the tunnel. The tunnel was successfully driven in this weak zone by trifurcating it into three tunnels of 4.8 m diameter size each. The siphon tunnel below the Tons river from Chibbro underground powerhouse is also a unique feature constructed in the project. The Khara hydel scheme was constructed across the fragile Siwalik rocks comprising sand rock, boulder conglomerate, siltstone and claystone with water charged horizons to give rise to flowing and squeezing conditions during tunnelling. The 12 km long power channel was constructed in a hostile terrain with numerous streams and unfavourable unstable patches causing landslides and instability problem of cut slopes along its banks resulting into delays in commissioning of the project. Two important tectonic features passing through the Khara Hydel scheme are (a) Yamuna Tear a major fault with a sinistrial sense of movement showing a huge 4 km horizontal shift of Siwalik rocks along this fault and (b) Foot Hill Thrust (FHT) encountered during the excavation of Khara Power house and bypass channel which shows deposits of Indo-Gangetic alluvium overridden by middle Siwaliks. Neotectonic activity along Yamuna Tear has been witnessed in the area indicated by cracks and displacement of tiles in the lining of power channel after two years of the commissioning of Khara project.

Having done all this, we should have been leading the world. But we are not. If we have a problem today we still look to the west. Had we done our work systematically, instrumented our water resources development projects, analyzed our data and experience and involved our teaching and research institutes into this task of nation building, we would have been world leaders in this field.

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