Some Challenging Geotechnical Problems in River Valley Projects in India*

by

*Y.K.Murthy***

INTRODUCTION

I am very grateful to the Indian Geotechnical Society for the invitation to deliver the Address during the Annual General Session of the Society held at the University of Roorkee. I feel greatly honoured for this kind invitation.

Geotechnical Engineering mainly comprises of soil mechanics and rock mechanics. In the case of soils, the mass is heterogeneous and unisotropic. The inherent uncertainty is particularly characteristic of soil engineering. Since natural soils are extremely variable in their properties, the rational choice of suitable design parameters is generally the most difficult part of a design. The experiments done at the laboratory show much variations between their results and the actual behaviour at the site. There are considerable limitations for direct application of the results. The accuracy of the design method is also uncertain. Choice between the alternative designs or construction procedures is rarely clear cut.

However, attempts are made to express many of the uncertainties in terms of numerical probabilities thereby allowing quantification of judgement to some extent. It must be remembered that even the application of these parameters will have to be done with caution as many important aspects are omitted or treated very cursorily and are not exhaustive.

Similarly rock is one of the oldest constructional material known to man, and yet only till recently very little was known about its structural behaviour. The increasing size of civil engineering structures and the loads imposed on rock foundations have caused disasterous failures. In the design of structures to be founded on the rock, it is necessary to recognise that the foundation rock has definite limitations when used as an engineering material. A detailed in-depth study of these limitations is the essence of rock mechanics. Rock properties are controlled by the geological environment in which it exists and the geological history to which it has been subjected. The behaviour of the rock mass under loaded conditions is primarily dependent on the nature, orientation and inclination of structural discontinuities such as faults, joints which divide the rock material into a discontinuous system. Therefore, most essential requirement for determining the behaviour of the rock mass is the determination of the three-dimensional geometerical distribution of these structural discontinuities. There are no short cut methods in this process.

Another important factor in rock mechanics is the time-dependent deformation which can result from creep phenomenon continuing over long periods. This deformation is stress dependent and also influenced by seasonal variation of load, weathering etc. The precise assessment of this is extremely difficult, if not impossible. It is, therefore, difficult to reflect the true rock behaviour. Long-term in-situ testing to simulate the prototype structure loading conditions is not only impracticable but also expensive. Long-term laboratory testing would suffer from the same difficulties as in the case of in-situ testing. This method is unable to reflect the effects of any discontinuities in the rock mass.

It is also important to consider whether entire mass or at least a part of the foundation material is under plastic condition. Under plastic conditions, it is not possible to establish a correlation between stresses and deformations.

The new tool i.e. geo-mechanical models for analysing the behaviour of the rock under loaded condition has been greatly used. Even these models have serious limitations,for example, in a dam, the unknown factors are not the external loads but the resistance characteristics of the fractures and the joints in the rock. Though it is possible to make modification in the external loads while analysing but the resistance characteristics of the fractures, faults etc. of the mass cannot be modified. It will necessitate building a new model altogether.

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**Ex- Chairman, Central Water Commission, presently World Bank Consultant.

The fundamental influence of rock foundation on the static and dynamic behaviour of large concrete dams has been evaluated to some extent as a linear elastic continuum. In order to achieve a more realistic analytical information, a concerted effort is needed on for both numerical analysis techniques and geo-mechanical in-situ quantatitive characterisation of foundation rock mass. The object of dealing with the above mentioned aspects of geotechnical engineering is not to create a sense of uncertainty and fear in the minds of civil engineers engaged in the design and construction of large projects. The intention is to drive home the fact that this subject is not amenable to accurate mathematical evaluation.

With this background of the limitation of the acurate assessment of the behaviour of soils and rock, it is needless to emphasise the fact that in this field of Civil Engineering more than in any other field, sucess depends on practical experience. Large structures involved in major river valley projects comprising of unusual features call for extensive application of scientific investigations to design. Even the programme for the required investigations cannot be laid out nor the results interpreted wisely unless the engineer possesses the required amount of experience. That is why in the geotechnical engineering, the engineer is compelled to rely upon at least to some extent on the records of experience of others. If these records contain adequate details of the site conditions, they constitute certainly a storehouse of valuable information. It should be recognised that the theories of geotechnical engineering provide us only with working hypothesis because our knowledge of average physical properties of the mass and the orientation of the boundaries between the individual strata is always incomplete and utterly inadequate. In fact, in the design of structures and foundation "learn as we go" method is to be followed without any risk particularly in cases where the lower factor of safety than the factor which is customarily required in other fields of engineering is to be adopted.

CASE HISTORIES

In this lecture I would illustrate with slides some of the major river valley projects which have been recently completed or are under construction. Projects that I have chosen for today's illustration are the ones in which, I was personally associated, are as follows :

North (1) Baira Siul Hydro-Electric Project
- (a) Power Intake.

East (1) Loktak Hydro-Electric Project
- (a) Power Channel.
- (b) Penstocks Slope.

West (1) Kadana Dam Foundation.
(2) Dantiwada Reservoir
(3) Aran Dam Slope Failure.

South (1) Kalinadi Hydro-Electric Project
- (a) Supa Dam.
- (b) Hill Slope Projection for Nagjhari Power Station.

I will give below a short description of these projects and also a brief resume of the challenging geo-technical problems which have been baffling the ingeneuity of the expertise in the country. A few slides on each of these projects will further illustrate the problems more vividly.

BAIRA SIUL HYDRO-ELECTRIC PROJECT

The project is unique in the sense that it utilises the waters of three rivers for power generation through a single water conductor system and a single power house. It comprises of two weirs constructed across the two rivers viz. Siul and Baledh and a dam across the third *viz.* the Baira River. The waters of the Baledh will be taken through a feeder tunnel about 8 kms. long to the reservoir formed by the dam across Baira river whereas the waters of the Siul river would be made to drop down 90 m. through a drop shaft into the main head race tunnel of 7.63 kms. length, which takes off from Baira reservoir. The main head race tunnel terminates at the surge shaft from where three penstocks 473 m long take off to feed three units of 60 MW each. The head acting on the turbines is about 300 m.

The main geotechnical problems encountered in the project are :

(i) the hill slope consisting of debris overlying the phyllite rock above the power house is unstable. Heavy

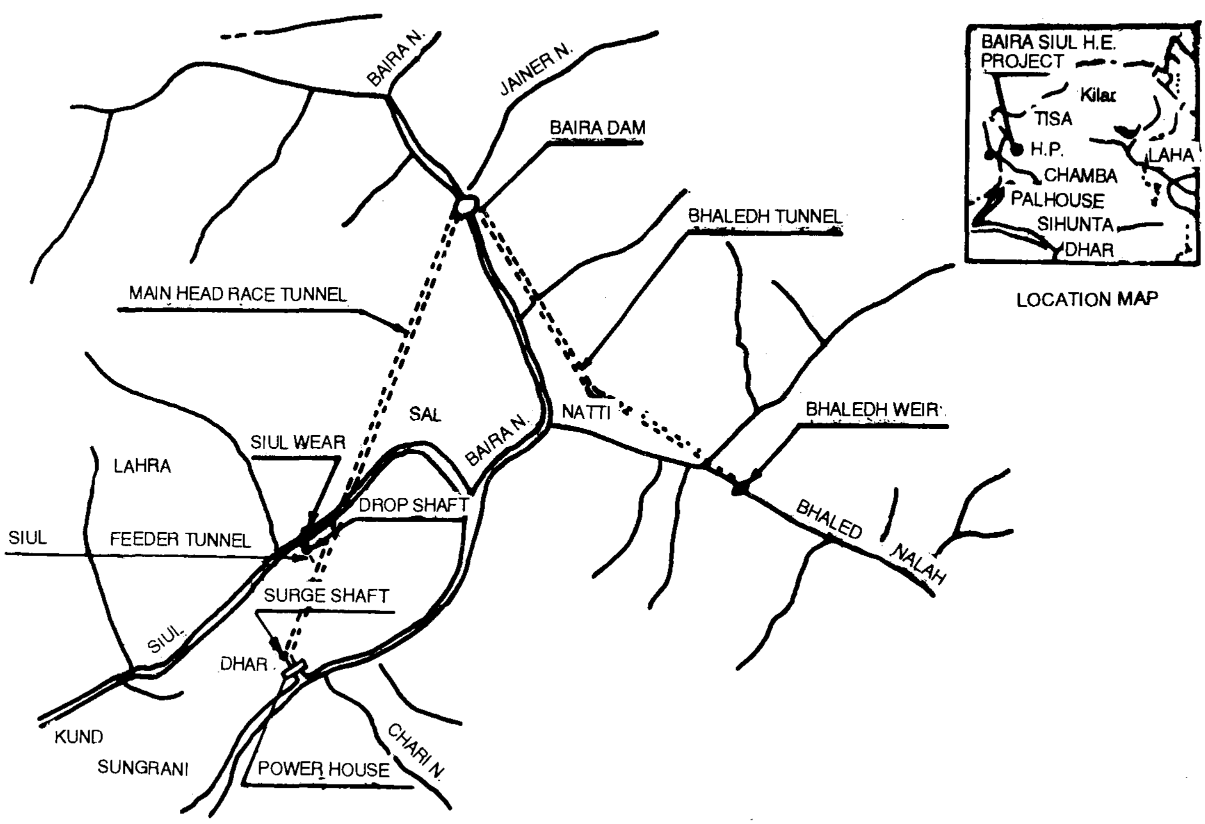

FIGURE 1 Layout plan of Baira Siul Hydro-electric Project

landslide occurred in the year 1976 bringing large mass of debris and filling up the power house area under construction.

(ii) in the year 1978 another 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 bed by about 10 m. along its length upstream of the dam axis.

(iii) after a heavy downpour in March 1979, the slump movement which had occurred in the previous year had extended towards the upstream in the overburden on the right side abutment hill of the Baira Dam. The zone of the movement extends up to approximately the point above the power intake structure of the Head Race Tunnel. A problem had arisen whether to shift the intake portion upstream to a stabler location. A detailed study of the contour of the slump area was made and was found that the present position is more appropriate and was retained with some protective works.

LOKTAK HYDRO-ELECTRIC PROJECT

The Loktak Hydro-Electric Project in Manipur envisages diversion of 60 m^3/sec of water from the existing Loktak Lake, to supply 16.8m^3/sec for lift irrigation and the balance for power generation. The power house is located in the Leimatak valley. The water from the channel is lead into a 6.5km. long tunnel terminating in a surge shaft from where 3 penstocks take off to feed 3 units of 35 MW each utilising a gross head of 312 m.

The main geological features of the area consist of thin bands of argillaceous sandstone and silt stones. In the area where the power channel passes, the overburden consists of organic clay, which has the characteristics similar to marine clay.

(a) Power channel

The power channel which leads the waters from the lake of the outlet of the tunnel is 2.25 km long and is designed to carry 60 m^3/sec. The shear properties of the organic clay is so low, the side slopes of the channel have been sloughing down. In some reaches extensive cracks have developed longitudinally and gradually there has been heaving of the channel bed by 1/½ to 2 mts. Several methods of protection of the slope such as stone pitching with gravel and filter backing, and driving of reamed piles to improve the shear characteristics of the clay

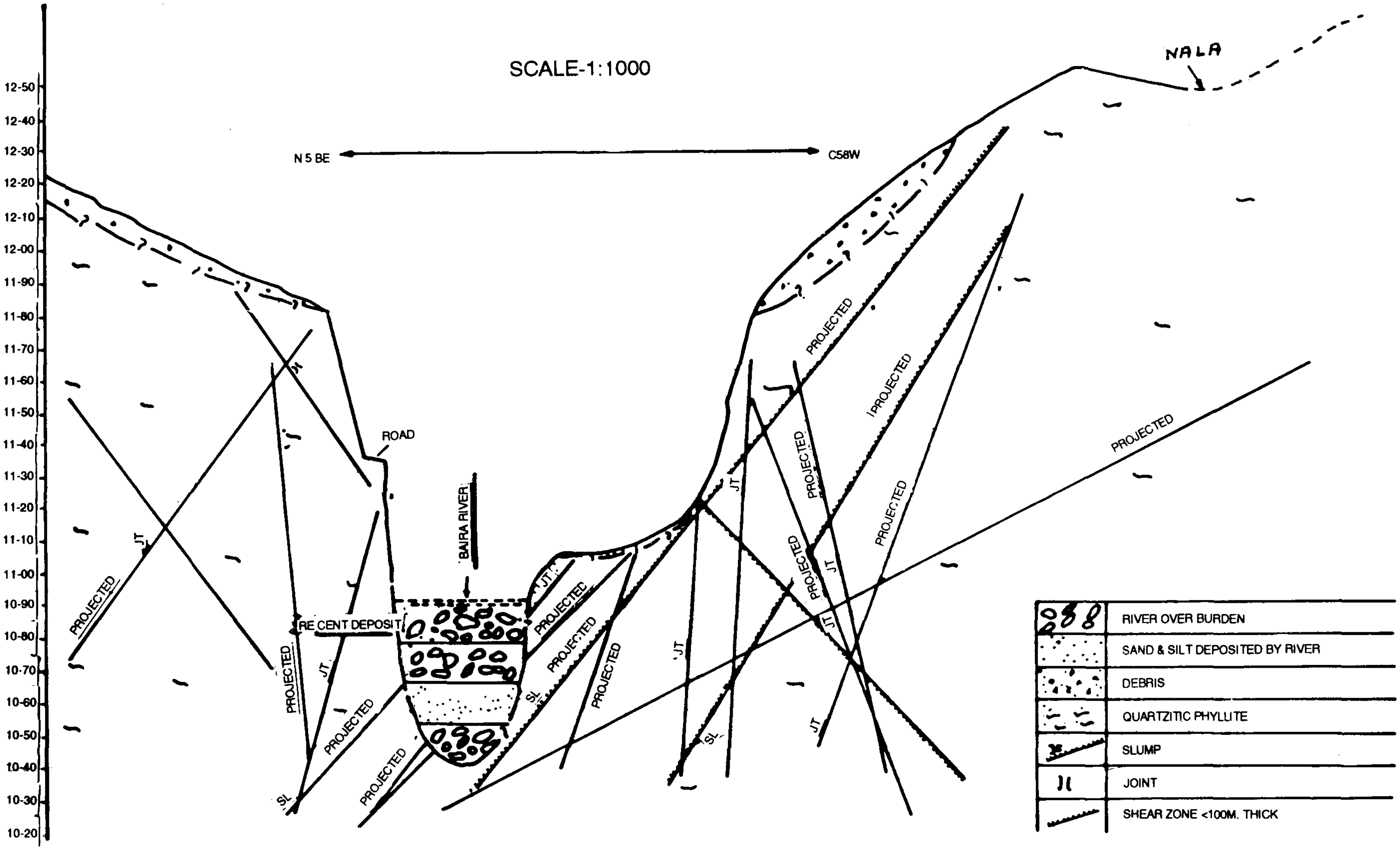

FIGURE 2 Baira Siul Hydel Project H.P. Geological section along 2-2

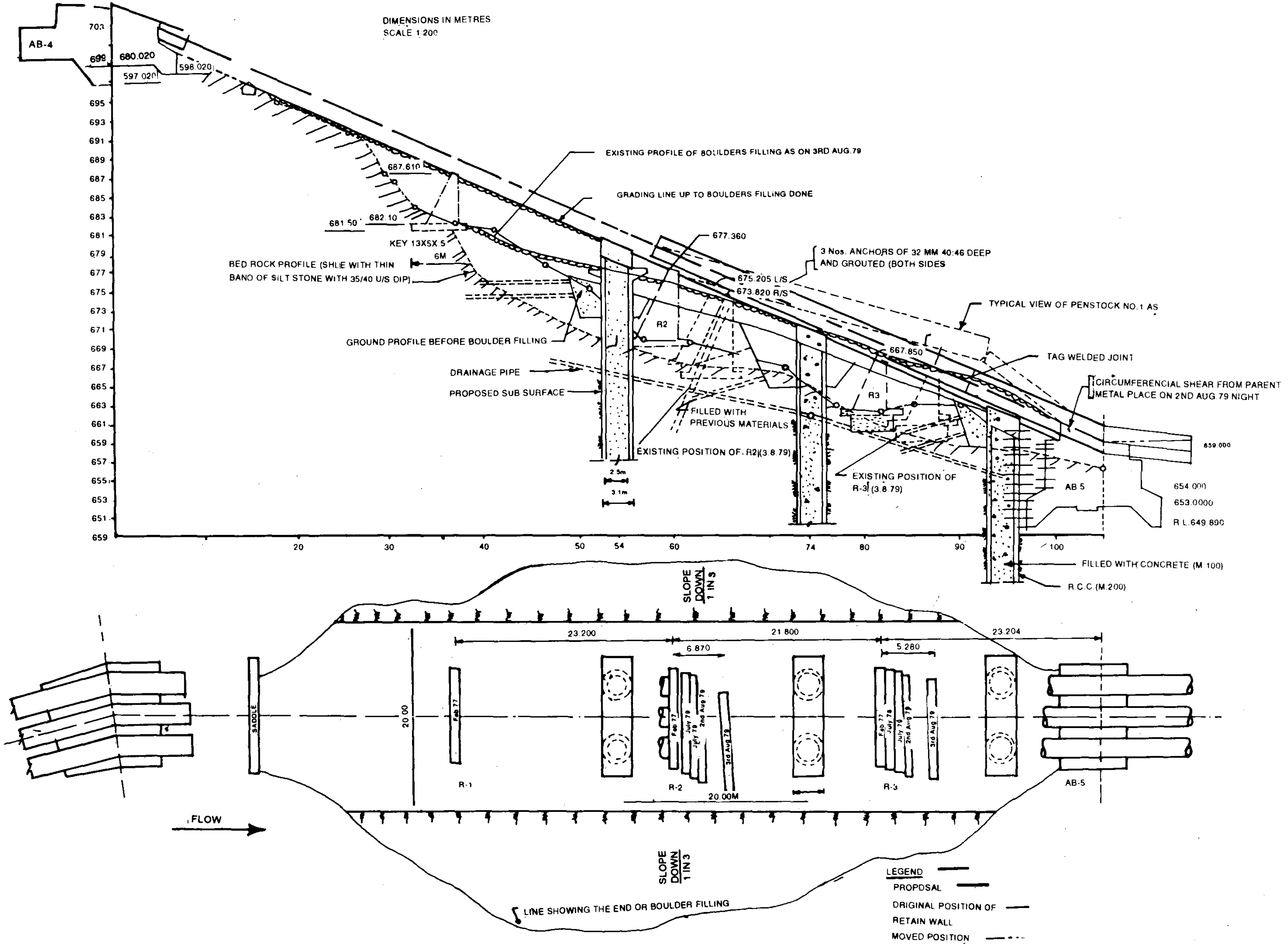

FIGURE 3 Loktak Hydro-electric Project, Manipur, Plan and longitudinal section of reach anchor block (4-5), showing the details of movement of retaining walls and proposed shafts and drainage

material in the slide slope of the canal, were tried.

Some of the slides show the extensive damages that have occurred in the power channel. In the reach where there is a cut-cover conduit, 1223 km long, number of alternative methods of excavation in the clay strata were tried and ultimately it was only possible to drive two rows of sheet piles heavily strutted, which could retain the excavated walls of the channel.

(b) Penstock slope

In the penstock alignment between Anchor Blocks (AB) 4 and 5, the lithology exposed are soft, crumbly saturated shales and slope wash materials. The shales are carbonaceous.

The penstocks are supported on 12 Anchor Blocks and 68 saddles. The reach between Anchor Block I to Anchor Block 4 and the reach between Anchor Block 5 and Anchor Block 8 have been completed. The strata particularly between Anchor Block 4 and 5 and 11 and 12 are highly unstable and often has been subjected to loughing resulting in the movement of the saddles.

In the years 1973 and 1974, massive subsidence in the area downstream of Anchor Block 5 occurred. As a result of the subsidence the saddle supports between AB-5 and AB-6 got moved laterally towards the right by about 2 m. Therefore, the saddle supports between AB-4 and AB-5 were reviewed and the saddles were proposed to be founded on the rock. Accordingly, three saddle-cum-retaining walls were constructed in 1976-77 and the reach from the excavated ground levels upto the penstock grade levels were filled with boulders with a carpet of shingle on the excavated ground level.

During the monsoon of 1977, there was settlement of boulder filling on the downstream of the retaining wall-cum-saddle support and upheavel between retaining wall 3 and Anchor Block 5. As a result of this, formation level between retaining wall 3 and Anchor Block 5 touched the penstock bottom and continued to rise, resulting in lifting of the penstock No. 1 by about 100 mm. Retaining walls 2 and 3 also moved laterally towards the right by 230 mm and 165 mm respectively.

Recently, after the rains, the following damages have taken place in the reach during August, 1979:

(i) Further longitudinal lateral and vertical movements of retaining walls 2 and 3.

(ii) subsidence of the boulder filling dowstream of retaining wall and upheavel between retaining wall 3 and Anchor Block 5.

(iii) high pressures are built up around anchor block No. 5 resulting in buckling and shearing off of penstock pipes; and

(iv) cracks in the surrounding area of Anchor Block 4 and 5.

Remedial measures to get over the problems have been seriously considered.

KADANA DAM

The dam is under completion. It consists of masonry gravity dam comprising of spillway and power dam and an Earth Dam on the left flank. Height of the dam is 65.8 m in the masonry portion and 32.9 m in the earth dam reach. It is a multipurpose project with an installed power capacity of 4 X 60 MW and an irrigated area of 7,80,000 acres.

The general geology of the dam site comprises quartz-mica-schists, mica-schists and phyllites. A number of faults have been encountered below the final foundation grade of the blocks, which are present in almost all the blocks of the gravity dam. The faults have a general dip towards the right bank as they continue towards the right abutment and they are met at deeper and deeper levels. The fault zones consist of clay and braccia and other fractured materials and vary in thickness from 10 cms to 40 cms. In the tranverse direction, the fault is continuous from upstream to downstream of the dam section. The materials of the shear zones have very low shear properties and therefore the zones present potential sliding planes. Sliding analysis on these blocks have shown very low shear friction factor of safety against sliding. An earthquake acceleration of 0.1g has also been considered in the stability analysis. In order to improve the resistance, shear keys were required to be provided intercepting the shear fault planes. A number of drifts had to be excavated in such a way that they will extend atleast 1 m both on top and bottom and sound rock. They are back filled with concrete to form shear keys. The total length of the drifts provided in all the dam blocks is about 300 m.

In addition to the provision of drifts, the following measures are adopted:

(i) provision of upstream impervious blanket about 60 m wide with a minimum thickness of 2 m along the entire

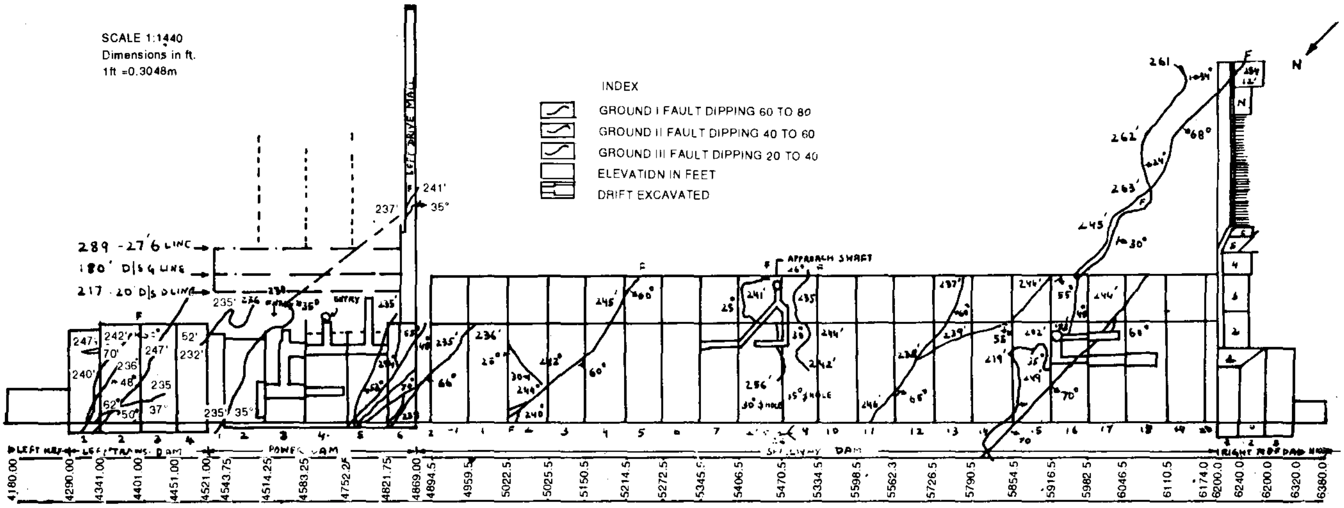

FIGURE 4 Kadana Reservoir Project, Plan showing the disposition of faults in the foundations of masonry dam and appurtenant structures

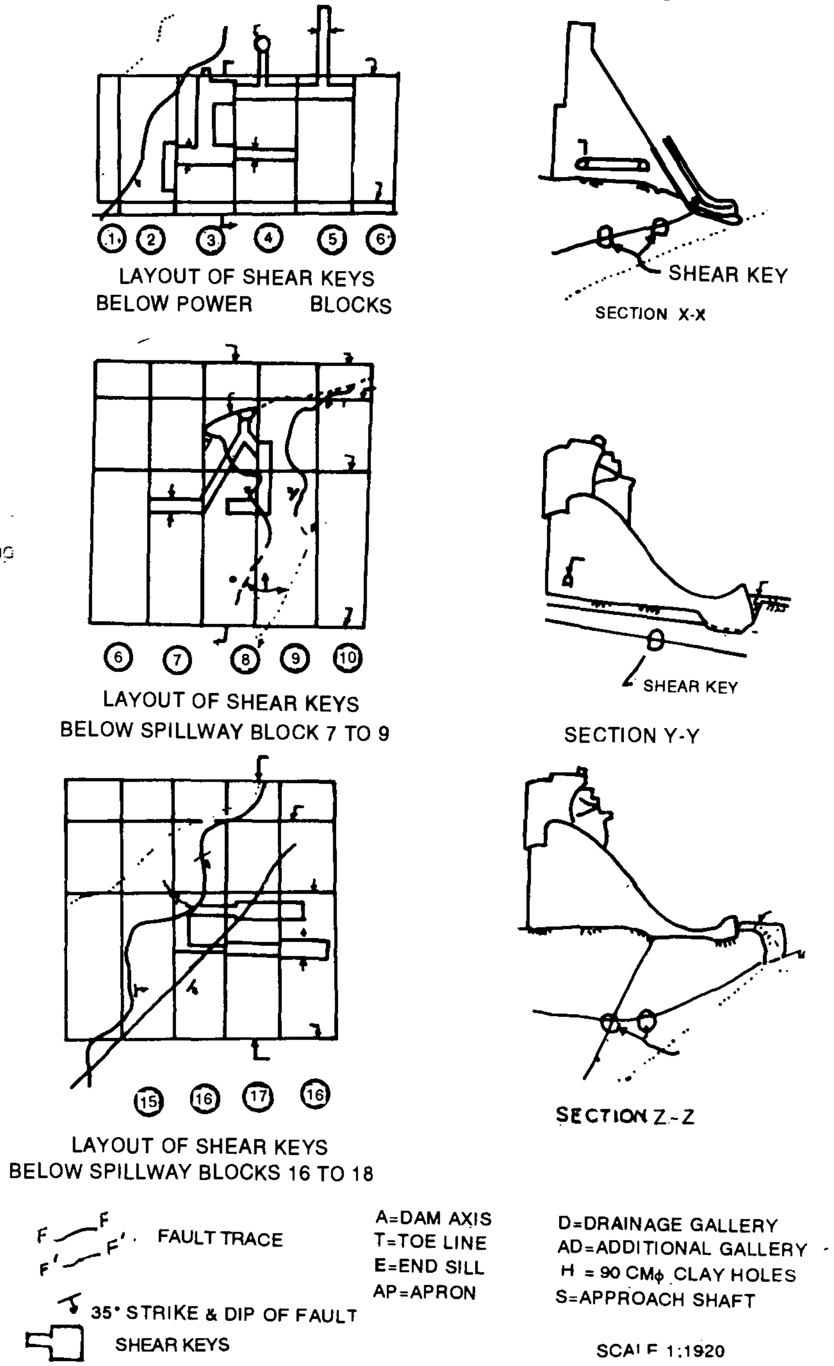

FIGURE 5 Shear keys for treatment of low angle faults below foundations of Kadana dam

length of the masonry dam.

(ii) as there was fractured rock above and below the fault plane, it was necessary to wash and grout the rock through the drifts.

(iii) in order to have adequate grouting and drainage additional galleries downstream of the upstream gallery have been provided.

With this treatment, the safety against sliding has improved to the desired and acceptable value.

DANTIWADA RESERVOIR

Dantiwada (Banas) Irrigation Project is located across the river Banas near village Dantiwada in Banskanta district of Gujarat State. The dam consists of a composite structure with a masonry spillway in the river gorge (masonry dam 1073' (328 m)) flanked on either side by embankment of aggregate length of 14,938' (4550 m). The masonry dam is about 135' (41.1 m) high and the embankment 120' (36.6 m) above the river bed. The spillway consists of 11 radial gates of size 41'X27' (12.5 m X 8.25 m) with discharging capacity of 2,35,000 cusecs (6680 m^3/sec).

The project benefits irrigation of 110,000 acres (44,570 hectares). The project was started in 1959 and completed in 1965. The crest gates were installed in 1969.

The Breach

A breach of the reservoir had taken place at a location near old village Ranavas. A deep cut to a depth of 15'(4.6m) to 25' (7.6m), width varying from 300' (91 m) to 500' (153 m) had occurred. A study of the banks revealed that the soil strata consisted of fine sand mixed with nodules of sand to a depth of 15'(4.6m) to 20' (6.1 m). The base of the breach has exposed a very friable sand-rock which displayed stratification, horizontal bedding and a dip of about 25° towards the east, i.e. towards the left bank of the breach section.

Probable reasons for the breach

The manner in which the breach has occurred in the periphery of reservoir indicates that this could be due to the following:

(i) impingement of sudden wave due to the breaches in west Banas Dam located upstream.

(ii) piping due to the seepage flow through the lenticular sand-rocks met in the foundation in the breach area.

(iii) wave action at the maximum water level at the point of the breach.

In the statistics of dam failures, perhaps we have not come across a case of reservoir breach as it had happened in Dantiwada.

ARAN EARTH DAM

Recent failure of Aran Earth Dam in Maharashtra during April 1978 is a classic foundation failure generated by seepage pressure built up underneath the silty clayey soil mantle which formed the foundation and the reduction of shear strength due to prolonged saturation of soil media forming the base of the dam. The Earth Dam has a maximum height of 30.3 metres above the river bed and a length of 775 metres and was completed by June 1977. The reservoir was impounded thereafter and full reservoir level appears to have been maintained since then. On 3rd April 1978, downstream slope of the dam for a crest length of 150 metres width, suddenly slipped which extended to 200 metres width at the bottom in the downstream. The maximum settlement of the dam at the slip section was about 6.3 metres.

It is inferred that seepage accompanied by high pore pressure built up due to prolonged saturation of weak silty clayey soil in the foundation reduced the shear strength. The fill must have sheared and glided along the soft layer and ultimately slumped in the process. This dam in spite of its moderate height was not instrumented. Had adequate instrumentation been done and monitored, advance action could have been taken to remedy the distress problem and such failure could have been avoided.

KALINADI HYDRO-ELECTRIC PROJECT

(1) Supa Dam Foundation:

Supa dam is a storage dam, which forms the backbone of the Kalinadi Hydro-Electric Project. The reservoir feeds a series of power stations in a cascade practically up to the point where Kalinadi joins the Arabian Sea. The reservoir formed by the dam at FRL will have a gross storage of 4400 X 10^6 m^3 and a live storage of 3977 X 10^6 m^3. Maximum height of the concrete gravity dam above the deepest foundations will be 101 m. A power house with two generating units of 50 MW is located at the toe of the dam.

Geology of the Dam Site:

The dam site area consist of Dharwarian meta-sediments-predominant unit being the banded magnetite quartzite. The river bed has outcrops of fresh and hard rock while in the abutments, the rock mass is highly folded and jointed containing at places weathered pockets and is covered with thick soil cover. The significant feature is that the rock mass is traversed by a few prominent, steeply dipping meta-dolerite dykes. These dykes are fresh in the river bed but are reduced to pulpy clay at higher elevations in the flanks.

There are also numerous shear zones containing fragmented soft material met with in the foundation area. Besides, there are weathered and weak pockets of varying dimensions at the heel and toe regions of the left flank blocks. The average modulus of deformation of the general foundation is quite low being 0.3 X 10^5 kg/ cm^2

The foundation treatment is posing a challenging problem. It has taken considerable time to think of the various alternatives of treatment for each block. In fact each block, out of the 22 blocks has some unique feature either at the toe or heel or in the central portion of the dam width.

(2) Nagjhari Power House Slope:

Nagjhari Power House is designed to have six units of 135 MW in the ultimate stage. At present 3 units are being installed out of which erection of 2 of them has been completed.

The power house is located on the right bank of the Kali river and involves cutting of the toe of the slopes almost vertically to a height of 40m to 50m in the deepest portion. The hillock has an average slope of the order of 50° and is about 330 m high. Thickness of the soil cover varies from 3 m to 10 m on the hill slope and the material is generally reddish sand to silt mixed with talus. Some of the large sized boulders stand precariously perched along the slopes. The slope is drained by two prominent nalas on either side.

The earliest landslips occurred in 1973 which was followed by a massive slip in 1975 as a result of unusually heavy rainfall. The slide brought nearly 50,000 m^3 of debris from the hill slope. A large part of the debris found its way into the power house excavated pit. Surface perched boulders caused constant threat to the safe working at the toe.

A systematic study of each case was undertaken and a scheme of drainage, rock pitching, rock fall barriers, anchoring by perfo-bolts, prestressed anchors and retaining structures were adopted to arrest the slides and rock falls from the hill slope.

TUNNELLING IN WEAK ROCKS

Another subject of geo-technical engineering which is posing challenging problem is tunnelling in weak rocks. Recent projects in the Himalayan Region such as Giri Hydel Project in H.P., Yamuna Hydro-Electric Project in U.P. have confronted us with most intriguing phenomenon. Weak rocks of the Sub-Himalayan region in which these tunnels are located has to be proceeded with unprecedented caution. Whereas the tunnelling in Giri Project has been accomplished, the tunnelling in Yamuna Project still remains a challenging problem.

In the case of Giri Hydel Project, 7.2 km long tunnel in its traverse encountered several fault zones and one major intra-thrust zone. Geological formations were mainly slates boulder slates, carbonaceous slates, phyllites, limestone and olive green slates. During the tunnelling operations, heavy rock closures were experienced in certain reaches of the tunnel. This was reflected in large scale deformation of steel supports of the tunnel. These closures were monitored by suitable instruments with the help of Central Mining Research Station, Dhanbad. Closure experiments conducted in a sample drift gave very valuable data. It was observed that the maximum closures ranging from 100 mm to 260 mm were noticed in a period of 276 days to 500 days. A critical study of the data indicates that the closure curves tend to flatten after a period of 90 days to 150 days.

There were differing views whether a rigid support on flexible support permitting substantial deformation to take place before permanent concrete lining is placed, should be preferred. The nature and the magnitude of the loads involved precluded the first alternative which was tried in certain reaches but given up after failure, creating more problems for dismantling. In the light of the experiments, the concrete lining in these reaches commenced only after allowing for the time gap for the dissipation of the rock loads and for rock stabilization to the extent possible. In certain reaches where the tunnel was already excavated, it became necessary to reduce the internal diameter of the tunnel from 3.66 m to 3.36 m to accommodate the heavy closures and obtain the minimum concrete lining thickness required. This also points to the need to develop a support system which can yield in a controlled way upto 20 per cent of the dia for use in such poor rock conditions.

It should also be remembered that such large convergences give rise to a large zone of highly dilated rock around the tunnel, which, if not grouted could become unstable when disturbed by shocks or ground water or when the tunnel carries water. An incident in the Giri Tunnel illustrates this point. The phenomenon was revealed when another part of the tunnel was being excavated from the two faces which were approaching each other. On one face excavation had been stopped and after convergence had substantially ceased with the installation of the steel rib supports in place, unreinforced concrete lining was cast to an internal diameters of 3.66 m but with the invert section omitted and without back grouting, Observations of the invert gap over several weeks showed no closure. But when the excavation of the other face was approaching 100 m towards the concreted length of tunnel, the concrete lining suddenly developed cracks and the closure was reactivated as the excavation by blasting from the other face came within about 50 m and then ceased. Fortunately such cracks in the plain concrete lining due to these movements were not deep. These cracks also stabilised after their formation. Before putting the tunnel into operation of the plant, these were treated with epoxy.

These studies have given an insight into the behaviour of the tunnels in weak rocks under high over burden and will influence greatly the future tunnel design practices. It would have been much more valuable if the instruments were obtained and embedded in the tunnel lining to monitor the long-term behaviour and development of loads if any.

CONCLUSIONS

It is in this context, that these challenging problems, though exciting, require concerted efforts on the part of the Engineers to undertake adequate investigations, collect data and analyse and apply appropriately to each problem and derive the solution on the basis of the long experience and the intutive judgement. In order to facilitate this, the following suggestions are made to be treated as only a reminder to the Civil Engineering:

(1) Geotechnical investigation must start with a review of the local geology, general in character, if that is sufficient. Otherwise, detailed geological features should be identified by a competent Engineering Geologist. Every Civil Engineer should be given a sound introduction to elementary geological principles in his basic training.

(2) Special courses in Geotechnical Engineering, especially Rock Mechanics should be organised and competent engineers should be deputed for training.

(3) Instruments should be installed to study the behaviour of the structures founded on weak foundations. Compilation of data from the instruments and analysis should be carried out periodically and an evaluation of the behaviour recorded and published.

(4) A close liaison between the research worker, designer and construction engineer should be established in the study of the geotechnical aspects and its impact on the behaviour of the structures.

(5) When a project goes into construction geotechnical engineers should not regard their own work as complete until they have done everything possible to ensure that all the geotechnical features are incorporated in the construction drawings, with most accurate delineation of the geological conditions.

(6) Highest priority should be given to the preparation and publication of case histories of experiences encountered during the execution of the project. Such information will be of special value in most of our developing countries. No doubt a large number of papers are published in various journals devoted to Geotechnical engineering, but usually they are "theoretical" in nature, arising mainly from post graduate studies from the Engineering Institutes. These are very valuable and they are needed. But they lack balancing amount of information from the field that is assuming serious proportions.

In conclusion, I would quote from an eminent Geologist who said:

"The successful Engineer of the future should know not only how to locate his work, but how to locate it so that Nature will aid him in its building and take it under her protection. Too late, he may know that Nature has resented his intrusion and in spite of his efforts, is surely undoing his work.".