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Analysis of Exhaustive Field Data for the First Underground Powerhouse in the Lesser Himalaya - A Review

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ABSTRACT

Where theory, experience and model studies fail to lead an engineer, observational approach provides him with a trustworthy torch. Crevices of darkness may still remain, if the path is “Rocky”. An engineer or a geologist may find himself on-the-rocks, if he doesn't have a “rocky heart”. The first underground powerhouse complex at Chhibro village of the Indian Himalayan state Uttarakhand was constructed over five decades ago in dolomitic limestone of the lesser Himalaya. Long-term instrumentation was planned during construction to check the safety and adequacy of the support system and to monitor the post-construction behavior of the excavation. The present paper discusses the importance of precious instrumentation data collected over ten- year period (1973-83) and its comprehensive analysis. The observed roof support pressure on the powerhouse cavern has been compared with the support pressures estimated from different theories. Significant time-dependent effect was observed in saturated rock mass where there's seepage problem, i.e. on the roof of the cavern and also near underlying thick shear zone. The analysis of exhaustive data suggested that the ratio between long-term and short-term support pressure may not be a constant equal to 1.7 as generally assumed but would vary from 1.7 to 5.9 depending upon seepage condition, soluble/erodible nature of joint fillings and the types of support system. Passive support systems (such as steel ribs, concrete arch etc.) appear to attract more load during the lifetime of the cavern while the active support systems (such as cable anchors /rock-bolts with shotcrete etc.) are likely to encounter lesser long-term support pressures. The analysis of the exhaustive field data has played a key role in bridging the gap between the purely theoretical and empirical approach. The lessons learnt from this classical case study may help the practicing engineers to carry out proper planning, designing and execution of the underground powerhouse complexes to be constructed in geologically challenged, weathering prone and seismically active regions as in the Himalayas. In order to consider the effect of recurring earthquakes on the long- term support pressures near the thick shear zone, a new empirical theory was proposed. The observations taken after the 1991 Uttarkashi earthquake appear to confirm the proposed hypothesis on the mechanism of dynamic support pressures near thick shear zone.

***Keywords*:** Long-term support pressure; Thick shear zone; Saturation; Instrumentation; Seepage; Time-dependent; Seismic area; Recurring earthquakes

1. INTRODUCTION

In India, several river valley projects with an important component of underground structures have been constructed successfully during the last few decades and some more projects are also under the planning, investigation or construction stage to exploit the large untapped hydro-power potential. The underground excavations include tunnels, surge chambers, powerhouse cavern, pressure shafts etc. The design of the support system for such openings is based on past experience, model studies and theoretical computations. The support parameters so determined, may not be a true representative of actual rock mass conditions because the in-situ properties of rock mass and its behaviors are not exactly known. Moreover, the nature of movement and stresses around underground opening varies from place to place. Consequently, the temporary supports were either under designed or over designed as observed in most of the tunneling works specially in the Himalayas.

The basic aim of any underground excavation design should be to utilize the rock itself as the principal construction material, creating as little disturbance as possible during the excavation and adding as little as possible in the way of concrete and steel support. A good engineering design is a balanced design in which all the factors which interact, even those which cannot be quantified are considered. An increasing demand for more realistic safety factors as well as the recognition of the money saving potential of rock mechanics will lead to greater application of rock mechanics design in tunneling. While extensive research has been conducted in rock mechanics, there still seems to be some problem in translating the research findings into innovative and concise design procedures. The advancement in the instrumentation technology has enabled the significant use of instruments to monitor the rock behavior in large underground excavations. The underground structures in the complex geological formations / poor rock mass generally present major design and construction problems. The numerical analysis of rock mechanics problems associated with the design of large underground excavations can be relied on reasonably when it is based on field measurements.

Experience shows that most new or modified construction techniques are not readily accepted by the construction or engineering professionals until proven satisfactory on the basis of actual performance. Data obtained from instrumentation can aid in evaluating the suitability of new or modified techniques. Thus, performance observations and long-term monitoring on the underground powerhouse cavities may lead to advances in construction technique and improved design. Keeping the above objectives in view, instrumentation data for studying the long-term behavior of the first underground powerhouse complex designed and constructed in the lesser Himalaya were collected and analyzed in detail for over a ten years period (1973-1983).

2. PROJECT UNDER DISCUSSION

Construction of underground excavations for hydroelectric projects in the lower Himalayan region has always been a challenging task due to support problems under complex hydro-geological conditions and tectonic influences. Further, these problems could be attributed to the non-homogeneous and anisotropic nature of rock mass and their time-dependent behavior. In this context, the Chhibro underground powerhouse complex has set a major precedent by being the first venture of its type in the lower Himalayan region. This 240 MW underground power station exploits the drop of about 124 m along the first loop of tons river, a tributary of the Yamuna between Ichari and Chhibro, which is part-I work of Yamuna hydroelectric scheme stage-II and was necessitated because the location of a surface power station would have involved large scale excavation of steep slopes. Finally, the powerhouse complex was sited in a band of limestone which has horizontal width of 193 to 217m. The complex comprises a network of excavation for the machines, transformer, turbine inlet valves and control room and also provides operating galleries and hydraulic connections to the 120 MW Khodri surface power station utilizing the remaining drop of 64 m along the second loop. Figure 1 shows the general layout and the geological cross-section along the long axis of the powerhouse complex.

The cavern has a semi-circular crown (rise/span ratio of 0.27) and the entire roof of the entire cavern is at the same level to facilitate excavation of the cavity and operation of the gantry crane. However, the floors are at different levels as per the requirement of machines. The surge shaft has a 20m diameter underground surge tank of the restricted orifice type and is sited in limestones (lower half) and slates (upper half).

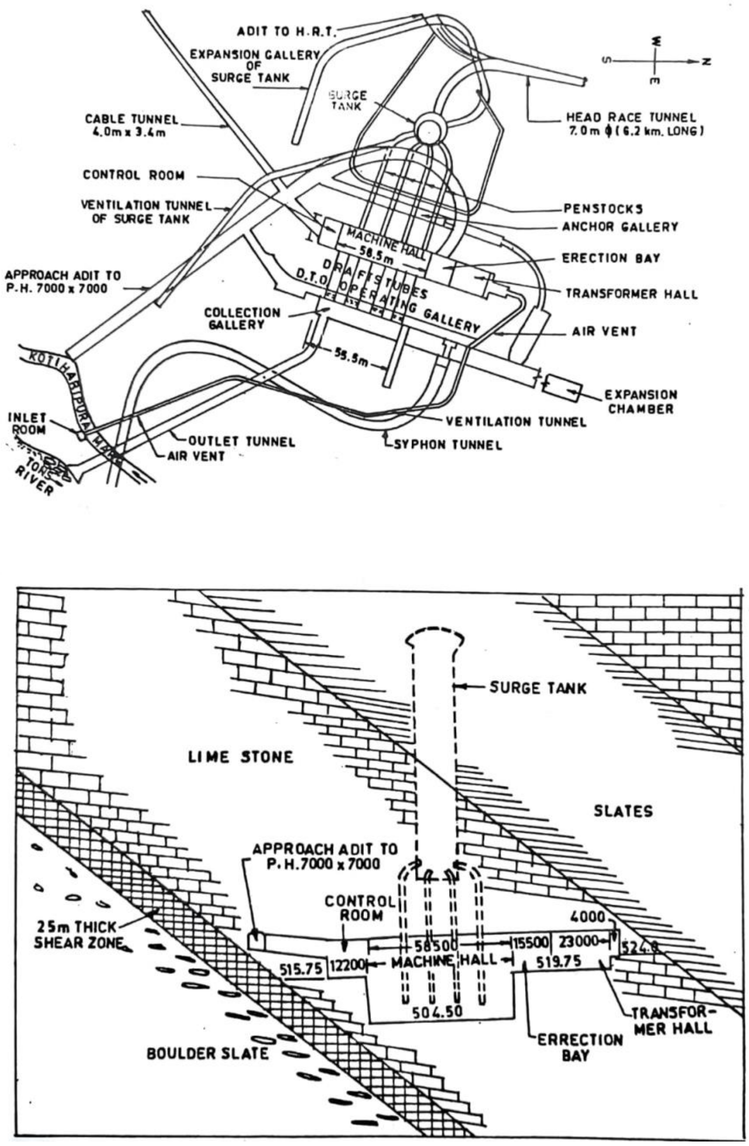

Figure 1 - General layout of powerhouse complex and its geological cross-section along the long axis

2.1 Salient Features of the Yamuna Hydel, Stage-2, Part-1 Project

- A 60 m high concrete diversion dam across the river Tons at Ichari with spillway, Intake sedimentation chamber, flushing arrangement and tunnel control structure.
- A 6.3 km long concrete lined headrace tunnel (HRT) of 7.3 m finished diameter with design discharge capacity of 235 cumecs.
- A 20 m dia. and 100 m high underground surge tank at the end of HRT.
- Four steel lined pressure shafts of 3.8m dia. feeding the four turbines.
- Underground powerhouse complex at Chhibro with four machines of 60 MW each; The powerhouse cavern (Machine Hall) is 18.35 m wide, 32.5 m high and 113.2 m long with a rock cover of 208 m above Control room to 250 m over transformer hall;
- The complex is located in the limestone band of approx. 140 m thickness (Horizontal thickness 210 m across in the horizontal direction with a series of shear zones of 2 cm to 50 cm thick, interlayered with minor slates)
- Tailrace works comprising collection gallery, expansion chamber, outlet tunnel and a syphon tunnel interlinking the tailrace works of part-1 with the headrace tunnel of part-2.

3. GEOLOGICAL FEATURES

3.1 General

The powerhouse complex is located in a band of stratified limestone of 140 m true thickness (210m across the horizontal direction) with numerous shear zones ranging from 20 to 500 mm thick and which are sub-parallel to bedding. A major shear zone of 25m thick approaches to within 10 m of the lowest draft tube level in the powerhouse area. The formations dip at 45° towards N10°W to N12°E. The excavation is aligned normal to the strike of the rock formations with cover ranging from 208m over the control room to more than 250m over the transformer hall, control room, erection bay etc. The machine hall, control room, erection bay, and air conditioning cum ventilation chamber are located in a single continuous cavity with a longitudinal length of 113.2m and is sited in limestone band. The geological cross- section of the powerhouse cavity with prestressed anchors (Fig. 2).

On the basis of the study of structural discontinuities in the rock mass, it was deduced that 11 sets of such structural discontinuities are present. It was inferred from analysis of joint data that a 16 m high triangular rock wedge, bounded by prominent shear and joint planes was likely to separate out from the crown unless this is adequately supported. This was estimated to be equivalent to a rock load of 8 m rectangular slab of rock mass. The design load as adopted finally was equivalent to 14 m of rock in view of other unknown factors such as superimposed load, self-load of steel ribs and erection load etc. Final design support pressure was estimated to be 0.38 MPa (Table 3).

The observations during geotechnical investigations revealed that the use of rock- bolts may not be suitable for supporting the cavern roof. Geological studies indicated the presence of minor shear zones having crushed and water laden material together with complex jointing patterns in the limestone band where the powerhouse was to be located. Consequently, a special sequence of excavation and conventional mode of supporting the cavern was considered safe.

The study further revealed the existence of five prominent geological discontinuities, which were used to evaluate the support pressure from analytical method as proposed by Hoek and Bray (1981). These five prominent sets were identified out of 11 sets as also indicated by the Geological Survey of India.

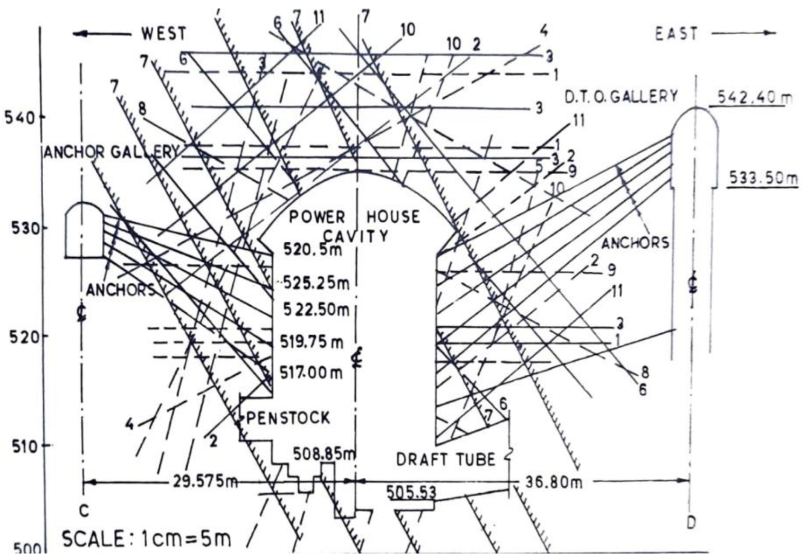

Figure 2 - Geological cross-section showing 11 sets of structural discontinuities along with prestressed cable anchors tied to an anchor gallery on the left-hand side and draft tube operating (DTO) gallery on the right-hand side

3.2 Engineering Properties of Rocks Around Powerhouse Area

Since the construction of large underground powerhouse complex was the first venture in Himalayan rocks, no prior experience or case records were available for reference. The design work was therefore carried out on the basis of assumptions inferred from the scanty data about rock characteristics using old traditional methods of investigation and testing. The construction of this powerhouse complex was considered as an experiment in prototype because there was hardly any rational method of design or any empirical or numerical method at the time of start of the project in 1966. Geological mapping of 76 ha of the proposed powerhouse area was done by 5 exploratory drifts with a total length of 690 m and 22 holes of 1700 m of drilling. The results of laboratory tests on representative samples obtained by diamond drilling and in- situ tests in a trial cavity of 8m span in one of the drifts are shown in Table 1.

Figure 3 depicts the interior perspective of the cavern with the control room located on the upstream side. The penstock ends and draft tube operating galleries (DTO galleries) are shown in the left and right side of the cavity.

4. NECESSITY OF LONG-TERM MONITORING

According to Terzaghi (1946), "There is little doubt in my mind that the forces which act on the tunnel are very much smaller than those assumed by the designers. The real load and stress conditions will be disclosed by the pressure cell and extensometer observations." The need for monitoring the behaviour of rock mass and the support system in underground excavations is obvious from the above two statements.

Table 1: Rock material and rock mass properties near the powerhouse area

Sl. No.	Properties	Value	Sl. No.	Properties	Value
1	Specific gravity of rock (γ)	2.74 g/cc	9	In-situ modulus of deformation (E_d)	14 GPa
2	UCS of air-dry sample (σ)	50.5 MPa	10	Horizontal in-situ stress	3.6 MPa
3	Flexural Strength (F_s)	18.2 MPa	11	Vertical in-situ stress	4.4 MPa
4	Modulus of elasticity (E)	56.2 GPa	12	Number of prominent discontinuities	5
5	Cohesion (c)	7.6 MPa	13	RQD	50 - 60%
6	Cohesion of gouge material (c_g)	0.01 MPa	14	*Jn and Jr	12 and 3
7	Coefficient of internal friction (f)	0.67	15	*Q	1.03 to 1.23
8	Angle of internal friction for gouge material (ϕ_j)	30^o			

Note: * = Evaluated during the course of the study; J_n and J_r = Barton's parameters; Q = Barton's rock mass quality

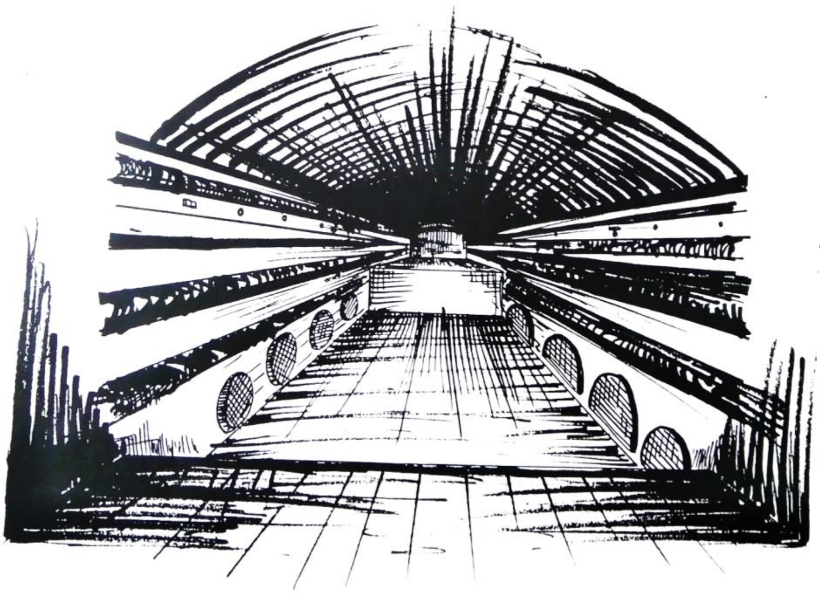

Figure 3 - Interior perspective of the cavern

In integrated design procedures, a number of concepts have been incorporated to make a design approach distinctive on its own. Generally, design methods such as empirical, observational and analytical are not required to be used separately to the exclusion of another method but all three should be utilized to provide a cross check on the design assumptions and the design recommendations. In some instances, this has met with considerable success and has led to the development of a new design philosophy or procedure which systematically integrates various design concepts into one design approach. However, the design method by observational approach or instrumentation may bring out the deficiencies in the existing empirical approaches or even in theoretical / analytical / numerical approach since these approaches and methods for estimating support pressure and designing support system for underground openings have not been found to give satisfactory results in case of Himalayas as was observed by Dube (1979) and Jethwa (1981). Significant portions of a majority of tunnels constructed in the Himalayas had to be rectified due to intolerable steel-rib deformations under squeezing rock conditions.

As there had been little information available on long-term performance of underground chambers, the present case study on the instrumentation data and evaluation of the long-term performance proved to be a very interesting case record leading to new insights on the subject.

Because of the lack of precedence for this work there was considerable interest in monitoring the post-construction behavior of the cavern. Long-term instrumentation was therefore planned to observe the behavior of the underground complex so that design assumptions could be checked against the results of the ongoing observations and timely action could be taken where problems are indicated. Therefore, a network of instruments (e.g. extensometers, rock-bolt load cells, strain meters, pore water cells etc. with different numbers in its brand name 'MDS' - Measuring Device System) was installed in the powerhouse complex.

In this project, all instruments were imported from a reputed firm and installed carefully. Most of the instruments functioned satisfactorily. In view of the unique opportunity and great demand for long-term data, observations were taken carefully. Also, the job was quite easy and automatic. The data of several instruments were found to be consistent even upto 10 years in some cases. The performance of the receiver was checked after eight years in 1981 at Tehri dam project and it was found satisfactorily (Mitra et al., 1988). The real problem is that voluminous field data at several projects is seldom analyzed carefully by the researchers. Often it is difficult to connect the whole puzzle well. If this is done attentively in future, the great value of the field data will be recognized soon. Figure 4 shows the perspective view of the underground powerhouse with its instrumentation layout.

5. INSTRUMENTATION IN THE POWERHOUSE COMPLEX

The steel-ribs provided for the support of the roof vault are subjected to rock pressures after its installation, ongoing sequences of excavation downward and post construction of the cavern. As a result of redistribution of stresses in the surrounding rock mass, the steel ribs undergo minute changes in the form of strains along its length. The installed instruments (strain meters - Type: MDS-16) measured such strains. The strain meters were of the vibrating wire type, firmly connected to the flanges since these had the best long-term performance. This wire was sealed against all external effects by protecting the tube, outside which the electromagnetic system was located. This system converts mechanical vibrations of the wire into electric oscillations which are transmitted back through the cable and the observations of strains are read by a remote readout system known as receiver (Type: MDS-3). Thus, the stresses can be computed from the observed strains.

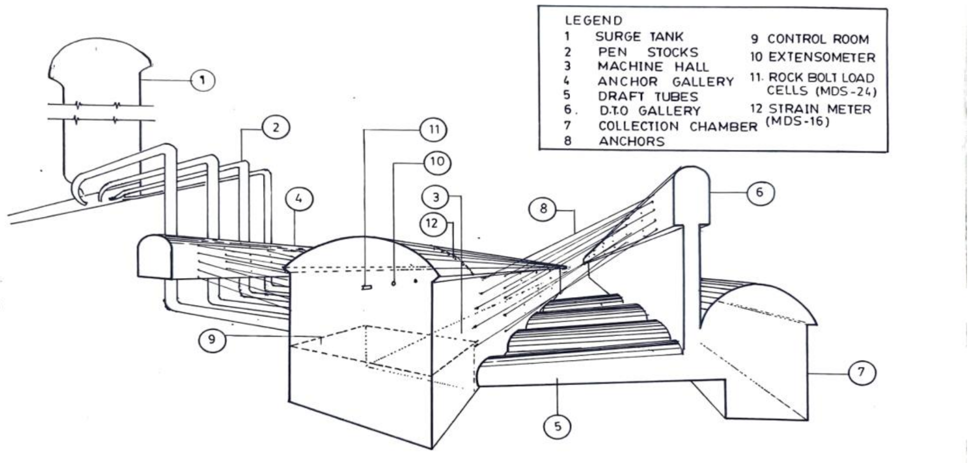

Figure 4 - Perspective view of the powerhouse complex showing its different components with instrumentation layout

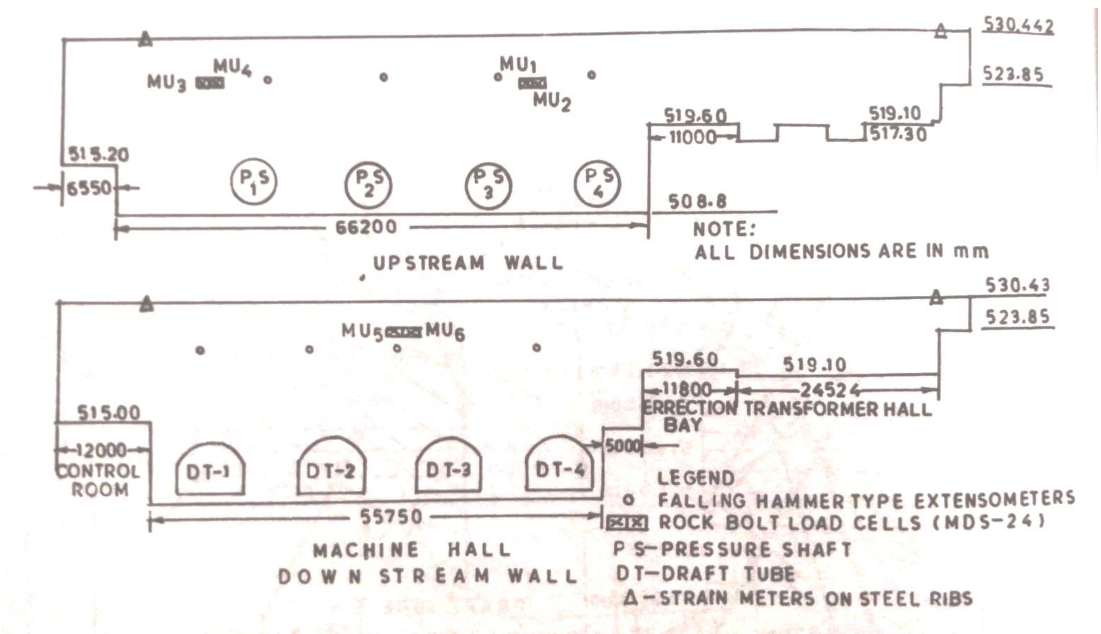

Figure 5 - Longitudinal section of the machine hall cavity showing positions of the various loadcells (MU1 to MU6)

Similarly, the rock bolt load cells (Type: MDS-24) were mounted axially on the steel rod popularly known as rock bolt and the tension or compression to which the rods are subjected at any moment is transmitted to the instrument. Thus, the rock bolt load cells were used to measure these forces directly and these forces are normally tensile in nature because of dilation of rock mass. The rock bolt load cells were mounted in pairs: named as MU (1 & 2), 3 & 4 and 5 & 6 as shown in Figure 5 at three different locations: two pairs on the upstream wall and another on the downstream wall.

The installation details of the strain meters, rock- bolt load cells, falling hammer type extensometers and pore water cells at different locations of the powerhouse complex are given below:

Table 2: Installation details of Instrumentation in powerhouse complex

Instruments and Location	Date of Installation
Strain Meters	
• Chainage 11m of cavern (12 nos.)	March 1973
• Chainage 108.5m of cavern (12 nos.)	April 1973
• Roof vault of the surge tank (8 nos.)	March 1973
Pore Water Pressure Cells	
• Pressure shaft liners (3 each in 4 shafts)	March 1973
Rock Bolt Load Cells	
• Two pairs on the upstream wall and one on the downstream wall	December 1974
Falling Hammer Type Extensometer	
• Four each on both the walls of the cavern	December 1974

The purpose of the extensometers was to measure the rock dilations near the surface of the excavation. A knife edge suspension arrangement was attached with a 4.8 m deep (40 mm dia.) grouted anchor bar, and an attachment of 6 mm dia. rod carrying a load (hammer) is suspended to keep the load in the proper position. The gap between the edges at the time of installation was kept 2 mm with the provision that if the same exceeds 6 mm, the hammer would fall down. This also acted as a warning system.

Figure 5 shows the longitudinal-section of the powerhouse cavern showing the positions of different instruments. Figure 6 shows the perspective view of the cross section and plan of the Underground powerhouse complex with its instrumentation layout.

6. ANALYSIS AND INTERPRETATION OF DATA

In the present paper, analysis of several field data generated from strain meters, rock bolt load cells, extensometers, pore water cells etc. for powerhouse opening, surge tank and pressure shaft were presented. The roof support pressures for the powerhouse cavern were estimated from strain meter observations while wall support pressures were worked out from the observations made by rock-bolt load cells.

The observed support pressure was compared with different empirical theories (Mitra, 1991; Mitra & Singh,1995).

6.1 Powerhouse Cavity

6.1.1 Cavern roof

Two test sections were chosen for installing strain meters at chainages 11.0m and 108.5m (Figs. 4 - 6). They were fixed at the lower flange of steel ribs after concrete backfill. The instruments showed relative change in strains with time from the date of their installation. Despite failures of a few instruments, some useful data of the excavation were collected, specifically with those instruments which had given consistent readings.

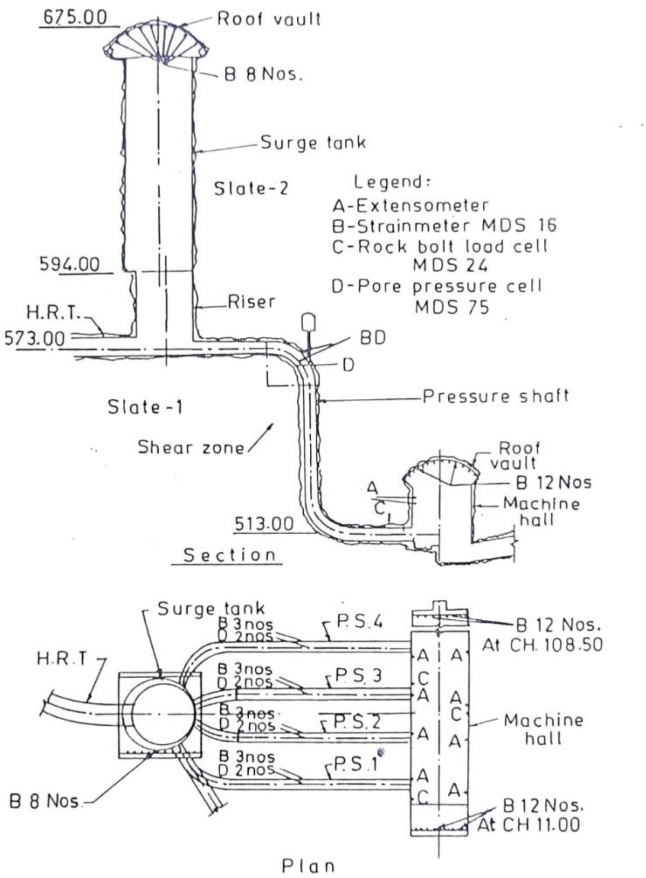

Figure 6 - Schematic diagram showing the cross section and plan of the underground powerhouse complex with instrumentation layout

The observations were taken regularly every month through a receiver. The subsequent readings at any moment of the time indicated the change in stresses at the lowest fibre of the ribs due to change in the support pressure. Since the ribs were erected much earlier and the transfer of immediate support pressure on the steel ribs had more or less taken place, the determination of actual strains in the ribs soon after their erection could not be possible. However, the instruments showed relative change in strains with time from the date of installations of the instruments.

It may be mentioned here that the excavation of the cavern was started in May 1967 and the entire work of excavation of the powerhouse cavity was completed in July 1969. Simultaneously the erection of steel- rib support system in the roof vault of the cavern was also carried out and was completed in April 1968. A total of 423 ribs were erected involving about 1200 tons of steel and the total time taken in excavation and supporting of the roof arch was approximately one year (Tandon and Agrawal, 1978).

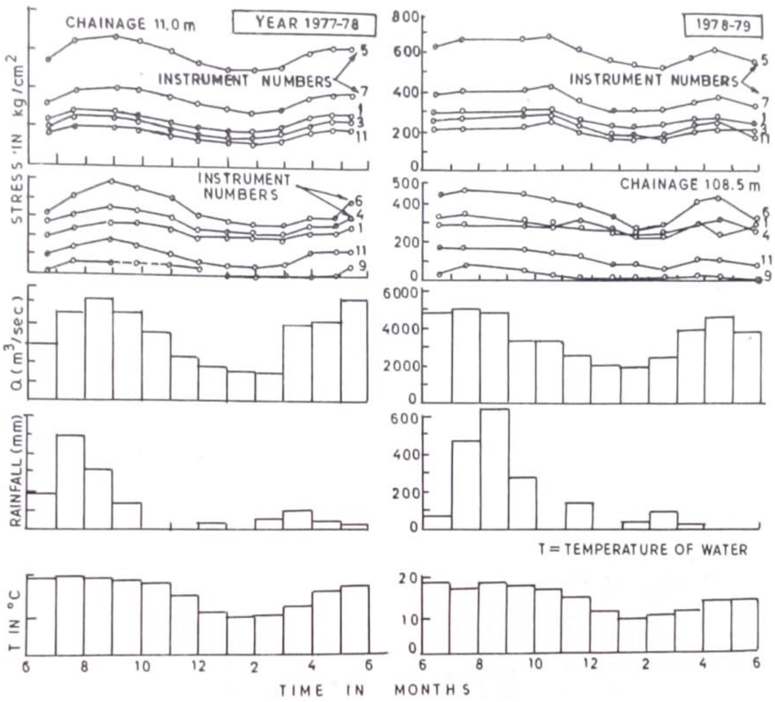

Figure 7 - Variation of stresses in steel rib (1 kg/cm^2 =0.1MPa), quantity (Q_d) of water drawn for power generation (m^3/s), rainfall (mm) and temp. of water in ^{o}C with time

Typical pattern of variation in stresses on the steel ribs with time has been shown in Fig. 7 for two years only. The date / month of observation is shown along the horizontal axis and the vertical axis shows – (i) the observed stresses in kg/cm^2 (ii) rainfall in mm, (iii) average quantity of water drawn for power generations every month in terms of discharge in cubic meter per second and (iv) average temperature of water in degree centigrade. The curves in above figures show the relative value of stresses in the steel ribs. Although the stresses measured by these instruments vary significantly, it may be seen that the stress variation with time, i.e., relative change in stresses are very consistent from instrument to instrument. More or less similar patterns of variation in stresses were noticed in subsequent years and the data collected till 1983, were used for analysis purposes.

A network of drainage galleries around the surge tank and drainage holes (spacing 3.5 m c/c) along the periphery of the roof of the powerhouse cavern across the self-drainage support system were provided (Fig. 8). Although the excessive seepage through limestone band towards the crown of the powerhouse cavity is being drained out by such a self-drainage support system, yet the effect of seepage on the temporary development of roof support pressure was noticed during monsoon and during peak period of power generation.

The variation in stresses fairly matches with the rainfall pattern (Fig. 7). During monsoon, the entire rock mass gets recharged due to rains which increases seepage through jointed rock mass and, therefore, the support pressure also increases temporarily. Similarly, it is also clear from the above figure that the stress variation in steel ribs had a definite trend with the pattern of quantity of water drawn for power generation. Similar patterns of variation in stresses, quantity of water utilized for power generation and rainfall with time were obtained in subsequent years also. The reason appears to be that an additional pressure on the roof of the powerhouse cavern is created due to extra seepage during the peak period of power generation when a higher quantity of water is drawn. It was also observed that the support pressures, in general, on the roof and wall (near underlying 25 m thick shear zone: 10 m deep from the lowest draft tube level) had increased after charging of the water conductor system.

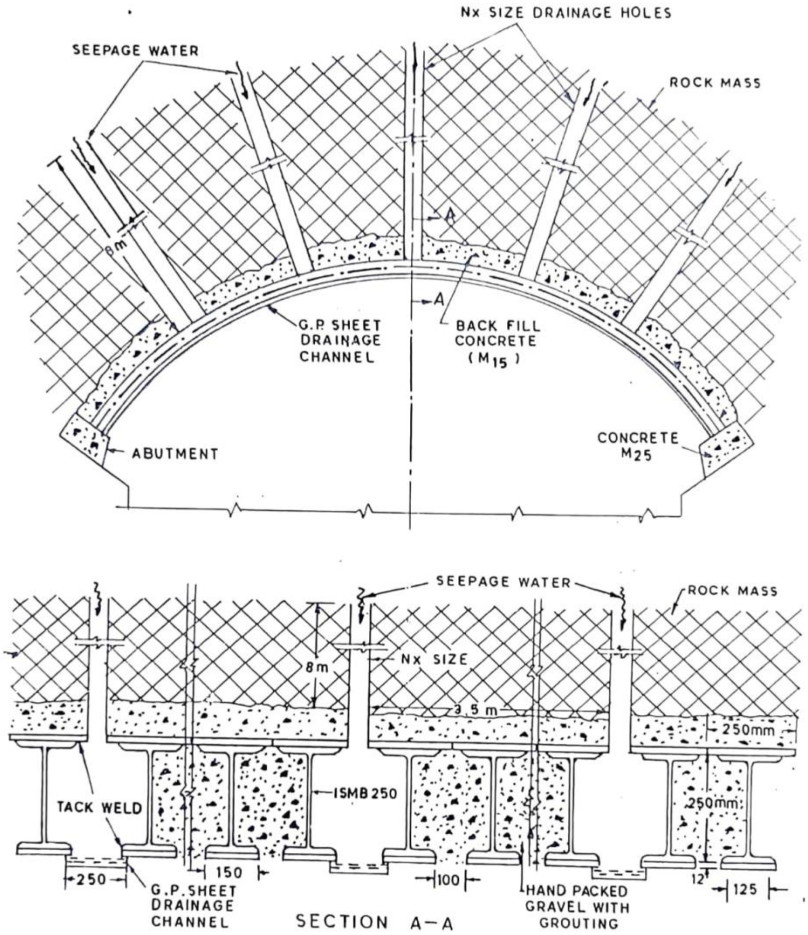

Figure 8 - Self-draining support system

From the above observations, it may be inferred that seepage caused by monsoon rains, charging of the water conductor system and upstream headrace tunnel / surge tank affects the roof pressure not only temporarily but also permanently. The seepage problem seems to be the major cause of persisting saturation of rock mass leading to significant time-dependent behaviour and ultimately results in substantial increase of long- term roof support pressure. On the other hand, no significant time dependent behaviour was noticed in case of cavern walls except near the underlying thick shear zone. This may be because the roof-vault is supported by a passive support system i.e., steel ribs backfilled with M-15 concrete while the cavern walls are designed by an active support system with a provision of prestressed cable anchors and shotcrete. The cause of wall support pressure developed near the thick shear zone has been attributed to recurring earthquakes of small intensity: M<5 as explained below in para: 6.2.1. It appears that the cumulative effect of recurring earthquakes has a significant effect on saturated rock mass near the underlying thick shear / fault zone.

Figure 7 also shows rhythmic variation in stresses matching with the variation in water temperature pattern. It appears that the decrease in water temperature during winter would cause an increase in viscosity of water and thus reduce seepage pressures during winter season and vice-versa. However, Cording (1991) mentioned that there has been a significant variation in flow in the vicinity of concrete lined tunnels on one of the hydro projects in the Eastern U.S. The effect was attributed to temperature strains in the rock that resulted in changes in joint aperture and a resulting change in permeability as the cube of the aperture. In the present case also, such phenomena seem to have occurred as the water cools, temperature decreases would cause a tensile change in strain and a resulting opening of the joint aperture, which would create more avenues for water to seep down faster. It may therefore be concluded that the effect on flow of seepage water due to change in viscosity is counterbalanced by the aperture change caused by temperature strains during winter season. Eventually, there seems to be no effect on support pressures due to variations in water temperature during the winter season. However, it is very clear that the rhythmic changes in stresses on the steel-rib supported roof are essentially caused by the two factors i.e., seepage water due to monsoon rains and the quantity of water drawn for power generation.

6.1.2 Cavern wall

Three test sections were chosen for installing the rock bolt load cells on the walls of the machine hall cavity (two on the upstream wall and one on the downstream wall). In all six load cells in three pairs at each section were mounted axially in between the prestressed rock anchors. The capacity of these anchors is 60 tonne with spacing of 2 to 5 m in both directions. These instruments were fixed at a height of about 16 m from the bottom of the base of the cavity at chainage 18 m, 42 m and 57 m respectively.

The length of the cable/rock anchors is 20 to 30 m., which is much higher than the width of the opening. The length of the downstream rock anchors is approx. 1.4 times the length of the upstream wall anchors. Due to the existence of shear zones, slip planes and joints in an erratic pattern, it was decided to provide a active support for walls of the cavern for long-term stability. Prestressed grouted cable anchors, threaded between the machine hall walls and anchorage galleries were provided with positive anchorage at both the ends. Immediately after the installation, the cable anchors were stressed to 50 to 55 tonne of load. The anchors were not stressed to its full capacity to allow for deformation of the rock after the anchors were grouted in place.

The rock loads induced on the cavern wall with time were measured by load cells. Results obtained for over a six years period are shown in Fig. 9 which shows variation in anchor load with time together with formation of intermittent peaks. It may be seen that the temporary increase in anchor loads (wherever peaks are formed) are due to either occurrence of earthquakes (M~ 4 to 5 occurred within 100 km range) or during heavy rain falls.

The time dependent increase in anchor load recorded by instrument no. 3 & 4 fixed on the wall near underlying shear zone and additional pressure observed in the steel ribs supporting the roof at chain age 11.0 m also located near the above thick shear zone are attributed to the cumulative effects of recurring earthquakes (M ~ 2 to 5 occurred within 100 km range). From the above figure, it may also be seen that the charging of the water conductor system had caused one of the largest irrecoverable increases in load on rock anchors at instrument no. 3 & 4 close to the underlying thick shear zone (Figure 1). As the walls were designed with active support system (i.e., provision of long cable anchors and shotcrete), no time dependent behaviour was observed due to effect of saturation caused by seepage after charging of water conductor system, monsoon rainfalls, upstream surge tank, and recurring earthquakes except near thick plastic shear zone.

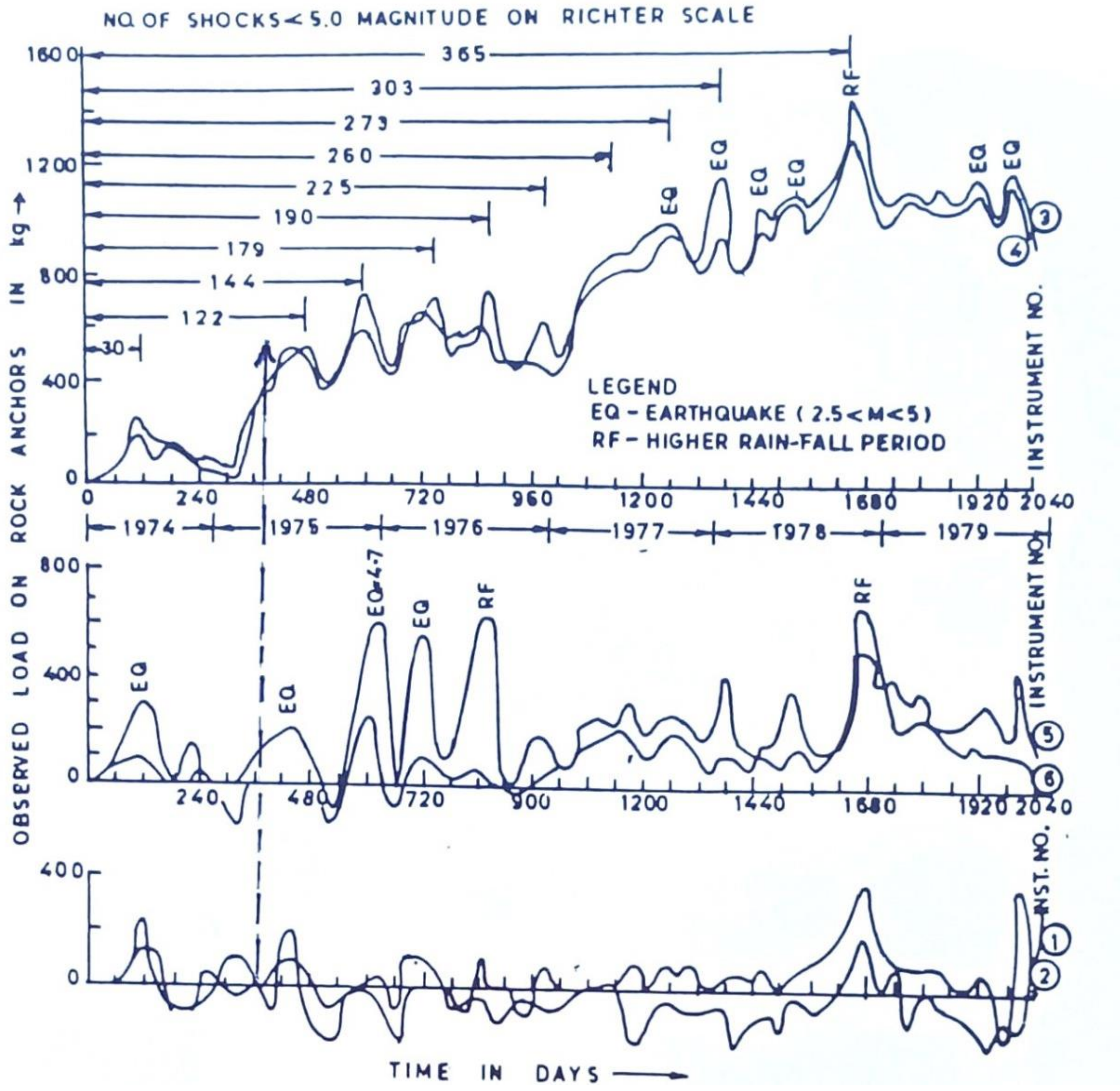

Figure 9 - Variation of anchor load with time together with earthquake shocks, magnitude, higher rainfall period (1974-79); vertical arrow showing the date of charging of the water conductor system in March, 1975

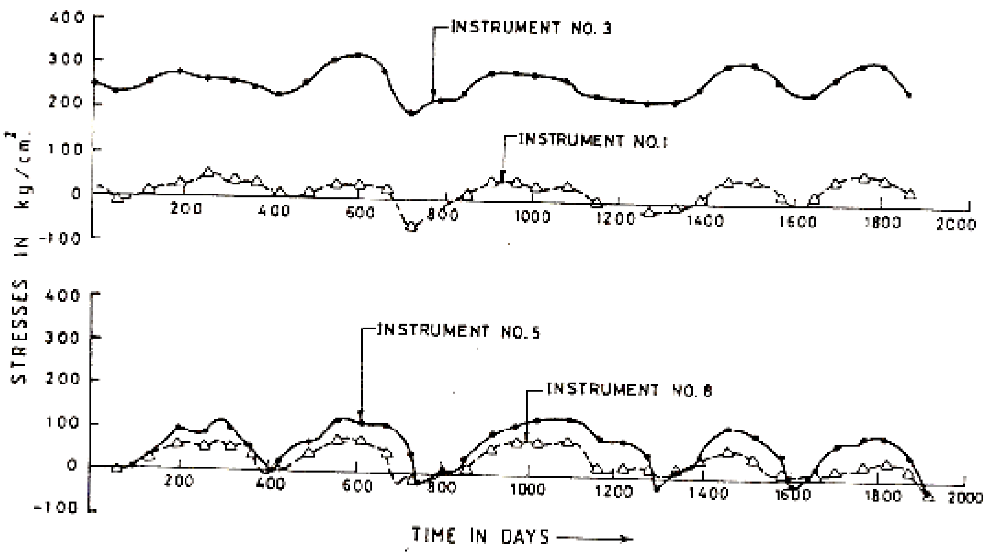

Figure 10 - Variation of stresses in steel ribs of surge tank roof with time

6.2 Surge Tank

Like a powerhouse cavity, a set of eight strain meters were installed on one of the ribs provided in the roof vault of the surge tank. The upper half portion of the surge tank is located in impervious slates whereas the bottom half of the structure lies in limestone band. The rock cover over the surge tank is about 200m. Figure 10 shows variation in stresses on the steel ribs with time. The observations taken for over seven years period indicated a periodic type of rock movement. It may be seen in the above figure that the stresses on the steel rib due to time- dependent deformations has not increased as in the case of powerhouse cavity (Fig.11). The roof of the surge tank, which is located in relatively impervious slate, remains more or less dry throughout the year because the effect of seepage through the walls of the surge tank is only downwards, i.e., on the roof of the powerhouse cavity as one would expect. Consequently, no effect of seepage on the roof support pressure is likely to take place in case of the surge tank. The effect of earthquake was also found negligible on the roof support pressures of the surge tank.

6.3 Pressure Shaft

The four steel lined pressure shafts (3.8 m internal dia) feeding the four turbines are located in limestone which is thinly bedded, interlayered with thin slate bands and intensely jointed. The two pair of shafts are 110 m and 120 m long backfilled with M-20 concrete having a minimum thickness of 600 mm. The steel liners are designed for internal hydrostatic pressures as well as the external pressures which may arise either during grouting or when the shafts are dewatered.

Four sets of three strain meters each were installed in the steel liner of the penstock limb to monitor the performance of pressure shafts (Mitra and Singh, 1989). The instruments recorded the induced strains in the liner and hence allowed the hoop stresses to be calculated. Despite a number of instrument failures, data obtained for over a three-year period from only one pressure shaft could be analyzed. From the analysis of data, it was found that the stresses in steel liners are significantly influenced by the water level in the surge tank although earthquake vibrations do not. This may be because the shaft is inherently more stable than high caverns.

6.4 Revelation of the Importance of Close Monitoring

A set of rock-bolt load cells (instrument nos. 3 & 4) installed for the observation of time dependent behaviour on the upstream wall of the machine hall cavity near the underlying shear zone (25 m thick) showed continuous increase in wall support pressures, although other two sets of instruments (one each on upstream and downstream wall) installed far away from the above shear zone did not show any increase. Similarly, a set of strain meters installed at the chainage 11.0 m (near the control room located just above the underlying shear zone) on the roof of the machine hall cavity also showed additional support pressures which is 23% more than the stresses observed at chainage 108.5 m near transformer hall which is far away from the underlying shear zone. It may be because of the fact that the time dependent deformations are more predominant on the roof of the cavern designed with passive support system (steel-ribs backfilled with M-15 concrete), where the rock mass is also saturated due to seepage problem caused by rain falls, charging of the water conductor system and also from upstream surge tank / headrace tunnel (HRT). On the other hand, the walls of the cavern supported by an active support system with cable anchors and shotcrete are least affected by time dependent deformations due to creep, saturation and recurring earthquakes except near underlying thick plastic shear zone.

Although the rock mass around the entire periphery of the cavern (especially the upstream wall of the machine hall) also faces seepage problems as discussed earlier, no time dependent effect was observed on both walls of the cavity as shown by two sets of instruments- no.1 & 2 and 5 & 6, located far away from the above shear zone. This may be due to the fact that the walls of the cavern were designed with an active support system i.e., long rock/cable anchors tightly tied at the other ends with shotcrete (Figs. 2 & 4). This shows the visionary quality of the efficient engineers involved in the design and construction of this first underground powerhouse complex planned, designed, constructed and instrumented in the complex geological formations of the lower Himalayan region.

Thus, an obvious question arose as to why there is consistent / rhythmic rise in support pressure with time both on the roof and wall of the cavern near the thick shear zone. An attempt was made to reveal the cause of rise in support pressures on the wall as well as on the roof where an extra pressure was recorded. It appears that there are some residual strains in the rock mass (also saturated due to seepage problem) after each earthquake shock (M~ 2 to 5 occurring within 100 km range) near a major shear zone as a result of which the support pressures keep increasing with time. In Figure 9, the number of small earthquakes is mentioned from the start of observation to the formation of peaks, and the corresponding anchor load. It was also found that out of 15 peaks formed in the above figure, about 12 are attributed to earthquakes, 2 to heavy rainfalls and 1 to charging of the water conductor system.

It was also found that the temporary increase in anchor loads are directly proportional to the magnitudes of earthquakes and inversely proportional to the epicentral distance (Mitra and Singh, 1992). In short following are the key points of observation:

- Significant time dependent deformations due to saturation caused by seepage from upstream surge tank / charging of the water conductor system / rainfalls etc. are observed on the roof of the cavern which has a passive support system i.e., steel ribs backfilled with M-15 concrete.
- Time dependent deformations due to saturation caused by seepage on the walls of the cavern are found negligible. This is because the walls were designed with an active support system ie., provision of long cable anchors with shotcrete. This type of support system seems to have "Stitched" the neighbouring rock mass effectively making it monolithic and massive in nature.
- Time dependent deformations caused by recurring earthquakes were clearly observed both on the roof as well as on the wall of the cavern near the underlying thick shear zone.

7. SUPPORT PRESSURE

Support pressures for the powerhouse cavity were estimated from the empirical approaches of Terzaghi (1946), Deere (1969), Barton et al. (1974), and the analytical method of underground wedge analysis by Hoek and Bray (1981). The predicted support pressures from above theories were further compared with long-term support due to creep phenomena of rock mass for extrapolated observation in the roof of cavern. The long-term support pressure has been evaluated by extrapolating the 10-year data of stresses observed on steel-rib support for a 100-year period (Fig.11). This extrapolation appears justified to some extent because extrapolation is from one log cycle to two log cycles only. Generally, the time-dependent behaviour follows the power law in rocks.

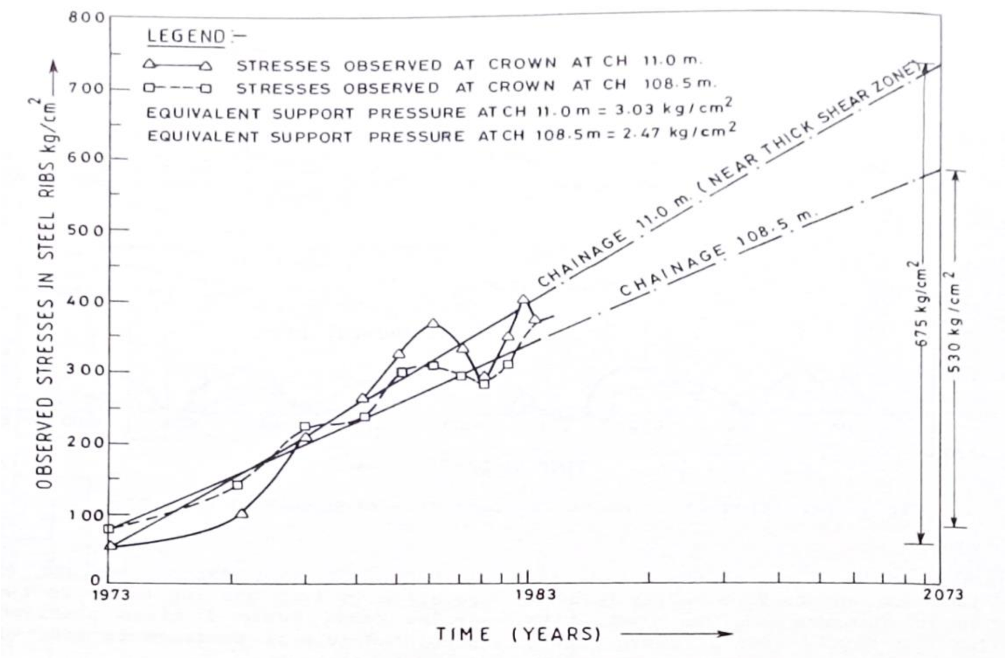

Figure 11 - Variation of stresses in steel ribs of powerhouse cavity at crown with log-time (yearly maximum average stresses observed with instrument no.7 at crown)

It is with this view also that the 10 years data was extrapolated to 100 years in a semi-log plot to estimate the long- term support pressures. The support pressure so estimated, being on the safer side, considers the significant time-dependent deformations of soft rock mass and due to gradual erosion of soluble gouge during seepage flow in the limestone roof. Table 3 gives observed and predicted support pressures in the roof of the cavern.

One of the test locations, i.e., chainage 11.0 m was at 40m from the underlying (25 m thick) shear zone. This location at chainage 11.0m falls in the control room section of the powerhouse cavern. Another test section, i.e., chainage 108.5m was located far away from the above shear zone (140 m) and falls in the transformer hall of the powerhouse cavity.

It may be seen in the Fig.11 that the roof support pressures have not stabilized and have been increasing almost logarithmically with time for 10 years from the date of start of observation. The increase in stresses at crown due to time-dependent deformations at chainage 108.5m, when extrapolated for 100 years is estimated to be approximately 530 kg/cm^2. Moreover, the time- dependent behavior is found to be more predominant near the neighborhood of thick shear zone as observed at chainage 11m. The stresses observed at this chainage, when extrapolated for 100 years, are found to be 675 kg/cm^2. This is 1.27 times the stresses observed at chainage 108.5m which is at a far distant place from the underlying thick plastic shear zone. The observations clearly indicate that long-term support pressures are as high as 3.3 times the short-term support pressures even after 8 years. This ratio may increase to 5.9 after 100 years.

Thus, the assessment of support pressure for powerhouse caverns from Barton's theory was found inadequate in view of the observed effect of hydro-geological conditions. Barton 's Q system does not take into account the soluble nature of rocks and seepage effect after charging of the water conductor system, i.e., after commissioning of the project. The seepage problem after charging of the water conductor system appears to affect the time dependent deformations significantly as observed in the cavern roof. Therefore, Jw should be reduced to about 0.66 according to seepage pressure at a later date.

Barton (2009) seems to have endorsed the above findings and mentioned "While more Indian engineering geologists may now be utilizing the Q- System in large hydroelectric projects in India and surrounding counties; the limitations in certain extreme conditions are not to be ignored." He further referred to a case record data from Q-system designed final support of a large rail tunnel in Norway highlighting a twelve-fold increase in tunneling cost and a ten-fold increase in tunneling time over the range of Q from 10 to<0.01. The steep curve seen in the middle of these measured trends, where 0.01<Q<1.0 are areas where pr-injection can improve bad ground, raising the quality of most or all of the six Q-parameters, by (high- pressure) penetration into one or more of the joint sets. Deformation modulus, wall deformation, velocity and permeability, and support needs, are each benefitted by the grouting, if high enough pressures are used.

In this context, the author would like to highlight the importance of grouting in the walls of the Chhibro underground powerhouse cavity (under discussion), which was constructed during the late sixties (1967-1968) when the "birth" of "Q" system had not even taken place.

Table 3: Observed and predicted roof support pressure for powerhouse cavern

<table>
<tr><th rowspan="2">Sl. No.</th><th rowspan="2">Observed/Predicted Support Pressures</th><th colspan="2">Location</th></tr>
<tr><th>North side of the cavern ch.108.5m far from shear zone</th><th>South side of the cavern ch.11.0 Near Shear zone</th></tr>
<tr><td>1(a)
(b)
(c)</td><td>Observed stresses in steel ribs at the crown
Equivalent increase in support pressures
Total observed support pressures taking into account Barton's short-term pressure</td><td>25.0MPa**
(after 8 years)
0.096 MPa
0.138 MPa
(after 8 years)</td><td>35.0 MPa
(after 10 years)
0.135 MPa
0.177 MPa
(after 10 years)</td></tr>
<tr><td>2(a)
(b)</td><td>Observed stresses extrapolated for 100 years
Equivalent long-term support pressure (observed)</td><td>53 MPa
0.247 MPa</td><td>67.5 MPa
0.303 MPa</td></tr>
<tr><td>3 (a)
(b)</td><td>Barton's short-term support pressure
Barton's long-term support pressure</td><td>0.42 kg/cm^2
0.73 kg/cm^2</td><td>0.42 kg/cm^2
0.73 kg/cm^2</td></tr>
<tr><td>4.</td><td>Short term support pressure assessment by Underground wedge analysis - Hoek & Bray (1981)</td><td colspan="2">(a) 0.028MPa for dry condition
(b) 0.095 MPa for water charged condition Pu=0.1 MPa (according to 'Q' system)
(b) 0.203 MPa for water charged condition assuming Pu=0.25 MPa</td></tr>
<tr><td>5.</td><td>Support pressure by
(a) Terzaghi's Rock Load classification (1946) (Considering the rock mass as moderately blocky and seamy as per Category 4 of Terzaghi's table)
(b) Deere's Classification Approach (1969)*</td><td colspan="2">0.164 to 0.230 MPa
(Height of the rock load is considered as H_p= 0.25 B to 0.35 (B + H_t). As under no circumstances the height of the unsupported cavern would be more than the width of the cavity and the same is excavated in sequence, the term H_t has been ignored for assessment of support pressure.
0.23 MPa</td></tr>
<tr><td>6.</td><td>Adopted support pressure for the design.</td><td colspan="2">0.38 MPa</td></tr>
<tr><td colspan="4">* Classification approach by Deere et al. (1969) gives the maximum value of roof support pressure (0.230MPa) considering the rock quality as fair (RQD ranging between 50 to 75%) as per Deere (1964)'s table of engineering quality of rock. For assessment of support pressure, H_t (height of opening) was ignored in this case also.
** The stresses observed at Ch. 108.5 m and Ch. 11.0 m have been converted into equivalent pressure as follows:
• Design Roof Support Pressure for 12m rock load (taking the rock density = 2.74 g/cc) = 0.329 MPa
• Calculated stresses in the steel-rib corresponding to 12 m rock load: 84.7 MPa
• Since 84.7 MPa of stresses corresponds to 0.329 MPa. Therefore, 25.0 MPa of stresses observed at Ch. 108.5 m after 8 years will correspond to an increase in support pressure: (0.329x25.0) / 84.7 = 0.096 MPa.</td></tr>
</table>

Well planned sequence of operations for the support of cavern walls through the pre-grouting of soft and jointed rock strata (consolidation of the strata carried out at a pressure of 0.4 MPa or more depending upon the intake of grout) followed by installation of cable anchors in the holes, its prestressing and finally the grouting of the entire length of holes at a pressure of 0.7 MPa led to significant improvements in rock mass quality. This was also confirmed by observations through rock-bolt load cells which showed negligible increase in long- term support pressure on the cavern walls except near the thick shear zone.

In short, following are the important inferences drawn from the comprehensive analysis of the instrumentation data:

- The percentage increase in support pressures caused by recurring earthquakes near thick shear zone after 100 years / designed life of the cavern is expected to be 23% of the long- term support pressure (0.303-0.247= 0.056 MPa as observed at Ch 11.0 and Ch. 108.5 (Table 3).

- An approximate estimate of increase in support pressure due to recurring earthquakes after 100 years from the author's dynamic support pressure theory proposed in his doctoral work (February 15, 1991) and published subsequently from time to time (Mitra and Singh, 1992, 1995, 1997; Goel et al., 2013) is found to be of the order of 0.038 MPa for walls and 0.052 MPa which is 21% of the long-term support pressure. The estimated and observed support pressure is likely to be small compared to long-term support pressure, 0.247 MPa. It may be noted that observed and estimated increments of dynamic support pressures (ranging between 21 to 23%) after 100 years due to recurring earthquakes is nearly 25% as suggested by Barton (1984).

- No time dependent deformations were observed where the rock-mass is dry i.e., roof of the surge tank and also the walls of the cavern (far away from the shear zone although the upstream wall of the cavity remains wet due to seepage problem). This may be because the walls of the cavern were designed with an active support system (provision of prestressed high tensile steel cable anchors with shotcrete) and therefore do not get affected by time-dependent behaviour neither by creep phenomena due to saturation of rock mass nor by recurring earthquakes. It is also worth mentioning that the upper half portion of the surge chamber is located in slates and remains almost dry throughout the year.

- The reason for the increase in roof support pressures appears to be that the modulus of deformation of rock mass reduces due to saturation after commissioning of the project and recurring rainfalls during monsoon and results in outward movement of rock mass. The increase in support pressure tries to counterbalance this movement depending upon stiffness of the support system. So, the effect of saturation should be considered properly in the design of underground openings for river valley development projects particularly in the rock mass whose modulus of deformation will decrease significantly after saturation.

- The underground wedge analysis by Hoek and Bray (1981) appears to give realistic values of support pressures for water charged rock, when compared with the extrapolated observed value. Therefore, underground wedge analysis may be used judiciously in the design of the support system for large powerhouse cavities.

The adopted support pressures as shown in Table 3 suggests that the powerhouse cavern would remain stable for 100 years as the effect of time-dependent rock deformations on the support pressures is well within the safe limits of provision of both active and passive support systems i.e., rock anchors-shotcrete and steel ribs.

8. EFFECTS OF WEATHERING AND EARTHQUAKES ON UNDERGROUND STRUCTURES

The effects of weathering and recurring earthquakes are generally observed on the structures located on the surface or at shallow depths in hilly areas. Weathering affects the mechanical properties of rock material as well as mass on the surface and at depths through physical and chemical weathering. However, there are no cases of underground structures which have suffered great damage in earthquakes. According to Barton (1984), there is a consistent record of underground structures having suffered much less than the surface facility during earthquakes. It is general observation that portal areas or fault crossings have suffered severe damage. In the case of portals within poor ground having stiff linings and amplified near surface shakings, make earthquake resistant design very difficult.

Powerhouse cavern generally constructed deep in the ground for hydropower generation may also be affected by continuous onslaught of weathering processes (for example: seepage caused by rainfalls / considerable variations in temperature of seepage water) and intermittent occurrence of small to high magnitude earthquakes in seismic/weathering prone areas, provided unfavourable hydro-geological conditions exist at project site. Such effects appear to have occurred in the present case also where one side of the cavern near the control room is located close to the major shear zone (25 m thick) and remains under the influence of seepage problems due to rainfalls and recurring earthquakes.

8.1 Role of Weathering

It is a well-known fact that the weathering of soft rocks is one of the primary causes of slope failure and shallow landslides in hilly areas. Most civil engineering works are located close to the surface where the ground mass is influenced by weathering that implies decay and change in state from an original condition to a new condition as a result of external processes. Weathering takes place in all environments but is most intense in hot, wet climates where it may be expected to extend to a great depth. The degree of weathering has been correlated with engineering properties and behavior of rocks by some researchers (Goel and Mitra, 2015). Both hard and soft rocks may behave differently when exposed to weathering conditions.

Santi (2006) proposed the relation between weathering grades (Viz. Fresh, slightly weathered, moderately weathered etc.) and various engineering properties (Compressive strength, Moisture content, RQD, Permeability, Seismic velocity etc.). With the passage of time, a fresh rock weathers to residual soil. Generally, the soft sedimentary rocks like siltstone, claystone, shales, mudstone, poorly cemented sandstone etc. having presence of fast weathering minerals and UCS less than 40 MPa are likely to be affected most by weathering during the engineering lifespan of the structures (Bell, 1983).

Gamble (1971) proposed a classification on slake durability with a group ranging from a very high durability to very low durability. Slake durability index gives an idea about the weatherability (resistance to weathering) of rocks. Nickmann (2006) proposed a classification system consisting of 6 categories of durability from VK0 (Hard Rock- UCS >150 MPa) to VK5 (Very weak - UCS< 10in poor soil or incompetent rock which have stability problems during excavation are more susceptible to damage during earthquakes, especially where wooden lagging is not grouted after construction of the final liner. Tunnels deep in rock are safer than shallow tunnels.

Howells (1980) reported that the International Tunneling Association (ITA) had set up a working group to study the seismic effects on underground structures. Its main objectives were to collect data on earthquake damage to underground facilities throughout the world and about current anti-seismic design procedures and to encourage research intended to explain the behaviour of underground structures during earthquakes. The working group setup by ITA felt that the observations are badly needed and since damage is fairly rare, instrumental records of the forces and displacement setup in tunnels which come through earthquakes are valuable.

During the last 4 to 5 decades, many researchers have worked on the performance of underground structures subjected to dynamic loading and published useful findings on this subject. Wang et al. (2001) studied the effects of over 100 earthquakes on tunnels and found essentially eight patterns of cracks induced into the tunnel lining during earthquakes. Singh and Goel (2006) reported that tunnels running across active faults suffer severe damage due to differential displacements which are incompatible with structure strength.

Bhasin and Thomas (2013) highlighted that 25% increase in support pressure according to Barton (1984) 's seismic rock mass quality ($Q_{seismic}$) is in the range of 15 to 44% increase in maximum axial force that was observed for seismic conditions through numerical model studies. It may be noted that in the present case study, the observed and estimated increase in dynamic support pressures (21- 23% as discussed in Section 6.4) are also within the above range.
Lined and grouted tunnels are safer than unlined tunnels in rock. Shaking damage can be reduced by stabilizing the ground around the tunnel by improving the bond strength between the lining and the surrounding ground through grouting (Hashash et al., 2001). Damage at and near tunnel portals may be predominant due to slope instability.

The author feels that like slake durability test for assessing the weatherability of rocks, some dynamic testing methodology on rock specimens collected from project sites located in seismic areas should also be devised to assess the strength properties of rock mass after each earthquake shock on a long-term basis. This may help the project / design engineers to have an idea of the in-situ engineering properties of surface and subsurface rock mass changing with time due to recurring earthquakes well in advance so that safe and economical design of surface/underground structures may be carried out.

8.2.1 A new hypothesis on the mechanism of dynamic support pressure

As Chhibro underground power house complex is located in a seismically active region of weathering prone area of Himalayas, special attention was paid during the course of the study to observe its long-term behavior with utmost care. The findings of the study as discussed above (article 6.4) may help the engineers for proper planning and the safe design of underground structures proposed to be located under unfavourable hydro-geological conditions in weathering prone seismic areas.

On the basis of detailed study, Goel et al. (2013) suggested to consider Mitra's 1991 empirical equation (which takes into account 3 parameters namely Joint roughness number (Jr); Rock-mass quality (Q_{wall}); and total number of small earthquake (N_{eq}) having shocks intensity (2.5<M<5) and occurring within 110 km range during the lifetime of an opening, generally 100 years) for assessment of an additional support pressure to estimate long- term support pressure, in case the tunnels or underground structures are required to be located near a thick shear or fault zone. The empirical equation relating seismic support pressure near thick shear zone due to recurring earthquakes with rock mass quality of the cavern wall (Qwall) is as follows:

$$\Delta\, p_{eq} = \frac{0.2\ x\ 10^{-4}}{J_r}\, .\, N_{eq}\ (Q_{wall})^{-1/3} \qquad \text{(Mitra, 1991)} \qquad (1)$$

where

Δp_{eq} = seismic support pressure due to recurring earthquake near thick shear zone in MPa,
J_r = joint roughness number,
Q_{wall} = wall rock mass quality (2.5Q), and
N_{eq} = total number of earthquakes (2<M<5) within 100 km range within the lifetime of an opening.

The above equation (Eq. 1) may also be applicable for small cavities including tunnels also. The above empirical theory suggests that there may be some residual strains in the rock mass due to the effect of the nearby thick/fault zone. The support pressures near the shear/fault zone appear to increase significantly with time due to strains accumulated after each earthquake shock. This problem may not be significant if the shear/ fault zone is far away, i.e. 1.5 times the span of the opening (Mitra and Singh, 1995). Equation 1 does not account for predominant period of shocks, which is found to have some effect on Δp_{eq} (Okamoto, 1973).

8.2.2 Effect of 1991 Uttarkashi earthquake on different structures

An earthquake of moderate to strong intensity measuring 6.3 occurred in the early hours of October 20, 1991, which was centered near Uttarkashi, approximately 100 km away from the project site and lasted for about 50 seconds. This earthquake (may be termed as "God-made Seismic Test") reconfirmed the proposed hypothesis on the mechanism of dynamic support pressure. As discussed earlier, the proposed empirical equation may be used for an assessment of extra support pressure on a long-term basis in deep underground structures located near major shear/fault zones. Recorded damage due to the Uttarkashi earthquake to the underground powerhouse complex at Chhibro was limited to minor cracks in the areas closest to the underlying thick shear zone.

All the major structures pertaining to Yamuna Hydel Stage-2 were inspected and found that practically there was no major damage to the surface or underground structures and no danger to the safety of the structure was observed. The minor cracks and other important observations on the project site were outlined as follows:

- No damages were noticed in the Ichari Dam except that the discharge in seepage galleries of the dam was increased by about 45%.
- Out of 8 extensometers installed on the sidewalls of the cavern, two extensometers on the d/s wall adjacent to the control room (near underlying thick shear zone) showed significant rock deformations which were of the order of 1 to 4 mm. Also, a deep crack (2 to 4 mm wide) were observed to have formed diagonally upto a length of 3.5 m between these two extensometers after the earthquake.
- Horizontal hair cracks at a height of 0.5 to 2.5 m were observed on each column of the control room and d/s wall.
- Few anchor plates in expansion chamber adit, which have been used to support prestressed rock anchors, were observed to have stretched slightly during the earthquake. This resulted in formation of cracks in the shotcrete.
- In the portal at the main entrance of the powerhouse adit, two horizontal cracks were observed, the length of which were 5 m and 4.5 m having thickness of 1 mm. Inside the adit, two vertical cracks at a long gap of about 80 m were observed at a height of 1 m having thickness of about 0.5 mm. These cracks appear to have formed in shotcrete.
- There was absolutely no effect of the earthquake on the surge tank. Seepage discharge in the lower drainage gallery (LDG) and Head Race Tunnel (HRT) were measured and found no change in the discharge of seepage water, before and after the earthquake.

9. CONCLUSIONS

The objective of the present study was to collect the wealth of Instrumentation data and analyze in detail so that field data may be used to verify existing theories and development of new concepts. Without suitable verification through field monitoring, the value of any theory is limited. The analysis of exhaustive field data has led to the following major conclusions:

- The quantity of water drawn for power generation and the quantity of rainwater falling during monsoon periods seem to be major contributors for the development of temporary support pressures. Therefore, the support pressure assessment should account for seepage effect also where adverse hydro-geological conditions may prevail in future. The empirical support pressure theory by Barton et al. (1974) may not give precise predictions of the long-term support pressures because the effect of seepage after commissioning of the project and the effect of thick shear zone and recurring earthquakes are not considered.

- The roof pressure (supported by passive support systems viz., steel ribs backfilled with concrete) in the powerhouse cavity was observed to increase significantly due to time dependent deformations caused by natural phenomena of creep and saturation of soft and jointed rock mass after charging of the water conductor system as well as rain falls during monsoon periods. On the contrary, no significant time-dependent behaviour was noticed where the rock- mass is dry, i.e., on the roof of the surge tank (located in slates). The upstream wall of the cavern, although remains saturated, has also not experienced any time dependent deformations. The reason may be that the walls were specially designed with an active support system i.e., pre-grouting, prestressed cable anchors and shotcrete.

- The time dependent deformations were noticed only where there is saturation of rock-mass due to seepage problem i.e., roof of the cavern supported with passive support system (steel ribs backfilled with M-15 concrete) and also near thick plastic shear zone. The analysis of data further suggested that the ratio between long-term and short-term support pressure may not be a constant equal to 1.7 as generally assumed but would vary from 1.7 to 5.9 depending upon seepage condition, soluble nature of joint fillings, and the types of the support systems provided in the cavern. Near the shear zone, this ratio of long-term to short-term roof support pressure may be as high as 7.2, which is approximately 23% more than the support pressure observed at Ch.108.5 m which is far away from a major shear zone.

- The above findings are based on the analysis of exhaustive instrumentation data (11 years) for the first underground powerhouse complex designed and constructed (with passive support system in the roof and active support in the cavern wall) in the soft rocks of the lesser Himalaya - a classical case study discussed in in this paper. However, this may also be applicable to smaller size tunnels i.e., 6 m, 9 m and 12 m.

- The quality of rock mass (Q-value / RMR) shall improve significantly after pressure-grouting of the surrounding rock- mass followed by installation of long rock-bolts /cable anchors and subsequent jet grouting / shotcreting. Therefore, the ratio of long-term to short-term support pressure may not only be a constant equal to 1.7 as suggested by Barton (1974) but would also reduce to nearly 1. This has been revealed by long- term monitoring in the case of the cavern walls.

- There may be some residual strains in the rock mass due to the effect of a nearby thick shear zone. The wall support pressure near the underlying shear zone was observed to increase significantly with time due to strains accumulated after each earthquake shock. This problem may not be significant if the shear zone is far away, i.e., 1.5 times the span of the cavern.

- For the assessment of long-term support pressure due to recurring earthquakes, a new empirical theory (Equation 1) was proposed to estimate additional support pressure near major shear/fault zone in seismic areas (Article 8.2.1). The empirical equation takes into account the

Joint roughness number (J_r), Barton's Rock Mass Quality (Q_{wall}) and total number of earthquake shocks: N_{eq} ($2<M<5$ occurring within 100km range) within the lifetime of an opening- i.e., 100 years. The increase in additional support pressure near the major shear zone from the proposed empirical equation is estimated to be 21 % which is approximately the same as observed in the roof of the cavern (23%). It may be noted that the observed and proposed estimate is nearly 25% of the long-term support pressure as suggested by Barton (1984). This may be a sheer coincidence and needs further investigation.

- A strong intensity earthquake of 6.3 magnitude occurred on 20th October, 1991 at Uttarkashi - about 100 km from the project site on account of which minor cracks developed in different locations of the power house complex. The majority of cracks were observed near the underlying thick shear zone which appears to confirm the proposed hypothesis on the mechanism of dynamic support pressure.

- The observed stresses on the roof arch of the surge tank, which is located in slates and remains almost dry throughout the year, have indicated a periodic type of rock movement. There is no increase in support pressure with time due to time dependent deformations as in the case of powerhouse cavern.

- The stresses in the steel liner of the pressure shaft are significantly influenced by the water level in the surge tank. However, earthquake vibrations do not appear to affect the liner stresses since the shaft is inherently more stable than high caverns.

- The support pressure in the power house cavern was observed to increase temporarily during heavy rainfalls and during earthquakes. The percentage increase in static anchor load due to a single small intensity earthquake ($M\sim5$) is less than 2%.

- The effect of weathering in deep underground structures appears to be less than 10% as compared to surface structures if the cavern is located under unfavourable hydro-geological conditions in rainfall prone areas.

- Underground wedge analysis by Hoek and Bray (1981) may give a realistic range of support pressures when the seepage water pressures after commissioning of the project are taken into account according to joint water reduction factor (J_w) in 'Q' system.

- The long-term effect of saturation due to seepage caused by charging of the water conductor system and recurring rainfalls should be considered properly in the design of underground powerhouse cavities for river valley development projects particularly in soft and jointed rock mass whose modulus of deformations will reduce considerably after post construction saturation and where adverse hydro-geological conditions may prevail in future.

- Laboratory test should be devised to determine the loss of weight, volume and strength of highly soluble rocks and gauge due to long- term seepage.

- The proposed concept on seismic support pressure due to earthquakes needs verification by similar field studies and dynamic instrumentation at other projects. Correction factor should also be introduced for the dominant period of motion. It is also likely that in jointed permeable rock, J_w may decrease significantly during major earthquakes in the neighborhood of faults.

- The value of any theory is limited without suitable verification through field measurements. After detailed study on other river valley development projects, efforts should be made for necessary modifications in the existing empirical theories.

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