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Crucial Issues In Geotechnical Engineering of Water Resource Projects

by

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Introduction

INDIA has immense hydroelectric potential of nearly 100 million KW at 60 percent load factor. It also has immense water potential of nearly 1800 Km^3, but is predominantly concentrated during 2 to 3 months of the year. This combined with high evapo-transpiration rates requires large conservation structures for storing water. Further while rainfall is heavily confined to certain areas, about 40 percent of the area of India is subject to drought, requiring long distance transfer of water for ensuring more reliable and substantial relief.

Hydrolectric power is economical and renewable with least damage potential to environment. However the development of water resources in general, and hydroelectric projects in particular, has been slow. The main cause of the slow development has been the large variation between the estimated and completion time and cost of the projects. There are several reasons for slow pace of development and these have been investigated by several committees. A Committee set up by the Government of India to go into the causes of the rise in costs concluded that the single major factor of cost escalation could be identified as attributable to inadequate investigations. The committee had also attributed some of the cost increases to inadequate provisions, though it could be argued that *some part of this could again be caused by insufficient* engineering appreciation of the site conditions. Changed conditions also result in extension of completion time of projects which in turn lead to escalation in cost. In fact the major criticism levelled against river valley projects is that of delay which upsets the planning process for development.

Quality and Quantity of Investigations

It must be recognised that there are great difficulties in investigations of river valley projects. Occasionally dams may be several kilometres long whereas tunnels and canals are invariably long involving wide variation in geotechnical conditions. It is essential therefore to have a preliminary

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study of the site by a team of geotechnical engineers, engineering geologists and general civil engineers to make an assessment of hazard at the site which may rule out the site or affect the priorities of site selection. There are major hazards like extreme karsticity, major landslides, deep loose cohesionless deposits in earthquake zone, active faults, long tunnels in potential squeezing grounds etc. In treatable hazards, one can include faults and seams in rock. However, the investigations necessary for identifying and classifying these hazards require considerable experience and expertise. For many projects some significant hazards were revealed during investigations and study but were not given due attention at that time.

In many cases the investigations are left to the detailed design stage and the project is cleared. But during construction, it is found that they are of much greater significance than at first thought. Central Water Commission is quite insistent in demanding more investigations at the clearance stage of the major projects but there is immense pressure to clear them to avoid further delays with an understanding that requisite investigations would be done at the time of pre-construction stage. It is generally accepted that the investigations of projects for investment decisions should be of such a standard as to limit the variations in quantities to about 10 to 15 percent. It is here that a considerable judgement is required from civil engineers with geotechnical background and engineering geologists.

It would be valuable to review two or three of our recent projects in this light.

Salal Hydro Electric Project utilises a 100m drop in the last reach of the Chenab before it enters the plains. Several alternatives were studied since 1956 to develop the drop in this reach. It was finally decided to utilise the drop by building a dam located in Dyangarh loop to take advantage of the loop in planning the diversion arrangements and spillway as shown in Fig. 1. It was also assumed that the ridge in the loop could be retained in a large measure.

Drifts were made in the ridge and they indicated that cross shear seams dipping downstream could lead to problems of stability. However, it was thought that this could be solved after determining their shearing strength. In the meantime the geologists had projected wide shear zones in the river bed both in the upstream and downstream limb of the loop. Himalayan shear zones can be wide but they may not in many cases create problems for the foundations. Some of the shear zones are only sheared rock but with rock to rock contact in the body of the zone and thin seams at boundaries.

It was difficult to establish the properties of the shear zone in the river bed except through a drift crossing the river bed which would take

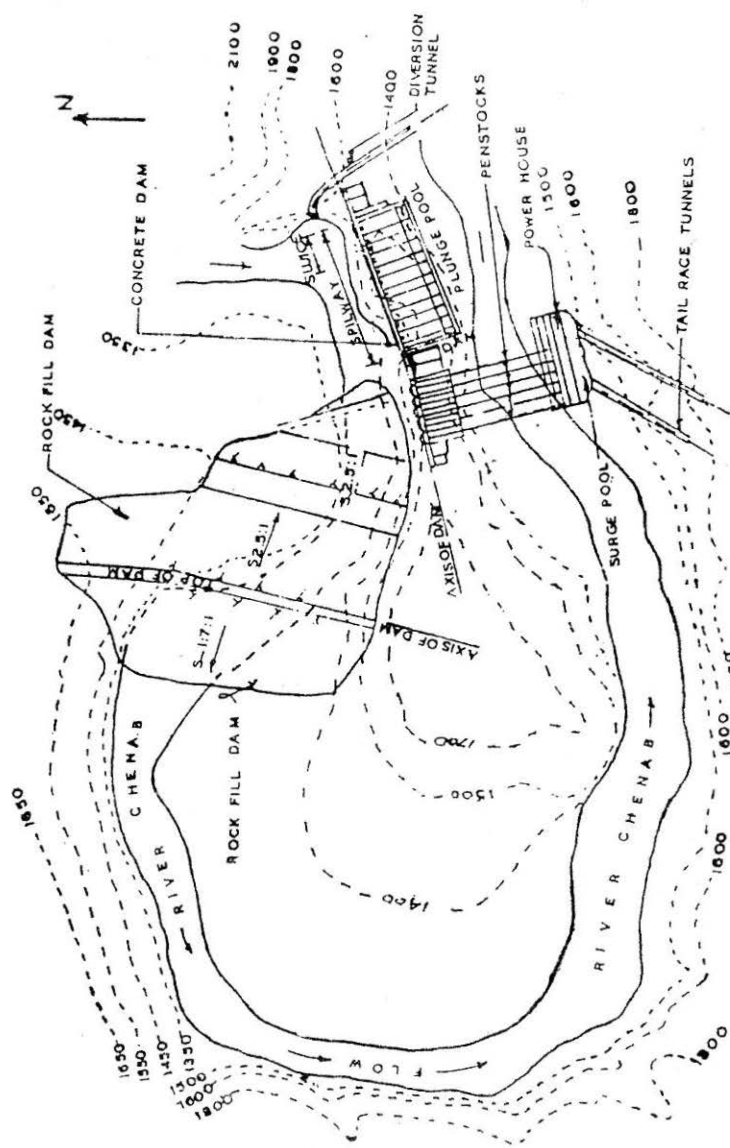

FIGURE 1 General Layout of Salal H.E. Project

considerable time. The design of the concrete dam therefore could not be proceeded with for some time till these could be finalised considering the importance of examining the stability along cross shear seam and through the wide shear zone in the river. It was, therefore, decided to base the designs on indirect test by conducting them in the shear zone material in a drift made through the extension of the zone in the left abutment. The insitu static modulus was found to be moderate (0.05×10^5 kg/cm^2) as revealed by the seismic wave velocity. Seismic wave velocity measured across the river from bank to bank gave high velocity indicating either the narrowness of the zone or its weak characteristics. Large scale insitu shear tests in drifts indicated angle of internal friction along cross shear seams of the order of 41°. Examination of the sheared surface in these tests indicated cutting across of rock in appreciable portions due to asperities.

With this, confidence was gained that the dam could be founded on the last daylighting 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 Fig. 2. Finite Element Analysis carried out indicated that the factors of safety across the assumed planes of failure could be brought to acceptable values and the dam design was finalised along these lines. The provisions of concrete around the penstocks cutting across the valley, a compacted rockfill buttress beam at El 420 m to El 470 m towards the abutments were the additional measures provided. The time required for

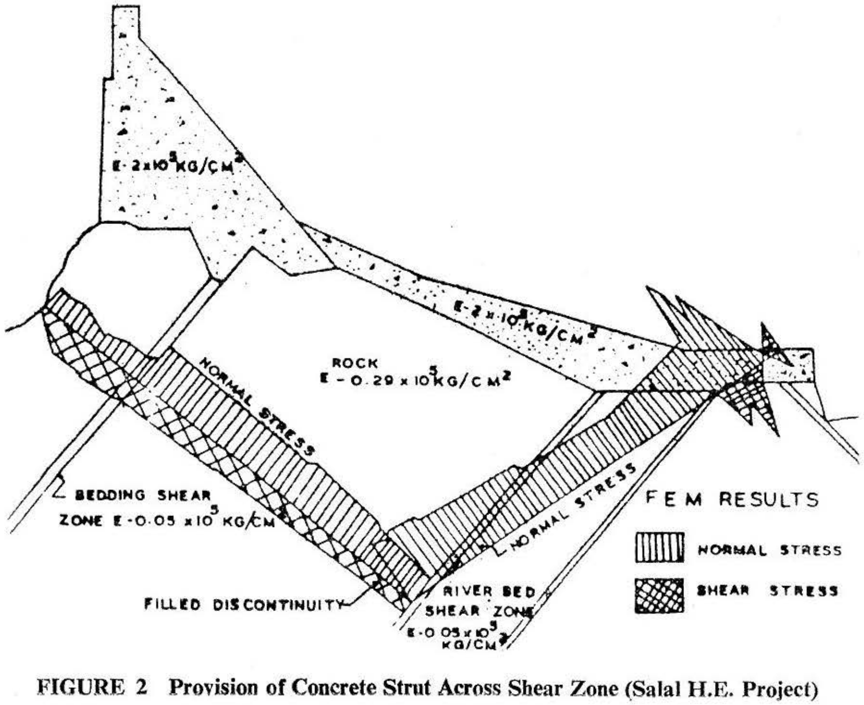

FIGURE 2 Provision of Concrete Strut Across Shear Zone (Salal H.E. Project)

these studies and the climate of uncertainties created during the period caused considerble delay.

Prioritisation of River Valley Projects

The experience of Salal H.E. Project also poses the question of mechanism of inter selection of Hydroelectric projects particularly in the Himalayas. The question arises whether projects where uncertainties are of less significance should not be given importance. It is evident that particularly in the Himalayas, there is an important role for the engineering geologists and geotechnical engineers in the planning for develpment and selection of projects.

If time and its close corollary costs have to be controlled, it is obvious that some mechanism has to be introduced regarding project selection and priorities. The question would arise whether Dulhasti Project with a low diversion dam and a long tunnel in generally good rock could not have been preferred to Salal Project. Experience at Chukha H.E. Project shows that projects selected on the basis of good geotechnical set up can be built in reasonable time and costs. With a number of projects in various geological set up, now built in India, it should be possible to make a risk analysis of the sites and include this in the project feasibility report. This could help in taking more rational decisions regarding project selection.

Evolution of Design During Construction of River Valley Projects

The civil engineering components in major river valley projects which require detailed designs are:

(1) Dams for storing water or a barrage for its diversion;

(2) Water conductor system consisting of an open channel or an underground tunnel

(3) Surge tank and forebay

(4) Pressure tunnel or a power penstock

(5) Surface or underground power house

(6) Tail race tunnel or a channel.

For project planning and design of various components, the input information is by way of field/laboratory investigations. The process of finalisation of a design for a structure associated with a river valley project is different than that is adopted for the usual civil engineering structures.

One major difference is that the investigations are never complete till the construction of the project is completed. In some cases design studies may continue even after completion of the project. The information revealed by way of new features encountered during construction often necessitates fresh design evaluation or at times major changes.

On the basis of geotechnical information available before taking up of constructions, construction stage designs and drawings are finalised and issued to the project authorities. The construction is planned and taken up on the basis of these designs and drawings. The information revealed during construction is reviewed by the designers in the course of their field visits, by the geologists during the course of their daily involvement in the field and by the field designers and the construction civil engineers during their continued involvement. When geological surprises arise, a designer has to review and supply the revised design accounting for the conditions as revealed during construction. A signal to this effect will be either in the form of minor incidents and distress while implementing the earlier proposed approach or some glaring feature which could jeopardise the safety of structure during operation. On the other hand, the conditions may be better than that were forecast earlier. An important question likely to be posed to a Project Designer is that :

> Is it not possible to have a perfect design input by way of extensive geotechnical investigations so that a design could be completed in one step without making changes during construction thereby to avoid cost and time overruns?

Answer to this question lies in replying two further questions;

(1) How much is a complete and extensive investigation that can be classified as adequate?

(2) Is it that despite all possible investigations surprises will be avoidable?

The word adequate is defined as meaning sufficient, enough or suitable. When applied to geotechnical investigations, it essentially means that they must provide sufficient information to ensure that the most suitable structure is constructed in the optimum location within a time scale and at a cost which were reliably estimated before construction is commenced. The application of such a definition of adequacy does not necessarily require that an unreasonably extensive and hence expensive site investigation is carried out in an attempt to define in infinite detail every aspect of the ground conditions which may or may not affect engineering design or construction. Such an approach is both ridiculous and impracticable. If one insists on adequate investigations to the idealistic horizon, one will be probably be continuing only the investigations and project may never take off. Definitely no one will like that situtation. The practical option is that investigations be carried out to a level which will allow the design and construction of the project to proceed with an acceptable level of confidence. Needless to say that we have to always guard against wrong and careless investigations. In fact, in underground works including rock excavations, the review of design during construction is inevitable.

The pre-construction stage investigations are carried out with a view to evolve a more precise and objective approach giving clearly all the basic data for taking up detailed design. A more thorough geological investigation of foundation consisting of extensive core drilling, drifts, shafts, etc., and excavation for the foundation in critical areas, pilot drifts across the gorges and in abutments which could be later utilised as grouting and drainage tunnels will be done to know the characteristics of foundation rock like presence of joints, their nature and orientation, homogeneity of the mass, continuity, strength, shear zones, faults, folds, tunnelling conditions, etc. Permeability investigations to assess the seepage losses through foundation and abutment are also done. Measures to render them water tight have to be explored by trial grouting tests. Trial embankment may also be justified in many cases. The existence of fissures, seams, cracks and their directions need to be studied. Model experiments are to be conducted to assess the flow patterns and scour patterns during construction and post-construction stages. Siltation problems of reservoirs are to be studied and soil conservation and afforestation measures in the catchments are to be suggested. The designs and drawings of various components of projects are to be made at this stage, based on the above detailed pre-construction investigations. These pre-construction investigations will be directed towards establishment of reasonable estimate of qualities within 5 percent to suit detailed construction planning.

Review of design during construction is required to accommodate additional data/information that becomes available during the course of above investigations. Of the many aspects indicated above which may require a review, the review required because of additional geotechnical features revealed during construction may be more frequent, whatsoever effort one may put in for investigations before taking up of construction.

Projects constructed all over the world prove the fact that inspite of systematic and detailed investigations, in many cases, some surprises are frequently encountered in the case of open cuts, land slides, underground works and foundations.

Open Cut Slopes and Land Slides

In many instances particularly in areas of North and North-Eastern India, the slopes created by deep erosion of rivers are in a condition of limiting equilibrium. In designing cuts and disturbing the equilibrium by another structure, careful studies are called for. In the case of rock slopes, it is easier to predict the behaviour depending on the joint patterns and the controlling failure mechanisms. Experience has shown that cuts should be designed with reasonable assumption regarding its characteristics and excavation started from the top downwards with protective measures taken. It is a common tendency to cut at the toe of the slope and allow it to fall until it reaches the natural slope. This procedure can lead to recessive slides affecting large volumes of material. This procedure is disastrous to the environment. An example is the slope around the open channel, penstocks and power house at Loktak Project.

This project in Manipur envisages the inter-basin transfer of water from Loktak lake to Leimatak valley to utilise a fall of 312m and generate 105MW of power. The water conductor system includes about 2.31 km open channel, 1.1 km cut and cover section, 6.6 km long head race tunnel, a 60 m high, 9.15 m diametre surge shaft, about 0.3 km steel lined tunnel and 1.3 km length of penstock for each of the three units as shown in Fig. 3.

The geological investigations revealed that the channel alignment traverse through lake deposits of recent origin comprising layers of clay, silt, fine sand and pebbles of various thickness. The depth of open excavation varied from 3 m at the lake intake to 45 m at the tunnel intake. Keeping this in view, it was decided to take up the construction of the open channel with 13 m bed width and 1.5 to 1 side slopes. The proposed design could work only for shallow depth and it proved unsuccessful and uneconomical for deep excavation in unstable soil conditions. Due to the wide variations of strata ranging from organic clay to gravel, it was difficult to obtain a representative value of the material characteristics. A number of subsidence and upheavel of the open channel took place even before the excavation reached its final bed level. At this stage, design was reviewed taking stock of the fresh geotechnical information, test results and also the experiences gained during tackling some reaches of excavation. The open channel was re-designed to provide for 18 m bed width with 3: 1 side slopes. In reaches where the slips had occured, the shear strength was further affected due to the sensitivity of the soil, the combination of cunnett concrete section and flat slopes upto 5 to 1 with 2 m deep gravel trenches down the slopes had to be resorted to. It is evident that in many cases it is advantageous to start excavation with a flatter slope and depending on the behaviour the same could be modified after monitoring the performance during construction.

The open channel section described above was successful for length of about 2 km when depth of excavation was of the order of 12 m. Further to this reach, depth of excavation was much more and based on the experience gained during excavating the channel in earlier reaches where a number of subsidence and upheavals had taken place inspite of adopting

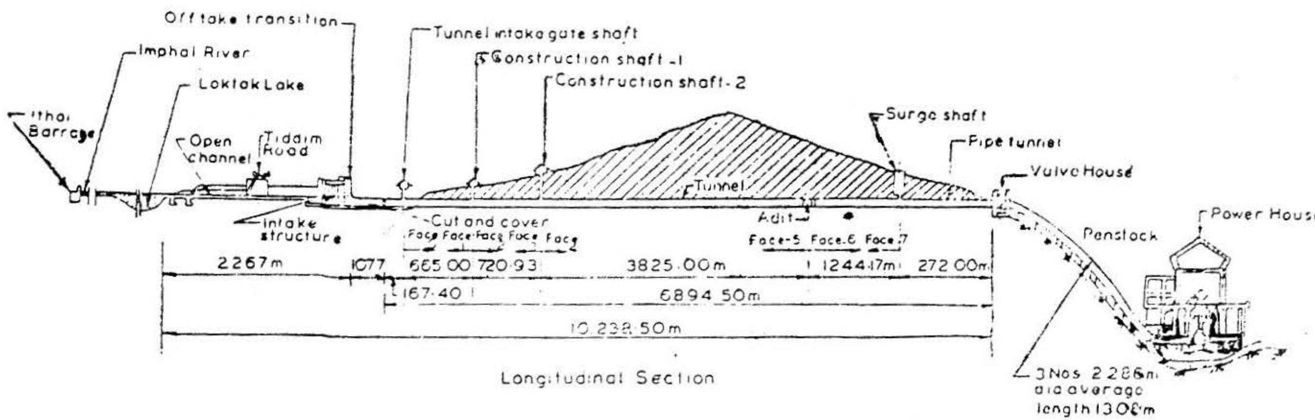

FIGURE 3 Longitudinal Section of General Scheme of Loktak H.E. Project

flatter side slopes, it was decided to discontinue the open channel excavation and adopt cut and cover section.

It was decided that when the depth of excavation was more than 10 m, further cutting will be carried out by using sheet piles. The excavation scheme for cut and cover section included provision of two rows of sheet piles, 9 m apart, with struts at three levels, as shown in Fig. 4.

In the course of excavation buckling of struts was observed and a number of additional struts and diagonal bracings were introduced. In the final design, number of strut levels were increased to four and the excavation was carried out in sequential fashion as shown in the Figs. 5 and 6. The concreting was planned in a sequence, like the bottom strut was removed when the raft was made to act as bottom strut. Removal of other struts was done after completing the concreting in successive layers for the side wall together with back filling.

The penstock slopes at the Loktak Project pose complex problems of slope stability and stabilisation measures include many techniques each suited to the particular location.

The general geology as revealed during the investigations of pre-construction stage indicated that the slopes are on the western limp of broad syncline with its axis in the north-south direction. There are several minor folds within the major syncline structures. The rocks are shales, silt stones and sandstones. The ridges and valleys generally follow the synclinal and anticlinal fold structures respectively. The bed rock is covered generally by slope wash or insitu weathered rock material. The material could also be result of product of weathering of rock which had slid down in early geological times. Geophysical profiles taken along the slopes indicated three types of materials in the vertical profiles, the top 4 to 6 m of slope wash followed by residual soil of weathered rock material and bed rock.

There are 12 anchor blocks as shown in Fig. 7, all founded on rock except 10 and 11 where they were founded on stiff weathered strata. Keeping in view the geotechnical set up of the area, suitable supporting structure for the anchor blocks and saddles were designed and drawing issued for taking up construction.

The reaches between anchor 11 and the terminal anchor-12 which is on rock, posed several problems. Though rock is available at the power-house and at anchor 12, the rock levels are practically horizontal after a height of 12 m above power-house level. The overburden consists of brownish clayey slope wash materials for 5 m to 6 m deep underlaid by weathered rock material. The clayey material is saturated and when excavated for cuts required for the penstocks and power-house, triggered large slides on the right side of the alignment. The southern boundary of

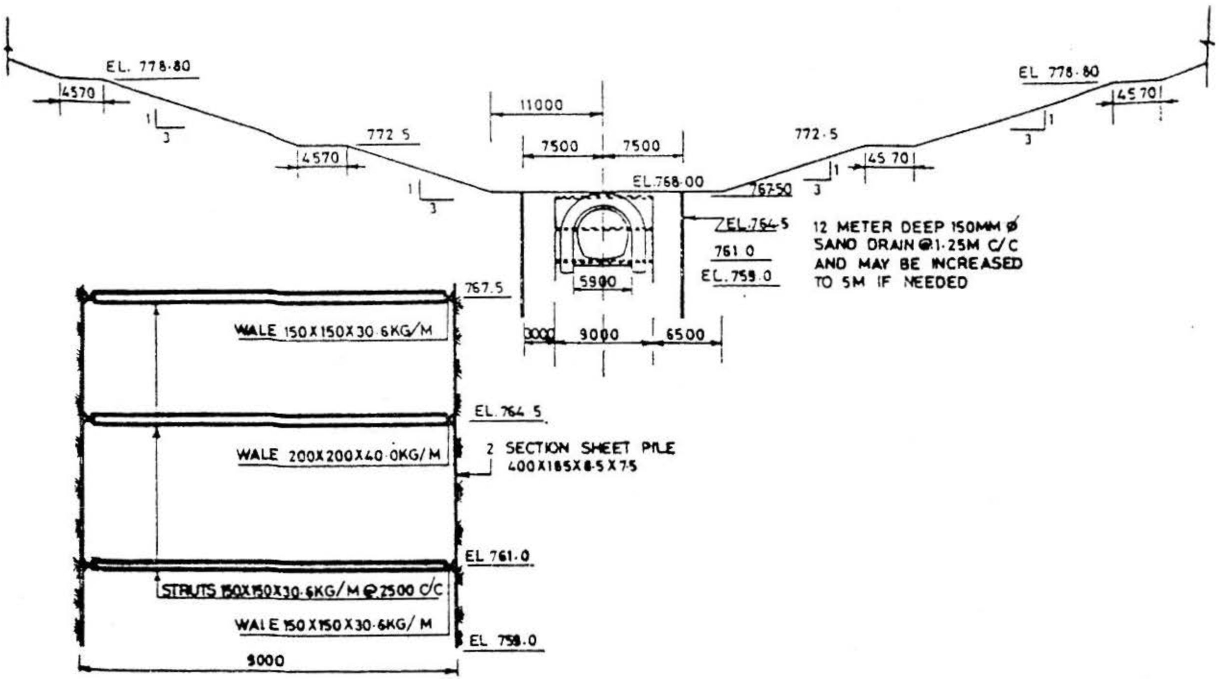

FIGURE 4 Typical Cross Section of Cut and Cover Section with Sheet Piles (Loktak H.E. Project)

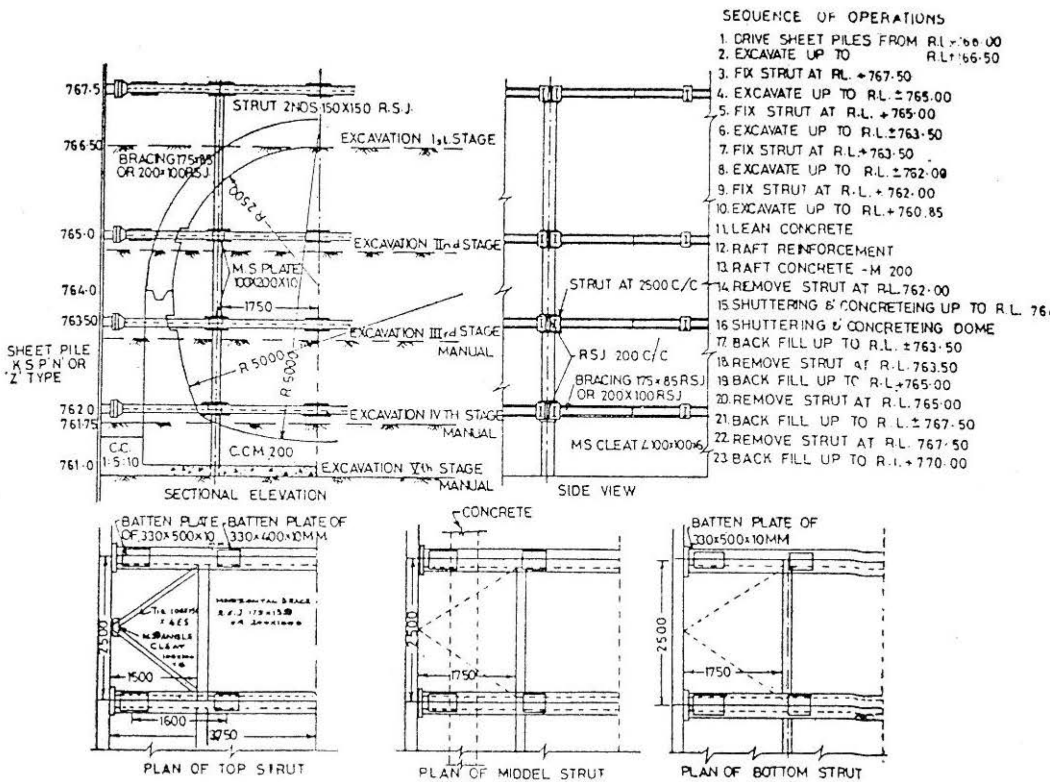

FIGURE 5 Construction of Cut and Cover Section, Sequence of Operations Loktak H.E. Project.

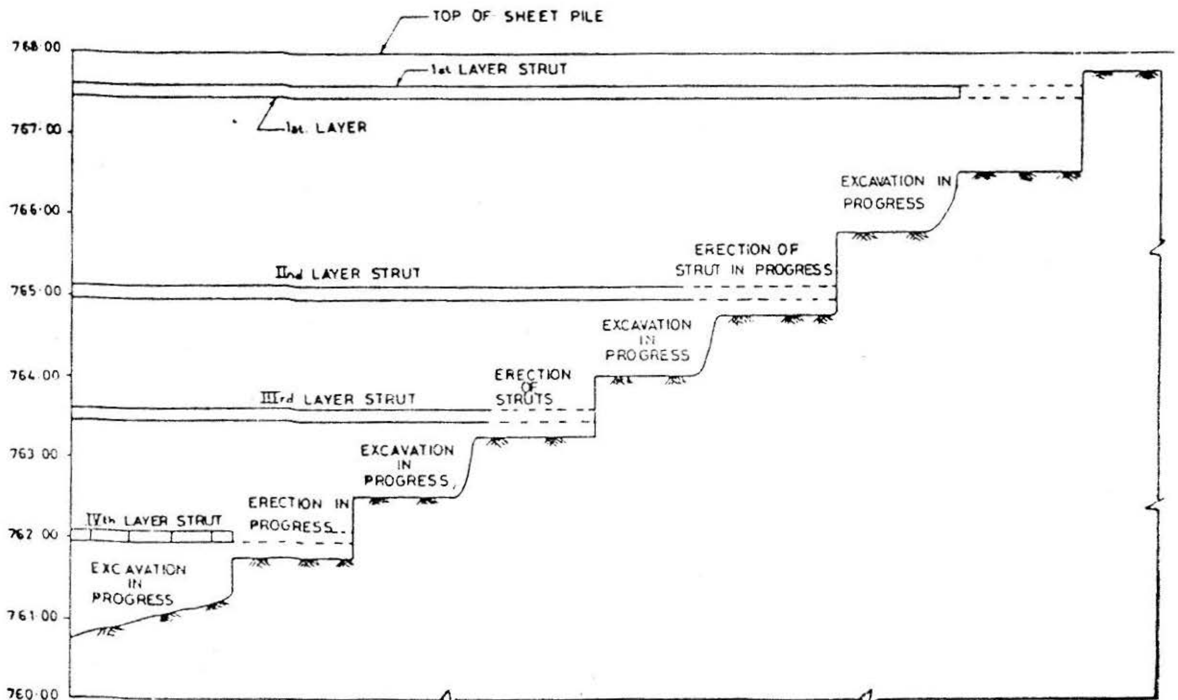

FIGURE 6 Sequence of Excavation and Strutting in Longitudinal Direction Loktak H.E. Project.

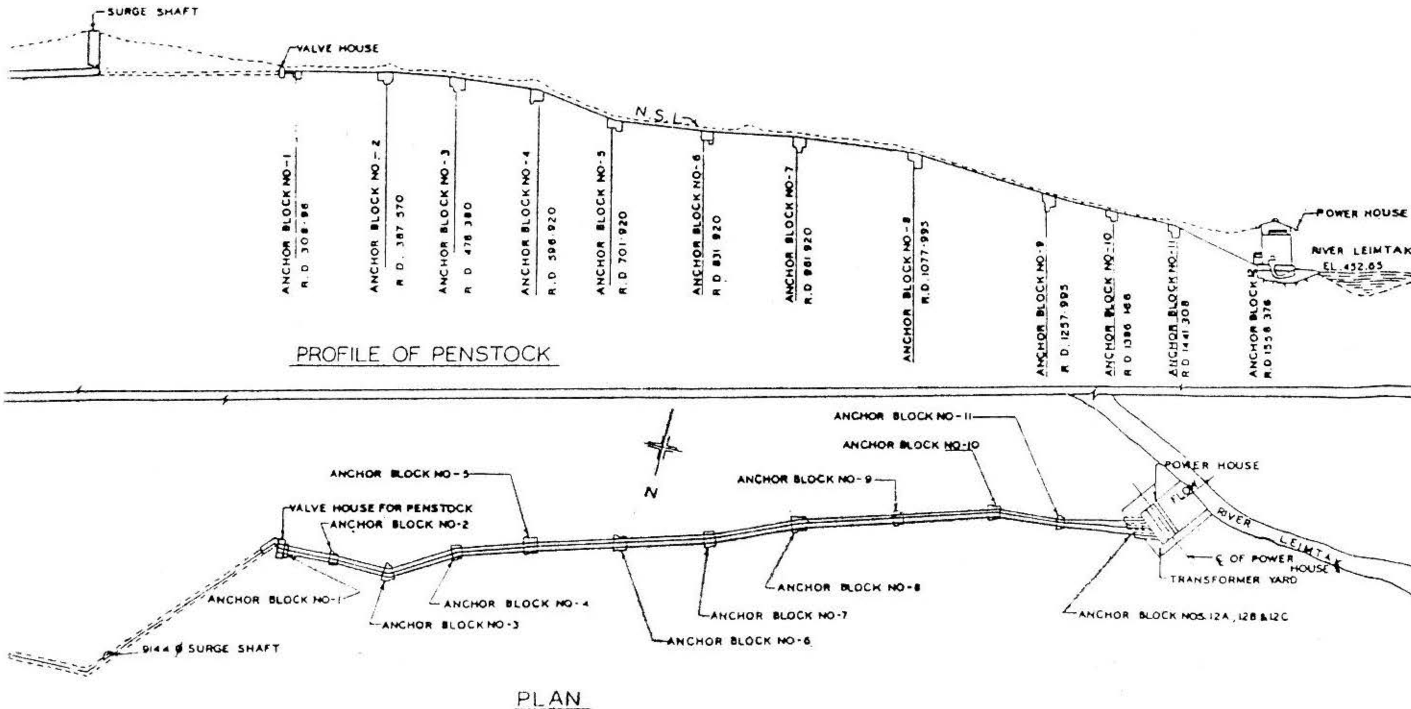

FIGURE 7 Plan and Profile of Penstock (Loktak H.E. Project)

the slide also affected the alignment of penstock. Excavation by means of shovels at the toe by taking cuts 5 to 6 m deep at a time could have aggravated such problems.

The treatment consisted of taking three lines of saddle supports well into the weathered material with the total depth of 12 to 15 m. In the overburden, the slide surface was assumed to be at the contact of the brownish clay and weathered material. Cracks had occurred in concrete lining at this level during construction of shafts. Shear strengths of weathered rock materials obtained from tests indicated it to be stable. The top of the shafts were tied by reinforced concrete ribs and fixed to the anchor 12. In spite of this, some movements of the shafts took place to mobilise the passive resistance. The rib 1 under the right side penstock was affected most by the boundary of the slide. It developed large crack, specially near saddle 2. Cracks were tensile in the rib between 1 and 2 and compressive shear between 2 and 3 indicated that the shaft 2 was subjected to the maximum pressure. Figure 8 shows longitudinal sections of the penstocks showing shafts and ribs.

In order to control the slide, following measures were taken. A number of horizontal drainge holes (30 m or 40 m deep) were installed. Three shafts were driven at an angle of 45° to penstock alignments near anchor 11 to relieve penstock structures from the pressure of sliding material.

The movements were controlled but did not cease. Further, three rows of protective structures are now installed as shown in Fig. 9. These consist of two rows of bored piles driven into rock and tied in triangular grid fashion at the top. In order to further drain the slopes, pums were installed in the shafts to maintain low piezometric levels. Bore-hole inclinometers have been installed to monitor the movements. The right side slopes of the penstocks have been protected by gabion retaining walls and the surface has been protected against erosion by stone boulder pitching.

In the reaches above anchors 11 and 12 particularly near anchor 5 and 6 landslides occurred. Over burden material was subjected to large flow slides starting from bottom of valley and extending towards hill top by a series of retrogressive slides involving movement of several meters, affecting penstock alignment. Based on the sequence of slides it was concluded that the failures take place in the contact zone between soil and weathered strata. The designs were reviewed accounting for the experiences gained in the observational approach adopted in this area. The problem was tackled by working in two fronts; (*i*) by controlling the land slides as far as possible and (*ii*) by strengthening the supporting structure to withstand slide forces in the event of its occurrence. Prevention of land slides were attempted by introducing measures like easing of slopes on the hill side, providing surface and deep hill drainages, carefully planning the disposal

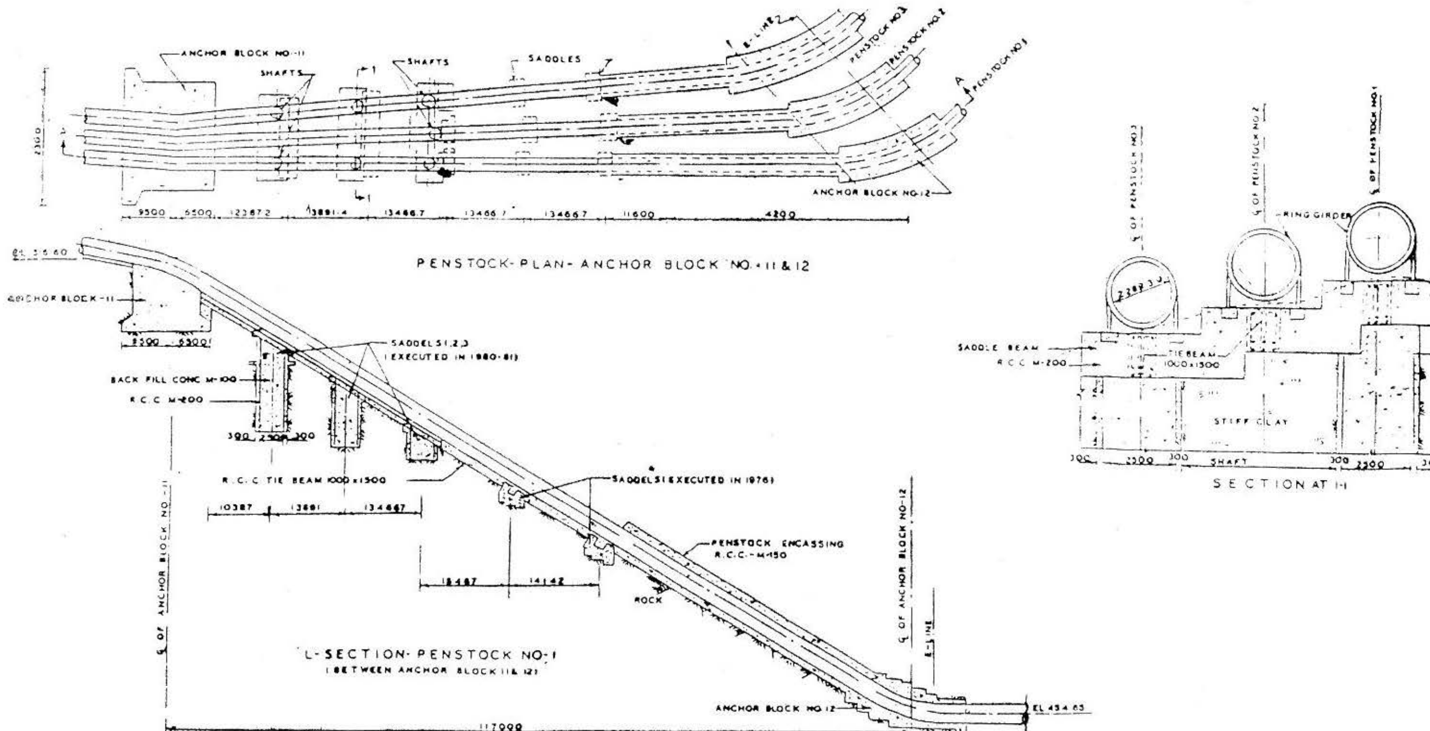

FIGURE 8 Anchor Blocks 11 and 12, Plan and L-section Showing Shafts and Sibs (Loktak H.E. Project)

PLAN OF ANCHOR BLOCKS 11 & 12
POWER HOUSE COMPLEX

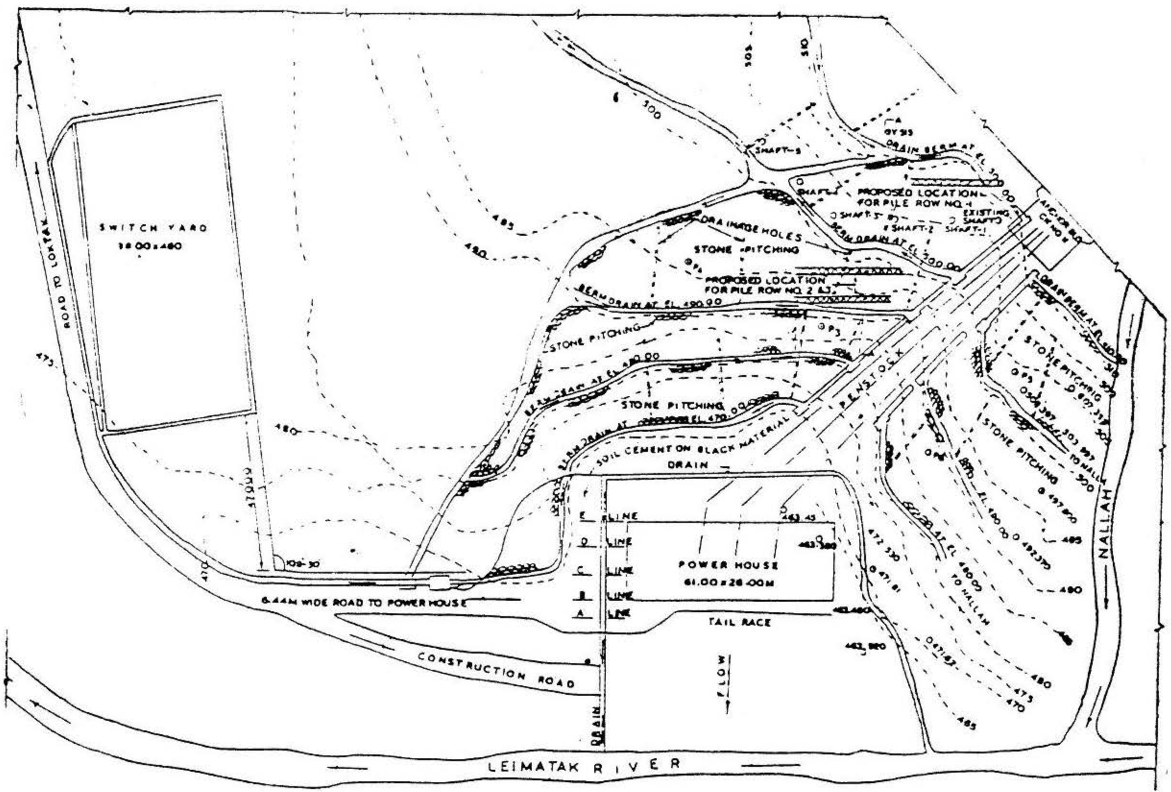

FIGURE 9 (*a*) **Problems During Construction of Anchor Blocks 11 and 12, Plan (Loktak H.E. Project)**

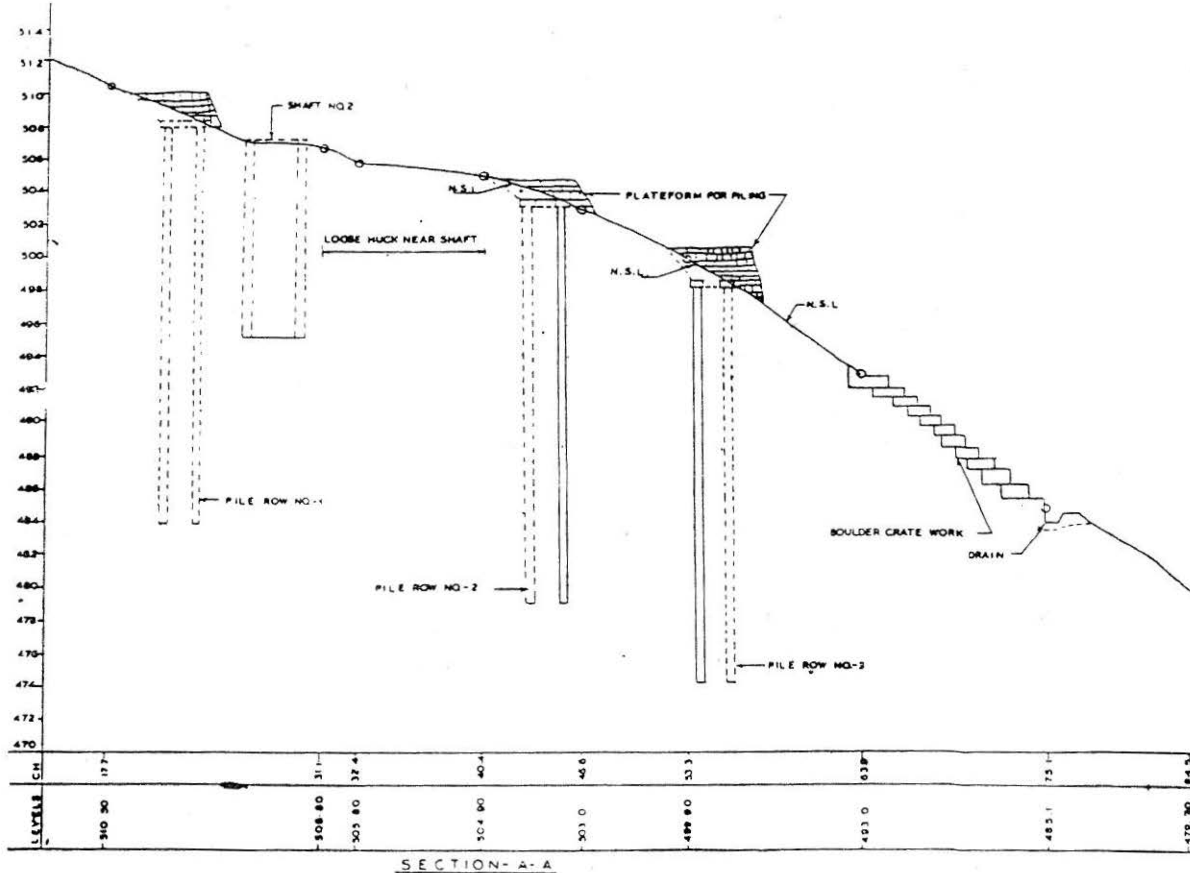

FIGURE 9 (*b*) Problems During Construction of Anchor Blocks 11 and 12, L-section (Loktak H.E. Project) of excavated material, training the nallah bed at the toe and sides with gabions to halt erosion. In respect of strengthening the supporting structure, the measures included, strengthening by boxing with RSJs driven all around the structure, constructing shafts by mining method adjacent to the structure on the valley side going deep into rock and back filling with concrete and interconnecting these shafts at the top to form as a grid as shown in the Fig. 10. This project provides an example of frequent reviews of design not only during its construction but even after its construction. The slopes are being monitored by inclinometers and surface geodetic surveys.

Tunnels and Underground Works

A decade ago the design of temporary support system for tunnels was based on rock loads worked out using a system of classification of rocks, based on the work done by Terzaghi. Based on experience it was seen that this classification system does not take some important rock parameters into account. Barton and Beiniawski introduced parameters like RQD, joint characteristics, insitu stresses in rock by giving a suitable ratings to each factor. These are being used for estimates of support system but in actual practice tunnel support system is based on systematic geotechnical evaluation at site and monitoring during construction. Convergence measurements assist in evaluating the adequacy or the provision and the present day systems essentially use shotcrete and systematic rock anchoring and make adjustments in the support strength to suit the rock conditions. It was also seen that the steel supports when used based on lighter sections but placed quickly could in many cases be better than heavier sections placed slowly.

The evolution of support designs of the tunnel at Loktak Hydro Electric Project is an example in this respect. The 6.6 km head race tunnel is of modified horse shoe section with finished diameter of 3.81 m and an excavated diameter of approximately 4.7 m. The tunnel geology as revealed by drilling and mapping of the out crops shows that tunnelling includes lake sediments charged with water and flowing ground conditions for a length of about 830 m followed by terrace deposits in a length of 1.235 km. Splintery shales with subordinate sand stone and silt stones bands of Desangs and Barail series account for the strata in the remaining reach as shown in Fig. 11. Based on the predicted geology, designs and drawings were firmed up by providing heavy steel sets and the construction was accordingly taken up by conventional heading and benching method. During excavation it was observed that the shales, silt stones and sandstones encountered are so intensely folded that the rock conditions change frequently. Water pockets were frequently concountered thereby worsening the tunnelling conditions. The shales were found to exert considerable pressure and cause heavy convergence as shown in Fig. 12.

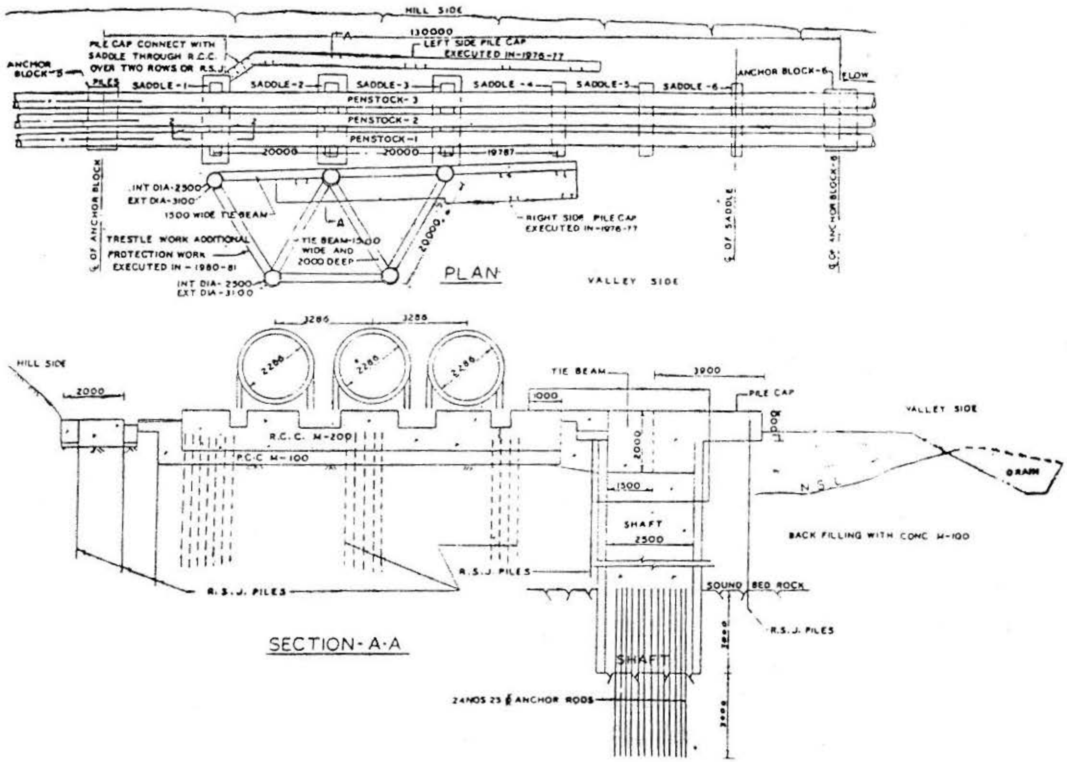

FIGURE 10 Problems During Construction of Anchor Blocks 5 and 6, Plan and Section (Loktak H.E. Project)

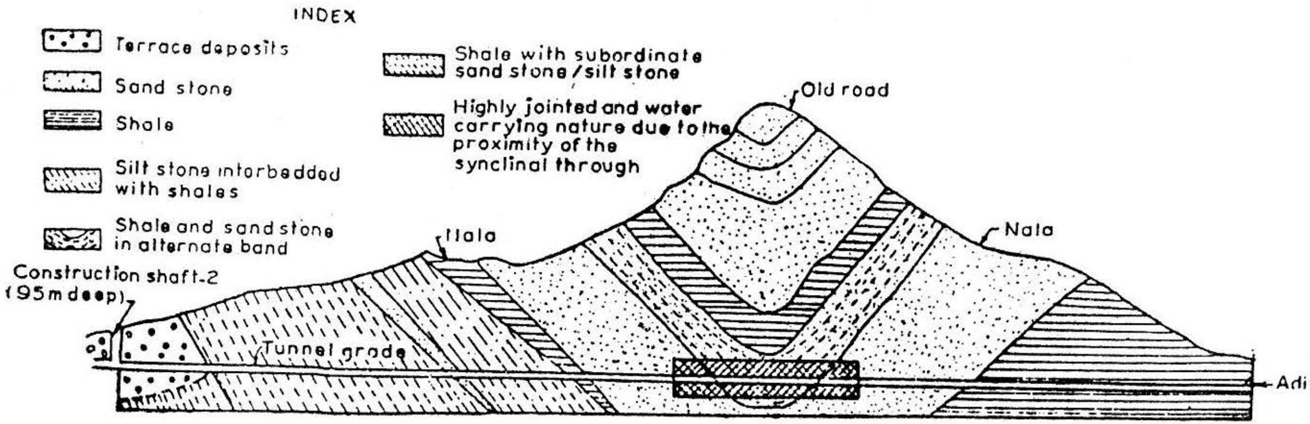

FIGURE 11 **Typical Geological Section Along Tunnel Alignement (Loktak H.E. Project)**

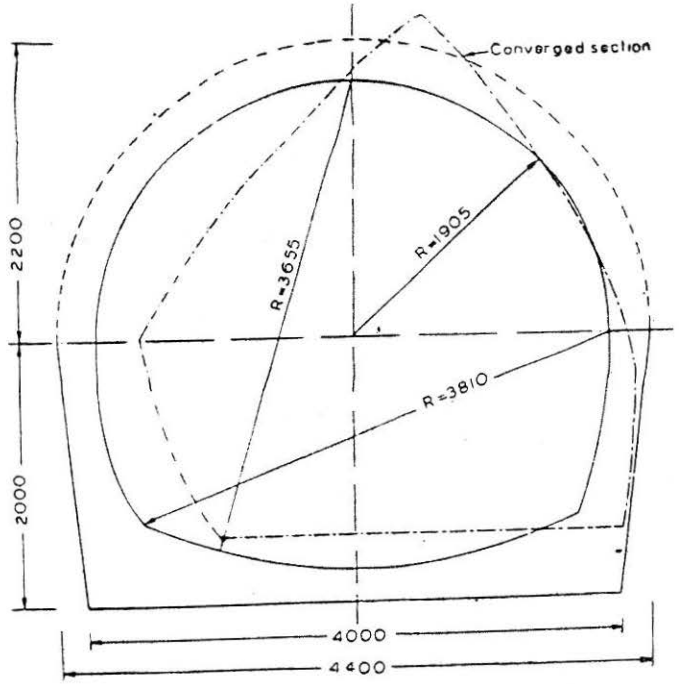

FIGURE 12 Convergence of Tunnel (Loktak H.E. Project)

Keeping in view that completion of the tunnel was the main component which was holding commissioning of the Project, all efforts were oriented towards success in this direction. In addition to providing a design more suited to the geotechnical conditions revealed during construction, the construction was to proceed using flame proof equipment to avoid the menace of methane gas. An explosion due to the gas cost 17 lives and tunnel work had to be suspended for sometime till a review of tunnelling equipment and methods was undertaken. Keeping in view the geology of the strata including the grim reality of tunnelling conditions as revealed during construction, it was decided to review the design by introducing New Austrian Tunnelling Method (NATM). To achieve targets, Messers Leonard Mill, West Germany and Meassers Geoconsult, Austria were engaged. To start with, based upon their experience of tunnelling, full face excavation together with shotcrete, wire mesh and systematic rock bolting was adopted as shown in Fig. 13. This procedure created its own problems. The tunnel being in squeezing shales, heavy convergence occurred in the excavated reaches. Invert heaved up to the extent of 1.5 m and the support system distorted and twisted leading to frequent rock falls. All this called for a further review of design. The full face excavation was stopped and heading and benching method was adopted as shown in Fig. 14. Besides making some changes in the steel supports and rock bolts, lateral dimensions of the tunnel were

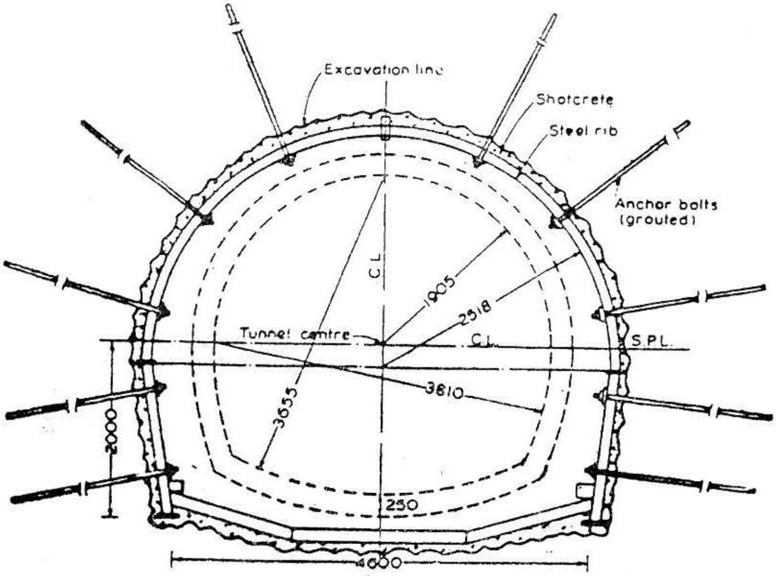

FIGURE 13 **Typical Cross-section of Tunnel, Full Face Sequence (Loktak H.E. Project)**

increased by 70 cm for accommodating convergence. To arrest these convergence, design was to be reviewed providing for very closely spaced grouted rock bolts. Ultimately the design and construction adopted proved very useful and progress of nearly 50 m per month could be achieved.

The use of shotcrete and system anchoring requires introduction of this new technology in Indian conditions. The increased tempo depends upon the closed interaction in the field by a team of geotechnical engineers, geologists and the contractor. Monitoring convergencies is also an essential feature of this method. The tunnelling industry in India is yet to gear up to adopt this technology. The progress of hydro electric development will depend upon a much faster rate of tunnelling than hitherto achieved in India.

Dam Foundations

Foundations of gravity dams on exposure reveal some fault or shear zones. Vertical or near vertical features are normally treated by excavation for a certain depth and backfilling with concrete. It was the practice to use the Shasta formulae but though this is still adopted in general for minor features, more detailed finite element analysis is done at the time of design for important features so that the excavation and backfilling could be reduced to the extent feasible. Insitu tests for modulus of deformation is done and stress analysis carried out.

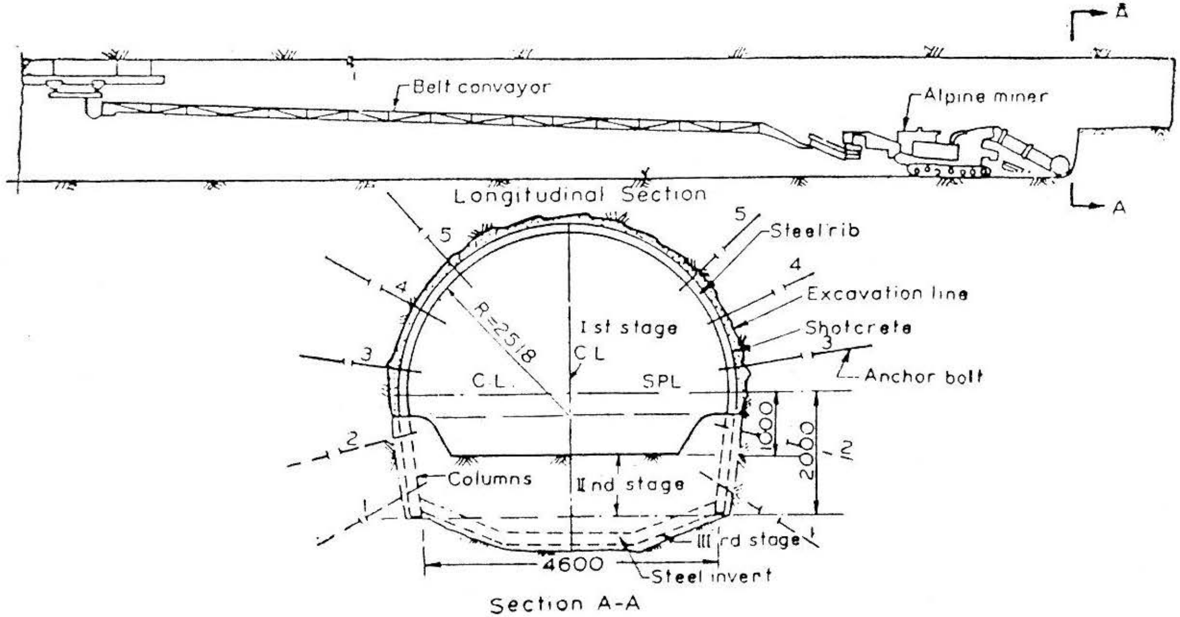

FIGURE 14 Heading and Benching Sequence (Loktak H.E. Project)

More important is stability of gravity dam foundation having near horizontal features. Several dam foundations have been successfully tackled during the last few years. Detailed finite element analysis studies have led to some important findings:-

1. Mobilisation of cross shear by excavating deeper and providing key trenches is effective to obtain the required safety.
2. Keys provided near the downstream zone of the dam are more effective due to the large compressive stresses and consequent increase in the lateral resistance.
3. Cross shear can also be utilised in block near the abutment.
4. Some question was raised whether mobilisation of the resistance of the concrete keys as well as the strength of the seam could be assumed due to the difference in stiffness. Detailed analysis in case of Karjan Dam have shown that it is justified in many cases due to the high stress near the keys causing movements of an order of a few millimeters sufficient to mobilise the strength of the seam.

The problem faced at Srisailam dam would illustrate these features.

Srisailam Hydro Electric Project is on river Krishna in Andhra Pradesh. It is a 770 MW Station, consisting of 143.9 m high massonry/concrete dam, a 15.2 m diameter and 731.5 m long tunnel, a 30.5 m diameter surge shaft, 7 nos. of 6.1 m diameter pressure shafts, a power house with 7 generating units of 110 MW each, a tailrace tunnel and a tail race channel as shown in Fig. 15.

The dam is located in a gorge, where there is a standing pool of water of 30.5 m depth, even in summer. Preliminary and pre-construction geological investigations of the dam foundations were done. Exploratory core drilling was done in foundations, below water, with the help of punts, over which drilling machines were mounted. These geological investigations had not revealed the presence of any major structurally unsound features, such as faults, folds, shear zones, etc. After the coffer dams were constructed and dewatering of foundation area was done, further geological investigations were carried out. These investigations and further borehole data, brought to light certain inherent weaknesses in the foundations. Horizontally disposed shear zones of varying thickness from 0.2 to 0.9 m were found to be existing at depths ranging from 13 to 20 m, containing highly sheared rock and gougy shales as shown in Fig. 16. Detailed probing indicated lateral and longitudinal continuity of these shear zones. The materials in these shear zones possessed low shear strength.

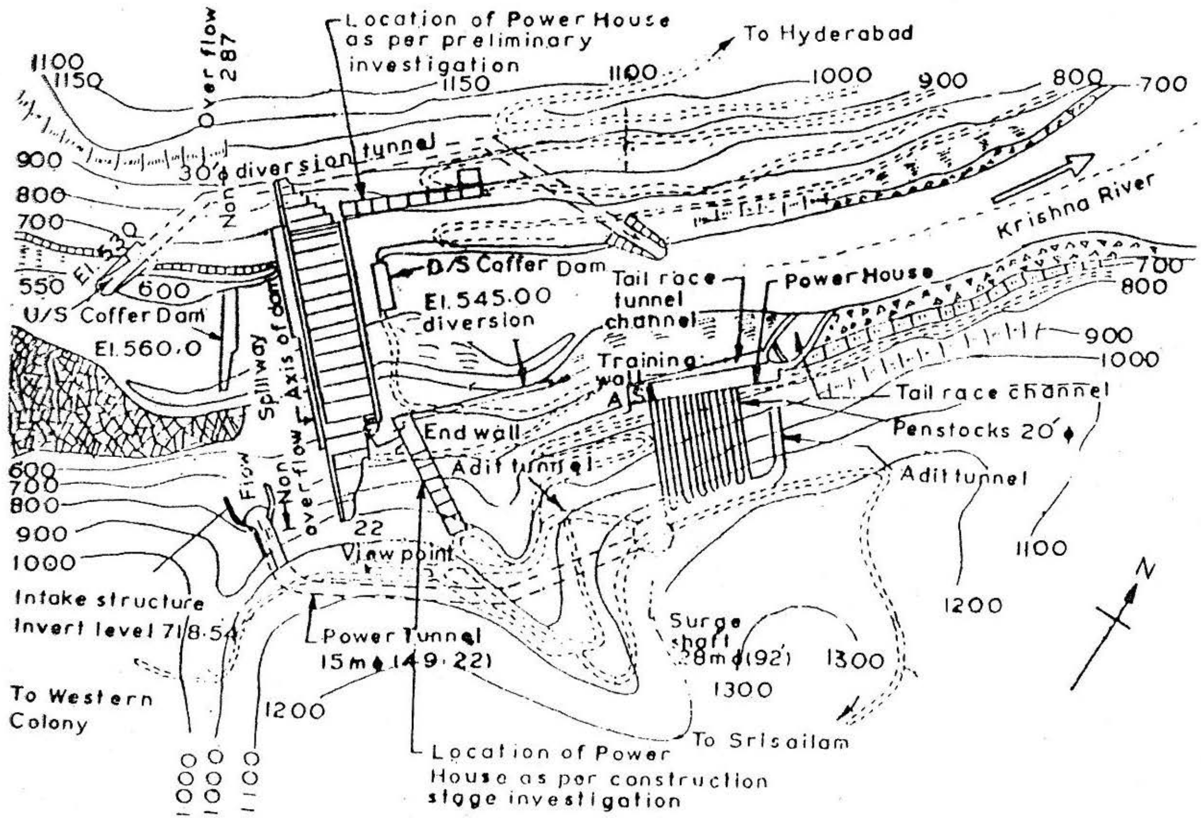

FIGURE 15 General Layout of Srisailam H.E. Project

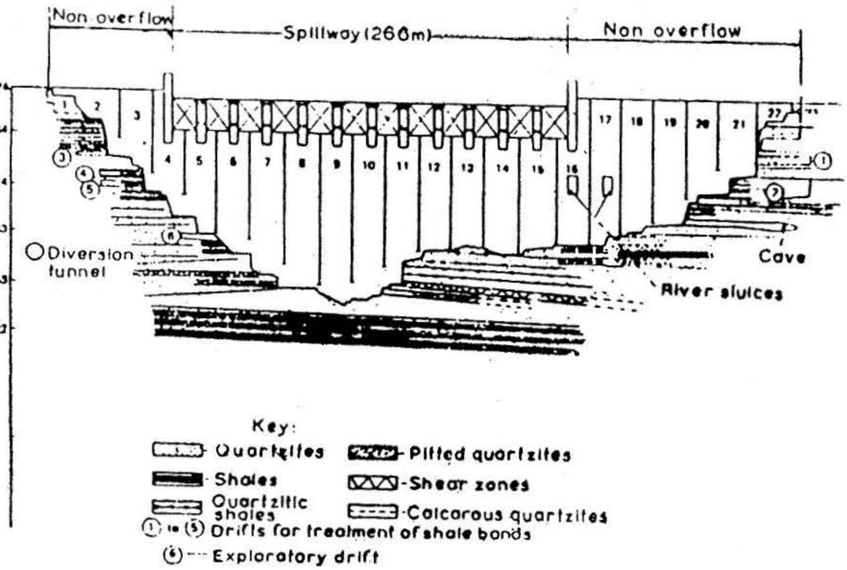

FIGURE 16 Upstream Elevation of Dam Showing Sectional Geology of Srisailam H.E. Project.

During the excavation of foundations, weak zones and pockets of pitted and weathered quartzites were noticed. Thick shale bands of 0.9 to 1 m thickness, in between the massive quartzite zones were seen on both the abutments. Due to the presence of these shale bands with gougy materials, the shear friction factor of the dam foundation will be considerably reduced. To make up for this, additional safety measures like increasing the section of the dam, providing shear keys in foundation, providing heavy RCC toe wall, excavating horizontal drifts cutting across the seams and backfilling with concrete, construction of the dam with a slight curve towards upstream had to be adopted.

During the foundation excavation, a natural cavern was noticed and on excavation it was found that the size of the cavern was $75 \times 4 \times 4$m. The cavern had to be treated and back filled with concrete, providing a gallery for drainage and inspection as shown in Fig. 17.

Need for Specialisation in Cadres

Geotechnical engineering has a very special role in water resources development. At every stage in a water resource project the involvement of geotechnical engineers is crucial. However, it has not been possible to develop cadres of geotechnical engineers in the government departments and organisations which are in charge of the projects. The very set up of cadres is a deterrent to specialisation because cadres by nature entail interchangeability. No complete solution has been found for this problem. One solution could be to have at least 15 to 20 percent of all cadres at

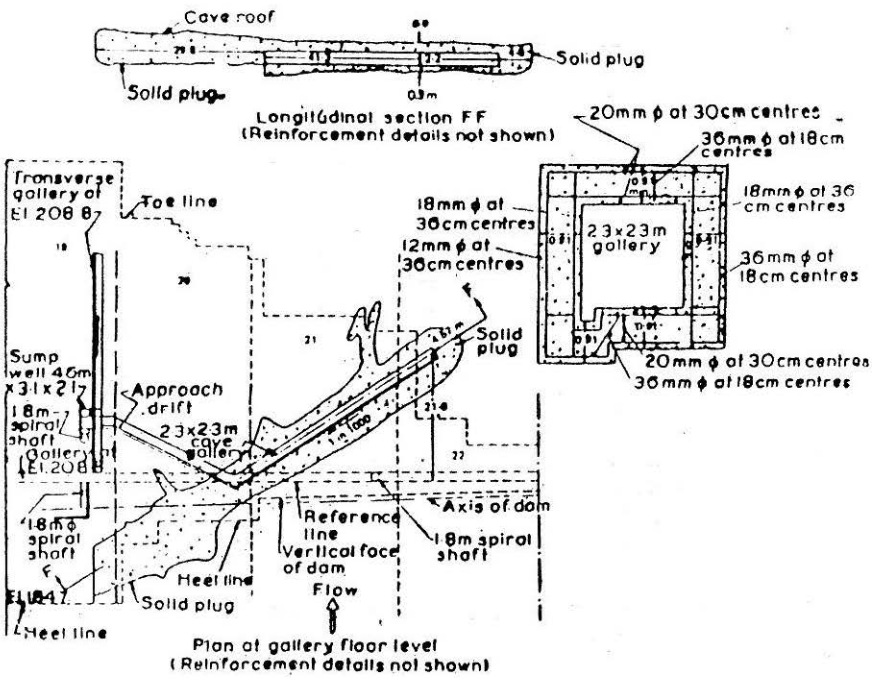

FIGURE 17 Cave Gallery in Dam (Srisailam H.E. Project)

specialists' posts in which specialisation can be developed. In fast advancing technological situation, keeping abreast of developments, putting the state-of-art to practice without lag requires confidence, which can be developed only by having a 'feel' of the discipline brought out after years of practice. It is also necessary to project out experiences to outside world. India has nearly 1600 dams, built during the last three decades, but the impact of this work of geotechnical engineering practices in the field is not commensurate with the achievements. This is also due to fast rate of movement of engineers within the cadres, preventing continuous monitoring of behaviour of structures during construction and performance afterwards which are vital for technological developments.

Organisations of Geotechnical Engineering Work in Water Resource Development Projects

It is clear that geotechnical engineering in Water development projects involve many special problems for which some organisational mechanisms have to be developed.

To presume that foreign consultancy or experts, contractors will relieve us from confrontation with these problems is misplaced faith. There can be no substitute to systematic development of experience, judgement and technology. The following measures are suggested :

1. A well knit and well coordinated team of general civil design

engineers, engineering geologists and geotechnical engineers and project engineers is very essential and it should work in close coordination right from the project inception to completion.

2. Development of probing technique in tunnels which could be utilised without interfering with actual excavation and utilisation of this as a routine in tunnelling works in the Himalayas.
3. Suitable provisions in the contract to cater to the surprises should be made. Independent committees which will examine any special problems during construction with regard to changed conditions will be useful.
4. A Pear Review Committee having very good experience of design, construction, geology, investigation should visit the project at periodical intervels. Regular meetings both at investigation site and in the design office, for review of the geotechnical data collected will lead to a more purposeful and realistic formulation of project initially and avoid time and cost overruns during constructions.
5. Certain incentive schemes to attract interested and dedicated workers in the investigation particularly in remote areas are very essential.
6. Accessibility of project site should be improved by free utilisation of available modern transportation means.
7. Efforts to involve independent consulting organisations in the detailed designs and of structures in minor and medium river valley projects should be made. In some important projects basic designs could still be done by governmental organisations. Designs of major national projects could be retained with government organisations.
8. Consulting organisations may be entrusted with post construction monitoring work.
9. Involvement of academicians in designs and post construction monitoring work by government and consulting engineers will lead to improved development of technology.
10. Constant value analysis of the design practices to achieve economy based on the experience of the project already constructed and post construction monitoring work.
11. There is practically no organisation to develop construction technology. In India technologies like jet grouting, diaphragm works are totally dependent on foreign knowhow. Setting up of plant design units in large project could contribute to this aspect.

References

GARY, R.E. GARDNER, G.D. and WIMBERLY, P.M. (1980); "Stabilization of Colluvial Slope at an Urban Site". *Proceedings of International Symposium on Landslides*, New Delhi, Vol. 1, pp 389-392.

KUNJAYAPPAN, M.V. (1983); "Geotechnical Problems and Construction Features of Loktak Hydro-electric Project". *Journal of the Institution of Engineers (India), Part CI 2*, pp. 74-80.

MADHAVAN, K. (1980); "Need for Improved Blasting Technology". *Proceedings of Seminar on Accelerated Hydroelectric Development in India*, National Hydroelectric Power Corporation Limited, New Delhi, Vol. 1, pp. 27-28.

MADHAVAN, K. (1982); "Tunneling in Rock with High Insitu Stresses Engineering Geological Problems of Tunnelling and Excavation of Cavities". *Proceeding of 4th International Congress*, International Association of Engineering Geology, New Delhi India, pp. V 79-84.

MADHAVAN, K. (1987); "Value Engineering", Major General Harbirat Singh Memorial Lecture, Institution of Engineers, Delhi.

MURTI, N.G.K. (1981); "Investigations of Hydel Projects". *Proceedings of Seminar on Accelerated Hydroelectric Development in India*, National Hydroelectric Power Corporation Limited, New Delhi, Vol. I, pp. 3-8.

MURTHY, Y.K. (1981); "The Importance of Field Investigation for Planning of Hydel Projects", *Proceedings of Seminar on Accelerated Hydroelectric Development in India*, National Hydroelectric Power Corporation Limited, New Delhi, Vol. 1, 45-50.

MURTHY, Y.K. MADHAVAN, K. and DIVABA, E. (1973); "Evolution of Design during Construction of River Valley Project", J. Inst. Engrs (India), Vol. 53. C I-III.

RAO, D.N. (1986); "Improper Planning and Application of Construction Industry Some Practical Suggestion for Improvements for Better Management". *Symposium on Modern Management Techniques in Irrigation Project, 54th Board Session*, CBIP, India, Paper No. 6, pp. 45-52.

SWAMY, M.L. (1981); "Inadequacies in Investigation of Hydroelectric Projects in Andhra Pradesh". *Proceedings of Seminar on Accelerated Hydroelectric Development in India*, National Hydroelectric Power Corporation Limited, New Delhi, Vol. I, 37-44.

TAKAHASHI, H. (1980); "Construction Works and Geology". *Symposium on Problems and Practice of Dam Engineering Bangkok.*

'REPORT (1973); "Investigations of Soft and Organic Clay Deposits from Power Channel of Loktak Hydel Project". Central Soil and Materials Research Station SD-73/3.

REPORT "Rise in Costs of Irrigation and Multipurpose Projects". *Report of the Expert Committee* (1973); Ministry of Irrigation and Power, Government of India, New Delhi, pp. 1-289.