Influence of Geological Features on Long-term Behaviour of Underground Powerhouse Cavities in Lower Himalayan Region - a case study

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When you can measure what you are speaking about, and express it in numbers, you know some thing about it but when you cannot measure, when you cannot express it in numbers, your knowledge is of a meagre and unsatisfactory kind, it may be the begining of knowledge, but you have scarcely in your thoughts, advanced to the stage of science. Whatever matter may be.

- Lord Kelvin (1824-1907)

ABSTRACT

The reported case history pertains to the first underground powerhouse constructed and instrumented about 27 years ago in soft rocks of the lower Himalayan region, which is seismo-tectonically active. The analysis and interpretration of data for over ten year period for Chhibro Underground Powerhouse Complex (in dolomitic-lime stones) has provided useful insight into the long-term behaviour of large powerhouse cavern. The study has proved to be helpful in identifying unknown deficiencies in the existing theories. Methodology for analysis and interpretation of complex instrumentation data has been developed. Seepage caused by charging of the water conductor system is observed to have affected support pressure significantly as has recurring earthquake shocks close to thick shear zones. Time-dependent effect has been observed significantly (in saturated rockmass) where there is seepage poblem, i.e. roof of the powerhouse cavity and also near thick shear zone. However, no considerable time-dependent deformations has been noticed where rockmass is dry, i.e. on the roof of the surge tank (located in slates) and the walls of the powerhouse cavity. An empirical support pressure theory has been proposed to account for accumulated strains in rockmass near thick shear zones due to recurring earthquakes in seismic region.

1.0 INTRODUCTION

"Most tunnel construction monitoring has in the past been of benefit to research and the advancement of tunnelling practice, wittingly or not. The development of monitoring instrumentation for the benefit of the project, from which the construction data are recovered, has very significant potential for risk and cost reduction in tunnelling".

- Dunnicliff and Schmidt (1974),

The design of support system for underground openings is based on past experience, model studies and theoretical computations. The support parameters so determined, may not be true representative of actual rockmass condition because the insitu properties of rockmass and its behaviour are not exactly known. Moreover, the nature of movement and stresses around underground opening varies from place to place. Consequently, the temporary supports are either underdesigned or overdesigned as observed in the most of the tunnelling works in Himalayas.

Monitoring the behaviour of underground excavations during construction has definitely provided reasonably useful aid in the design and construction of underground structures. This method of design, however, does not take into account the effect of long-term time-dependent deformations (creep effect) in soft rockmasses such as in the lesser Himalaya. The change in rock fabrics due to seepage problem and other environmental effects (viz. change in pH value) after long reservoir operations have not been considered in the development of empirical theories. The rockmass behaviour may change significantly after charging of the water conductor system, i.e., commissioning of the project and therefore the study of long-term behaviour of the structure becomes essential in order to verify the empirical theories. This also facilitates in improving the existing methods of design to a great extent.

In recent years the researchers and professional engineers have shown much interest in the field of instrumentation to study the long -term performance and consequently many power cavern and tunnels were instrumented to monitor the behaviour of the opening during construction. However the case studies regarding the post -construction performance of the large underground opening are found very rarely.

In order to study the post-construction behaviour of a cavern on long -term basis, the field data has been collected for 8 - 10 year period (from 1973 to 1983) from Yamuna Hydroelectric Project stage-II Part-I for Chhibro underground powerhouse complex, which has set a major precedent by being the first venture of its type in the soft rocks of the lesser Himalaya, which is seismo-tectonically active. The complex essentially comprises of a powerhouse cavity which is 113.2 m long, 18.20 m wide and about 32.5 m high, a surge tank with 20 m diameter and approximately 100 m high and four pressure shafts feeding the four turbines of the powerhouse. During construction, a net work of instruments (e.g. extensometer, rock bolt load cells, strain meters and Piezometers) was installed at different locations of the powerhouse complex. Regular records were also kept of other important data such as rainfall, earthquake, quantity of water utilised for power generation, temperature etc. which may be relevant for the purpose of analysis.

The analysis and interpretation of field/instrumentation data has proved to be an useful exercise leading to some important inferences in favour of the safe design of underground powerhouse cavity in soft rockmass conditions as prevailing in the lower Himalayas. The existing support pressure theories do not take into account the effects of thick shear zone, recurring earthquakes and time-dependent deformations due to seepage problem after commissioning of the project. The analysis and interpretation of field data for the first large cavern in lower Himalayan region published in parts from time to time (Mitra et al. 1988a, 1988b, Mitra & Singh 1989, 1991; Mitra, 1991; Mitra & Singh, 1992, 1995a, 1995b) and now presented in this paper in detail has been helpful in verification of empirical theories and development of new concepts.

2.0 PROJECT LAYOUT

Hydroelectric stations need heavy financial investment, but once commissioned have very low comparative running costs as the basic 'fuel' is free. With rising prices and scarcity bound to affect fossil fuels in future, and environmentalists exerting an increasing influence against nuclear power plants, it obviously makes sense for those countries with high mountain ranges to go all out for hydropower.

-Martin (1987).

2.1 General

It is well known fact that immense hydroelectric potential exists in the Himalaya in the perennial snowfed rivers. Several of these rivers are being harnessed for the generation of the much needed power by constructing storage dams or diversion weirs. The stored water behind these structures have been carried through long tunnels to the powerhouse. However, at many places, the construction of large underground openings for power stations in the Himalaya has become indispensable due to topographical, landslides and other restraints. Consequently, the large underground excavations for hydroelectric development in this region has assumed significant role in the context of complex geological nature of the prevailing rocks.

2.2 Yamuna Hydroelectric Project, Stage-II Part-I

The Yamuna hydroelectric scheme stage-II involves the development of the power potential of the Tons river, a tributary of the Yamuna, between Ichari and its outfall at Dakpather in Dehradun district, U.P., India. The total head available is about 188 m which is being utilised in two stages. Figure1 shows schematic profile of Yamuna Hydroelectric Project, stage-II.

Part I uses the 240 MW Chhibro power station to exploit the drop of about 124 m along the first loop between Ichari and Chhibro which was the first venture of its type in the Lesser Himalaya (Siwalik) and was necessitated because the location of a surface power station would have involved large scale excavation of steep slopes. Extensive geological investigations were undertaken to optimise the location of the underground power station.

The final decision led the siting of the power-house complex in a band of limestone which has a horizontal width of 193 to 217 m. The Chhibro complex comprises a network of excavations for the machine, transformers, turbine inlet valves and control room and also provides operating galleries and hydraulic connections to the part II stage. This latter stage involves a 120 MW scheme utilising the remaining drop of 64 m along the second loop between Chhibro and Dakpather. A general layout of the Chhibro underground power house complex is shown in Fig. 2. Part I of the Yamuna Hydroelectric scheme stage II commenced in March 1966 and became operational in March 1975. The components of the project are:

i) A 60 m high concrete diversion dam across the Tons river at Ichari together with spillway, intake sedimentation chambers, flushing arrangement and tunnel control structure.

ii) A 6.3 km long concrete lined head race tunnel of 7.3 m finished diameter with design discharge capacity of 235 cumecs.

iii) A 20 m diameter of 100 m high underground surge tank at the end of the headrace tunnel.

iv) Four steel-lined pressure shafts of 3.8 m diameter feeding the four turbines.

v) Underground powerhouse at Chhibro with four machines of 60 MW each.

vi) Tailrace works comprising of collection gallery, expansion chamber, outlet tunnel, outlet structure and a syphon tunnel interlinking the tailrance works of part I with the headrace tunnel of part II.

Because of the 6.3 km length of the headrace tunnel, it was necessary to provide a surge shaft upstream of the powerhouse to serve the following functions:

i) To provide a free reservoir surface close to the machines as a rapid means of compensating for water hammer effects, thereby limiting penstock pressures and materially reducing water pressure in the tunnel.

ii) To supply the additional water required by the turbines under increasing load demand until the conduit velocity has accelerated to a steady state value.

iii) To store water during load rejection until the tunnel water velocity has decelerated to a steady state value.

iv) To ensure that oscillations of water level in the surge tank due to the load changes are rapidly damped.

3.0 A BRIEF REVIEW OF DESIGN AND CONSTRUCTION DETAILS OF THE POWER HOUSE COMPLEX

3.1 Roof Support System for Cavern

The observations during geotechnical investigation suggested that the use of rock bolt may not be found suitable for supporting the cavern roof. Geological studies revealed the presence of shear zones having crushed and water-laden-erodible material together with complex jointing pattern in the limestone band. Hence special sequence of excavation and mode of supporting the cavern roof was considered safe.

The provision of structural steel arch support was considered in the roof of the cavity. The idea of providing steel ribs was to give positive rock support concurrently with excavation and exercising control on unsupported span during the course of construction, by limiting the pull and extending the roof support close to the surface. Also, the steel ribs adjust themselves to creep/rock dilation along the roof and the abutments without any significant damage.

The section of the roof supporting steel ribs was worked out as rolled steel joists of 250 mm x 125 mm with cover plate of 250 mm x 20 mm at the top and 150 mm x 20 mm at the bottom of the flange. The ribs were placed side by side at 250 mm centre to centre with provision of nominal drainage gaps at every 3.5 m interval. Thus, the top flange plates were continuous and no shuttering was required for placing the blocking concrete. M25 concrete was used for the abutments whereas M15 for backfill concrete .
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.

3.2 Wall Support System for Cavern

The support system of the cavern wall consisted of 5 to 10 cm thick chain link fabric and prestressed cable anchors (Giffor-Udall system of positive anchorage). On the western side (upstream wall) an anchor gallery (4 m wide and 6 m high) was constructed parallel to the machine hall cavity and this is located at a distance of about 18.5 m from the face of the power house so that one of the anchor ends could be tied in the gallery and the other end in the powerhouse wall. On the eastern side (downsteam wall), the existing draft tube gallery located at a distance of 25 m was used to similar positive anchorage (see Fig. 3.).

The prestressed cable anchors of 60 ton capacity each spaced at 2 to 5 m in both horizontal and vertical directions were provided to support the walls of the cavity. A total of 445 anchors were installed; 231 on the upstream and 214 on the downstream walls. The average length of the anchors is 23.5 m. Five rows of anchors were provided on the upstream wall and six rows on the downstream wall. Reinforced shotcrete of 7.5 cm thickness was also used, where necessary.

3.3 Support System for Surge Tank

The roof vault of the surge tank has a maximum clear span of 24.5 m. Rock load above the roof was estimated as per Terzaghi's empirical theory. Accordingly, the arch ribs were designed for a rock load of 19 m height. In addition to this, self load (dead weight of steel ribs) equivalent to 0.5 m height of rock and temperature effect of $\pm 15^0$ C were considered. The roof vault was supported by Rolled steel Joist (RSJ) ribs (300 mm x 140 mm @ 44.2 kg/m) strengthened by steel flange plates 210 mm x 25 mm at the top and 125 mm x 25 mm at the bottom, spaced at 215 mm centre to centre. Reinforced concrete lining was provided along the periphery of the surge tank. The thickness of the lining at the top and bottom are 400 mm and 1900 mm respectively. Contact and consolidation grouting were carried out through the holes drilled through pipes left in the concrete lining. The length of drill hole for contact grouting was kept 0.3 m and up to 6.0 m for consolidation grouting (Gupta et al. 1985).

4.0 GEOLOGICAL DESCRIPTION

It is extremely important that the quality of the input data matches the sophistication of the design methods. Determination of the input parameters for design should be so planned that as much quantitative data as possible is obtained rather than relying on qualitative description. Determining the input parameters for rock engineering design will be of prime concern both to the design engineer and the engineering geologist.

-Bieniawski (1984)

4.1 Regional Geology

Lesser Himalaya owing to their unique tectonic history presents a complex hetrogenous mass. The rockmass formations, in general have many folds, faults and thrusts which have been formed due to tectonic deformations. The slopes of Himalayan valleys are steep and unstable. The weak and soft rockmass are susceptible to weathering and erosion. The young rock formations comprises mainly of sandstones with bands of steep dipping shear zones of clay and siltstones. Such rocks are highly weathered and jointed.

The region under reference falls under the Kumaon Lesser Himalaya, adjacent to the main boundary fault. The Lesser Himalaya are thrusted over the foot-hill belt consisting of a narrow spread of Siwalik sediments. The Kumaon Lesser Himalaya are marked by a strong tectonic activity and are traversed by a number of secondary faults and thrusts. The secondary fault zones diverge in a westward direction and at a places roughly merge with the main boundary fault towards the east. For over 400 km, towards the east to Krol thrust runs parallel and almost coincides with the main boundary fault (Agrawal, 1970). The area under study lies in this zone.

4.2 Structural Features

The major structural features in this area are the two main boundary faults running from Punjab to Assam along the foothills of the Himalaya. The faults are known locally as the Nahan and the Krol thrusts. Their outcrops were observed across the river Tons near Khadar and at a few gully (nullah) exposures near Kala-Amb and Kalawar. The dips of the Nahan and the Krol thrust vary from 27^0 to 30^0 due N 10^0 E to N 10^0 W, and 26^0 due N 26^0 W, respectively. The Krol thrust has brought the Mandhalis over the younger Subathu-Dagshai Shales. the thrust plane shows heavy crushing and copious water seepage. The Nahan thrust has brought the Subathu-Dagshais in juxtaposition with the Nahan sand stones. The thrust plane does not show any marked crushing in the sandstones, possibly due to the weak character of Subathus which have borne the brunt of crushing, nor any water seepage. Thus the characteristics features of the Nahan thrust is only the stratigraphic position of the formations involved, and the drainage of the area appears to be controlled by structural features. Several streams flow along these thrust planes. In some places in this area, with narrow outcrops of the intra thrust zone material, these streams have removed the Subathus, bringing the Mandhalis directly over the Nahans, producing impression of the two thrust planes having merged. Thus misleading outcrops have been produced by the differential erosion in the region. Other planes of movements are in the form of tear faults or the recent scree material being over ridden by the Subathus (Jethwa, 1981).

4.3 Seismic History

Most of the earthquake activity in India occurs along the Himalayan belt extending from Jammu & Kashmir to Nagaland. The peninsular India and the Indo-Gangetic plains have also been affected by moderate earthquakes in the past. The entire sub-Himalayan region is prone to seismic disturbances as has been evident by earthquakes in the past. The Garhwal Himalayan belt along the Ganga and Yamuna valleys is also seismically active and has been marked by frequent occurrence of moderate size earthquakes (Arya et al. 1971). The project area lies in the Garhwal Lesser Himalaya adjacent to the main boundary fault. The region is transversed by a number of secondary faults and thrusts and is known for intense tectonic activity.

From the existence of major thrust and a great number of secondary planes of movements, it is evident that the region under reference has been subjected to seismic activity in geologically recent times. The age of these movements is not known with any certainty but Auden (1933) considered that the Krol thrust might have resulted from the seismic activity spread over a long period of time from pre-Pliocene to post-Pliocene. In recent times, the only major earthquake reported any where near the area as mentioned earlier was the 1905 Kangra earthquake and 1991 Garhwal earthquake. As a result of 1905 this earthquake, the town of Dehradun was lifted by about 13 cm relative to the city of Mussoorie. Other indications of the recent tectonic activity in the region are huge boulders of quartizites found embedded in the breciated, pulverised and gouged material along the Nahan Thrust (Jethwa, 1981).

4.4 Geological Features Around Powerhouse Area

4.4.1 Head Race Tunnel

The tunnel approximately runs from north to south and is located in a syncline fold locally known as Jaunsar Syncline. This is intersected on either side by two thrusts namely Tons & Krol dipping in the opposite direction. The axis of the syncline runs approximately in E-W direction.

The length of the head race tunnel is approximately 6.2 Km. The tunnel is horizontal from dam side in a length of 2.5 Km and have different gradients in remaining length. This gradient varies from 1 in 88 to 1 in 721.

The tunnel is supported by steel ribs with blocking concrete and 75 mm thick shotcrete. A total length of approximately 3.3 Km of tunnel is supported by steel ribs, 2.5 Km by shotcrete and remaining 267 m is unsupported in chainage of good rock conditions. The entire length of the head race tunnel is lined with concrete. The thickness of the lining increases with increasing internal water pressure in the tunnel.

4.4.2 Surge Tank

The surge tank at the end of long headrace tunnel on the upstream of powerhouse carries water from Ichari Dam reservoir. A100 m high and 20 m diameter underground surge tank is located in limestones and slates. Riser of the surge tank falls in limestone sub unit and overlying slate sub unit. The Dhaira limestones unit of Mandhali series is thinly bedded, interlayered with minor slate bands, and is intensely jointed. It also contains numerous thin bedding shear zones. The shearing is prominent mostly at the contact of the intervening slate layers. The slate sub unit is thinly bedded and jointed and contains subordinate thin limestone bands.

The Dhaira limestone unit in which the cavity is located dips at an angle of about 45^0 in $N15^0E$ to $N20^0E$ direction exposing thinly cleaned slate bands. The upper portion of the surge tank lies in thinly bedded slates whereas lower half of the cavity is located in limestones. The over burden at the surge tank is about 200 m. The terrain slopes towards powerhouse direction. The side rock cover around the surge tank at the bottom and top is about 100 m and 81 m respectively. The surge tank with expansion gallery is designed to contain the maximum possible upsurge as no spillway has been provided. Three tiers of circular drainage galleries and drainage holes are provided alround the surge tank

4.4.3 Pressure Shafts

There are four pressure shafts feeding the four turbines of the powerhouse. All the shafts are located in limestone which is thinly bedded, interlayered with thin slate bands and intensely jointed. The limestone also includes thinly bedded shear zones mostly at the limestone-slate contacts.

4.4.4 Powerhouse Cavity

The powerhouse cavity at Chhibro is 113.2 m long, 18.20 m wide and about 32.5 m high at the point of maximum excavation. The cavity has a circular roof arch with rise span ratio of 0.27 and vertical sides. The powerhouse cavity accomodates control bay, machine hall, erection bay, transformer hall, air conditioning unit and ventilation chamber.

The powerhouse excavations are located in band of stratified limestones of about 140 m thickness (210 m across in the horizontal direction) with minor or thinly bedded slate bands. The rock is closely jointed with numerous shear zones ranging from 20 to 500 mm thick and which are sub-parallel to bedding. A major shear zone approaches to within 10 m of the lowest draft tube level in the powerhouse area. The formations dip at about 45 towards N 10^0 W to N12^0. The excavation is aligned normal to the strike of the rock formations with cover ranging from 208 m over the control room to more than 250 m over the transformer hall.

Figure 3 shows geological cross-sections of the rock along the short axis of the powerhouse cavern. The entire cavern lies in a thick band of limestones, interlayed with minor slates. The rocks are closely jointed and a series of shear zones exist in the area in random directions. Due to existence of shear zones, slip planes and joints, the rockmass was not considered competent enough for large underground excavations at the time of investigation. Figure 4 shows geological section along the long axis of power house complex showing surge tank and pressure shafts.

The rocks exposed in the vicinity of the powerhouse area has been further investigated during the course of this study. This has revealed the existence of the five prominent geological discontinuities, which have been used to evaluate the support pressure from analytical method as proposed by Hoek and Bray (1981). The prominent geological discontinuities are shown below :

The prominent geological discontinuities

S.No.	Dip Amount	Dip Direction	Joints spacing
1.	40^0	N12° E	10 cm
2.	40^0	S5° E	2.5 to 5 m
3.	50^0	S80° E	1.5 to 4.0 m
4.	35^0	S85° W	3.5 m
5.	55^0	S30° W	5 to 8 m

These 5 prominent sets of discontinuites have been identified out of 11 sets as indicated in Figure 3 and checked by the field study.

5.0 INSTRUMENTATION LAYOUT

"Where a rock or soil mass is disturbed either by natural or man made events, it undergoes a redistribution of strees, accompanied by a change of shape. These adjustments, the exact nature and magnitude of which can seldom be accurately predicted, may involve major cost and safety measures. The adjustments are reflected in displacements, deflections pressures, loads, stresses and strains which can be detected and measured using instrumentation technique. These data are very useful for optimal design of an underground opening".

- Dutro & Perry (1987)

5.1 Planning of Instrumentation

The planning, design and construction of the Chhibro underground powerhouse complex was done with limited data on the rockmass as no prior data base of insitu characteristics of the rocks of this region was available. The behaviour of rockmass was observed as elasto-plastic-brittle and locked tectonic stresses were found to be invariably present. As a result of this situation, the excavation of large cavities usually gives rise to destressing phenomena in the parent rock in the close vicinity of the opening. This process continues with time and consequently, the micro-fissures already present in the rock, dilate gradually. It was observed at few locations of the project site that the rock faces which sounded solid by striking hammer just after taking blast, produced hollow and drummy sound after some time (Goel and Rajvanshi, 1978).

Because of the lack of precedence for this work in the year 1969, there was considerable interest in monitoring the post -construction behaviour of the underground powerhouse complex so that design assumptions could be checked against the results of the ongoing observations and timely action could be taken when problems would be indicated. Apart from the safety of the works, it was also felt that the data so obtained will help in assuring better economy and greater stability in the design and construction of future large underground powerhouse cavities in the soft rocks of the Lesser Himalaya.

5.2 Long-term Instruments

A net work of long term instruments (e.g. extensometers, rock bolt load cells, strain meters and pore water pressure cells) was installed in 1973 during construction in the powerhouse complex to check the safety and adequacy of the support system, to assist with the assessment of any required remedial measures and to monitor the long-term behaviour of the excavation. Figure 5 shows cross section and plan of the complex with instrumentation layout. Figure 6 shows the perspective view of the underground powerhouse complex.

5.2.1 Falling Hammer Type Extensometer

The purpose of this instrument was to measure the rock dilations near the surface of the excavation. Such dilations near the surface of an underground excavation are usually expected to occur particularly in soft rockmass conditions. These are produced as a result of release of internal stresses in the rockmass after creation of an opening.

A knife edge suspension arrangement is attached with a 4.8m deep (40mm diameter) grouted anchor bar, and an attachment of 6mm dia rod carrying a load (hammer) is suspended to keep the load in proper position. The gap between the edge at the time of installation was provided as 2mm. In case of any major deformations due to time dependent behaviour (creep) on the surface rock, there is provision in the instrument that the gap between the edges would increase and if the same exceeds 2mm, the hammer would fall down. This will act as a warning system also. Four such intsruments have been installed at four different locations on the upstream wall and four on the downstream wall of the powerhouse cavity.

5.2.2 Strain Meter

The steel-rib support system in the underground powerhouse cavity is subjected to rock pressures. As a result of redistribution of stresses in the surrounding rockmass, the steel-ribs undergo minute changes in the form strains along its length. Transducer type strain meters are used to measure such changes in the strains. The strain meters installed in the steel ribs are of the vibrating wire type since these have the best long-term performance. The observations are recorded through a remote read out system known as receiver. The strain meters were of the best quality available in the international market.

Twelve arrays of strain meters are installed in the lower flange of steel rib of the powerhouse cavity at two different locations, viz., chainage 11.0 m (on the top of control room) and 108.5 m (on the top of transformer hall) as shown in Fig. 7. Similarly, a set of 8 strain meters is fixed in one of the steel-ribs of surge tank roof towards the extreme end. In case of pressure shafts, three arrays of strain meters are installed in the steel liner of each penstock limb. The instruments are located at crown, bottom and the side of the penstock with respect to its cross-section.

5.2.3 Rock Bolt Load Cells

The rock bolt cells are usually 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 are used to measure these forces directly. The rock bolt load cells are mounted in pairs at three different location of the powerhouse cavity; two pairs on the upstream wall and another pair on the downstream wall.

5.2.4 Pore Water Pressure Cells

Three instruments are mounted around pressure shafts for the measurement of pore water pressure. The instrument consists of a filter, a diapharm, measuring wire, magnet, casing, over-voltage arrester, cable sealing cap and cable. The porous and water saturated filter separates the mechanical coil pressure from the hydraulic pore water pressure. The diapharm is deflected by pore water pressure (or seepage water pressure) which change the natural frequency of the measuring wire. The elector-magnetic system converts the mechanical vibrations of the wire into an electrical frequency which is transmitted to the receiver through a cable.

All these instruments such as strain meters, rock bolt load cell and pore water pressure cell do not show any direct reading as they are transmitter type. However, the cables from these instruments are laid up to the junction box, where one single instrument, known as Receiver is used for taking the readings received from transmitters. This is a portable set and is carried to the junction box only at the time of observation.

5.3 Other Useful Data

Regular records have also been kept of other important data, such as earthquake shocks, temperature of water and atmosphere, quantity of water utilized for power generation, water level in surge tank, rainfall etc. These data have proved to be very useful for the purpose of analysis and interpretation.

A seismological observatory is being maintained to record earthquake shocks in the area. This is situated at the left bank of the River Tons about 10 km from upstream of the power house complex and about 5 km of Ichari dam at Latitude 30^0 -37'50" north, longitude 77^0 -48' east with the elevation of 905 m. Some strong motion instruments like accelerograph and structural response recorder were installed in the year 1971. Subsequently other sensitive instruments like seismographs were installed in the year 1972. The main instruments installed in this observatory consisted of electromagnetic seismograph, wood anderson seismograph, accelerograph and structural response recorder. The regular earthquake records have been kept in the above observatory. The local shocks having epicientres within 110 km range have been used for the purpose of analysis and interpretation of data collected over seven year period.

The rainfall data as well as temperature of water and atmosphere at Chhibro have been collected daily whereas the quantity of water utilised for power generation have been recorded on monthly basis. Besides, the discharge in head race tunnel and water level in surge tank have also been recorded regularly.

6.0 ANALYSIS OF INSTRUMENTATION DATA

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.

- Terzaghi (1946)

6.1 Introduction

In view of the unique opportunity and great demand for long term data, instrument installation and recording of the data was done carefully. Moreover, this job is quite easy and automatic. The data of several instruments has been found to be consistent even upto 10 years in some cases. The real problem is that voluminous field data at several projects is seldom analysed carefully by the researchers. Often it is difficult to solve the whole puzzle well. If this is done in future, the great value of field data will be recognised soon inspite of scatter.

Ten volumes of data collected over 7 - 10 years period to study the long-term behaviour of powerhouse (Machine Hall) cavern and surge tank have been analysed. Special attention has been focussed on the factors contributing to the time-dependent deformations and an attempt has been made to identify them.

Since the supports were erected much earlier than the date of instrumentation and the transfer of short -term support pressure had already occured, it could not be possible to determine actual values of strains or stresses. However, the instruments showed relative variation in strains with time from the date of their installation. Instrumented data has been carefully checked and its reliability and sensitivity examined.

6.2 Roof of the Powerhouse Cavity

6.2.1 Observational Data

Typical pattern of variation in stresses on the steel ribs with time has been shown in Fig. 8 and Fig. 9 for two years only. The date of observation is shown along the horizontal axis, the vertical axis shows - (i) the observed stresses in N/mm^2 (and kg/cm^2) (ii) earthquake vibrations in terms of number of shocks ($M < 5$) occurring every month for epicentre within 110 km range, (iv) rainfall in mm,

(v) average quantity of water drawn for power generation every month in terms of discharge in cubic meter per second and (vi) average temperature of water in degree centigrade. The curves in above figures show 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. Despite a number of failures of the instrument, useful data have been obtained on the long -term behaviour of the excavation specifically with those instruments which have given consistent readings.

Similar pattern of variation in stresses, quantity of water utilized for power generation and rainfall with time are obtained in subsequent years also i.e., from 1979 to 1983.

6.2.2 Effect of Seepage

A net work of drainage galleries alround surge tank and drainage holes (spacing 3.5 m c/c) along the periphery of the roof of the cavern across the self-draining support system have been provided as shown in Fig. 10. Although the excessive seepage through limestone band towards crown of the powerhouse cavity is being drained out by such self draining support system, yet effect of seepage on the temporary development of roof support pressure has been noticed during peak period of power generation. However, the effect of seepage has been found to be more significant on the long term behaviour of the cavity and has been explained in a separate article under time-dependent behaviour. It is clear from Fig. 8 that the variation in stresses fairly match with the rainfall pattern. During monsoon, the entire rockmass gets charged due to rain which increase seepage through jointed rockmass and shear zones and, therefore, the support pressure also increases.

The effect of seepage has also been studied by pore pressure observations taken around each penstock. The pore water pressure around penstock number 1 (see Fig. 5) which is nearest to the underlying shear zone (25 m thick) gives maximum average pore pressure before charging of the water conductor system. The other set of instruments fixed around penstock 2,3 and 4, which are located relatively at a farther distance give gradually lower values. The observations are shown below.

Pore Pressure Observations at the Crown of each Penstock before and after charging of water conductor system

Instrument fixed at	Pore pressure in kg/cm^2 *Before charging*	*After charging*
Penstock No.1 *(near thick shear zones)*	0.103	0.219
Penstock No.2	0.094	0.395
Penstock No.3	0.086	0.171
Penstock No.4 *(relatively far from shear zone)*	0.076	0.210

The observations of pore water pressure as shown in above table appear to match with the prevailing hydro-geological conditions as it seems that significant amount of seepage takes place through underlying thick shear zone resulting in higher pore pressure in the neighbourhood.

It is also observed from above figures that the stress variation in steel ribs has a definite trend with the pattern of quantity of water drawn for power generation. It appears that the limestone bed with a number of minor shear zones (5 cm to 50 cm thick) dipping towards power house cavity provides avenues for seepage. Consequently, transfer of seepage from headrace tunnel near the surge tank to the roof of the downstream power house cavity appears to be activated during peak period of power generation. Therefore when higher quantity of water is drawn for power generation, more seepage takes place which affects the support pressures on the steel ribs (Mitra et.al. 1988a). The leakage phenomena of similar nature has been studied by Barton (1986) From data analysis, it has also been observed that the support pressure, in general, has also been found to increase after charging of the water conductor system.

From these observations, it may be concluded that the seepage caused by monsoon and upstream headrace tunnel/surge tank affects the roof support pressure in underground powerhouse cavity. Therefore, support pressure assessment should account for probable seepage if unfavourable hydro-geological conditions prevail.

6.2.3 Temperature Effect

There does not seem to be any effect of atmospheric temperature on the stress variation in steel ribs because the temperature in the underground powerhouse is more or less constant.

However, it is observed that the significant amount of seepage from surge tank takes place through jointed and sheared rockmass towards the downstream powerhouse cavity. It is observed that when the temperature of water decreases during winter, the viscosity of water shall increase significantly (almost twice the value in summer), which reduces seepage pressure during winter season and vice-versa. Fig. 9 shows rythmic variation in stresses which more or less match with the variation in water temperature pattern. Thus, the atmospheric temperaturė appears to affect the viscosity of seepage water and ultimately affect the stresses in steel rib support system.

6.2.4 Effect of Earthquake

As shown in the Fig. 8 the stresses in the steel rib support system do not increase with time as would be the case if strain energy was accumulated after earthquakes shocks. Thus, it may be inferred that variations due to individual earthquake do not affect the support pressure on steel rib support system in the underground opening. The effect could be instantaneous but the same has not been recorded.

6.2.5 Time-Dependent Behaviour

The time-dependent behaviour of an underground excavation in rock mainly depends upon the deformation due to intrinsic time-dependent properties of the rockmass, which continues for a long period before stabilizing. The phenomenon of time -dependent deformation is very complex, but an estimate can be made, if some simplifying assumptions are made.

The approximate estimation of support pressure due to long- term time-dependent deformation of the opening may be done on the basis of short-term measurement data by extrapolating them for engineering life time of the structure (generally 100 years).

Stress variation in steel ribs with log-time for each instrument at chainages 11.0 m and 108.5 m are shown in Fig. 11(a & b) and 12. One of the test sections, i.e., chainage 11.0 m is located at a distance of 40 m from underlying thick shear zone. The above location (chainage 11.0 m) falls in the control room of the powerhouse cavity. Another test section, i.e., 108.5 m is located far away from the shear zone. The distance of this test section from above shear zone is 140 m and falls in the transformer hall of the powerhouse cavity.

In above figures, time in terms of years are plotted in abscissa of the semi-log scale, whereas ordinate shows average stresses in kg/cm^2 observed in the steel rib support system. It may be seen in the above figures that roof support pressures have not stabilized and have increased almost logerithmically with time for over 10 years period. The increase in stresses at the crown due to time-dependent deformations at chainage 108.5 m, when extrapolated for 100 years is estimated to be approximately 530 kg/cm^2 which is slightly less than half the design stresses.

The time-dependent behaviour is found to be more significant in the neighbourhood of underlying thick shear zone as observed at chainage 11.0 m. The stresses observed at this section, when extrapolated for 100 years are found to be 675 kg/cm^2. This is 1.27 times the stresses observed at chainage 108.5 m which is at far distant place from the shear zone.

It is significant to note that the time-dependent effect has been noticed only where there is soluble and erodible nature of jointfillings in rocks with seepage problem, i.e., roof of the power house cavity and also near thick shear zone. However, no considerable time-dependent behaviour has been noticed where rockmass is dry, i.e., on the roof of surge tank and the walls of machine hall cavity. The cause of increse in support pressure appears to be leaching of joint fillings in the limestone due to seepage.

It also appears that the modulus of deformation of rockmass reduces due to saturation after commissioning of the project and results in outward movement of rockmass. The inrease in support pressure tries to counter balance this movement depending upon stiffness of the support system. So the effect of saturation should be considered properly in design of underground opening for river valley development projects particularyly in the rockmass whose modulus of deformation will decrease significantly after saturation.

6.3 Wall of the Powerhouse Cavity

6.3.1 Observational Data

Three test sections were chosen for installing the rock bolt load cells on the walls of 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 are 60 ton with spacing of 2 to 5 m in both di ections. These instruments were fixed at a height of about 16 m from the bottom of the cavity at chainage 18 m, 42 m and 57 m respectively.

The length of the cable anchors are of 20 to 30 m. The downstream wall anchors are approximately 1.4 times the length of upstream wall anchor. The rock loads induced on the cavern wall with time have been measured by these instruments.

Results obtained for over six year period have been shown in Fig. 13. It may be seen in the above figure that the variation in anchor load with time have intermittent formation of peaks. The cause of temporary increase in anchor load, i.e., formation of peaks has been studied. In above figure the earthquake magnitude and relatively higher rainfall for certain specific period have also been shown, during which temporary increase in anchor loads has been noticed, i.e., where peaks appeared.The number of earthquake shocks recorded from the date of start of observations and the date of formation of peaks have been shown in Fig. 14.

6.3.2 Effect of Seepage

In case of roof, the effect of seepage has been observed to be significant after charging of the water conductor system. However, this effect has been found to have little effect on the walls of the cavity. This is because the effective drainage system has been provided around surge tank opening. It appears that the effect of seepage from headrace tunnel and surge tank would be reduced to almost negligible along the walls of the cavity. However, slight increase in anchor loads has been noticed during heavy rains. During this period, deposition of calcium carbonate at few places has also been observed. It may also be seen in Fig. 13 that the wall support pressures increased significantly after charging of the water conductor system. This may be due to readjustment of stresses in the rockmass.

6.3.3 Effect of Earthquake

The earthquake data collected over six year period have been analysed to study its effect on the wall support pressure. It may be seen in Fig. 13 and 14 that anchor loads, i.e., wall pressure increases slightly during earthquake. The increase in static bolt tension is negligible and is found to be less than 2% for single small earthquake (Magnitude<5).

The events occuring within 110 - kms of the observatory located about 10 km from the power house complex have been considered for the analysis. The number of earthquakes were counted from the date of start of observation to the observed peaks of the anchor load at various time intervals. These numbers are shown in Fig. 14. Majority of the earthquakes considered for the analysis fall in the magnitude range of 2.5 to less than 5. As the recording device is wood Anderson type, it has limited detection capability and can only record earthquakes magnitude above 3. However, some events having magnitude below 3 could also be recorded, which is attributed to the occurence of events within 30 kms from the recording station.

Figures 15 and 16 depict the variation in anchor load with earthquake magnitude and epicentral distance respectively. While preparing this graph, the maximum magnitude earthquake occuring in a time interval from the onset of the peak to the maximum observed peak has been taken. From these figures it may be seen that the temporary increase in wall support pressure due to earthquake is found to be directly proportional to the magnitude of earthquake and inversely proportional to the epicentral distance as shown in Fig. 15 and 16. In the abscissa of the above two figures, the earthquake magnitude on Richter's scale and epicentral distance in terms Kilometres (0.1 represents 10 km) have been shown and in the ordinate, wall pressure in terms of induced load in kg on rock anchors has been shown. However these correlations may be improved further with more field data (Mitra and Singh, 1991).

In case of wall, the effect of earthquake shock appearse to persist for a longer duration (few days to few weeks). This may be because of the interaction of cable anchor with the surrounding rock mass during and after earthquake shocks. On the other hand the effect of earthquake shock on the steel rib support for roof appears to be instantaneous due to elastic and arching action of the steel ribs but the same has not been recored.

An important observation in the present study was that, the anchor loads in the wall of the cavity appear to increase with time due to effect of time-dependent behaviour of underlying thick shear zone and accumulated strains after earthquake shocks as shown in Fig. 14. This is revealed by a set of rock bolt load cells (instrument No. 3,4) located in the close vicinity of shear zone at chainage 18.0 m and 26.0 m vertically from above shear zone.
The other two sets of instruments located at chainage 42 m and 57 m respectively which are far from the underlying shear zone did not show time-dependent deformation or accumulation of strains in rockmass due to recurring earthquakes. However, the general trend in variation of wall support pressure

with time were found to be consistent for all instrument. Since the effect of recurring earthquakes near thick shear zone appears to be significant in the development of wall support pressure with time, a linear relationship between cummulative earthquake shocks and corresponding cummulative anchor loads was obtained as shown in Fig. 17. The correlation suggests that there may be some residual strains in rockmass after earthquake shocks near thick plastic shear zone and consequently the effect of recurring earthquake in seismic region would lead to significant time-dependent behaviour. This phenomena is revealed in Fig. 14 which shows gradual increase in support pressure with time. Such phenomenon may not be observed if shear zone is far away, i.e., approximately 1.5 times the span of the opening.

6.3.4 Time-Dependent Behaviour

It may be seen in the Fig. 13 that no significant time-dependent behaviour is observed except near thick shear zone. One of the reasons may be that the cavern wall is not under the direct influence of seepage from upstream headrace tunnel and surge tnak as in the case of roof of the power house cavity. The time-dependent behaviour has been noticed only where the rockmass is saturated due to seepage problem, i.e., roof of the cavern and near thick shear zone.

As already discussed in previous article that the effect of recurring earthquakes near thick shear zone appear to be a major factor contributing towards accumulation of strains in the rockmass which result-in gradual increase in wall support pressure. However, it is hard to predict the percentage contribution of effect of shear zone on the time-dependent behaviour.

6.4 Time-Dependent Behaviour of Surge Tank

Figure 18 shows variation in stresses on the steel ribs with time. It may be seen that no long-term variation in stresses on the steel rib is indicated as in the case of power house cavity. 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 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 surge tank cavity.

Similarly, the effect of earthquakes is also found to be negligible on the roof support pressures of surge tank. However, the observations taken for over seven year period indicated a periodic type of rock movement. Further, the effect of time-dependent rock deformations is also observed to be negligible.

The reason may be that the roof of the surge tank located for away from thick shear zone does not face any seepage problem after commissioning of the project as in case of machine hall (powerhouse) cavity. Thus the roof of the surge tank located in dry slate remains unaffected by the lime dependent deformations and earthquake shocks.

7.0 PERFORMANCE STUDY OF PRESSURE SHAFT

7.1 Observational Data

Four sets of three strain meters were installed in the steel liner of the penstock limb to monitor the performance of pressure shafts. The instruments recorded the induced strains in the liner and hence allowed the hoop stresses to be calculated. Despite a significant number of instrument failure, data obtained for three year period for one pressure shaft could be analysed.

Typical variations in liner stresses with time are shown in Fig. 19 for one year only i.e., for 1977-78. The curves in this figure show relative values of stresses in the steel liner. 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 among strain meters.

Simlar pattern of variation in stresses and water level in surge tank were obtained for subsequent years also i.e., 1978 to 1980.

7.2 Effect of Water Level in Surge Tank

It is clear from Fig. 19 that the liner stresses are affected by the water level in surge tank. Beyond the monsoon season, the water level of the reservoir/ surge tank is kept high and the discharge in the headrace tunnel/ pressure shaft is relatively low. Hence, the internal water pressure during this time is always greater than that in the monsoon period. Hence, it may be concluded that the stresses in the steel liner are significantly influenced by the water level in the surge tank as it increases internal water pressure directly in side the penstocks.

7.3 Effect of Earthquake

It may be seen from the above figure that the stresses in the liner do not increase with time as would be the case if strain energy was accumulated after earthquake shocks (Mitra and Singh,1989). This may be because the shaft is inherently more stable than high caverns (Terzaghi, 1943). Thus, it may be concluded that earthquake vibrations do not influence the stresses in the steel liner of the shafts.

8.0 SUPPORT PRESSURE

8.1 Support Pressure Assessment

The classification systems of Terzaghi (1942, 1946), Deere et al (1969), Barton et al (1974, 1975), and the analytical approach based on the underground wedge analysis by Hoek and Bray (1981) have been used to estimate the support pressures for the powerhouse cavern. The predicted support pressures from above approaches have been compared with the design and observed support pressures (Mitra & Singh 1995a). The geological details required for estimating the rockmass quality (Q) and input data for wedge analysis were collected from the exposed rock near the power house area.

8.2 Classification System of Terzaghi (1946)

In accordance with Terzaghi's rock load classification criteria, the rockmass condition was described as "Moderately blocky and seamy" (Category 4 of Terzaghi's table) for which the height of rock load is given as,

$$H = 0.25\ B \text{ to } 0.35\ (B + H_t)$$

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 may be ignored in the design. Today, the feeling of designers is that support pressure would be unrealistic if H is taken in to account (Singh et al, 1988). However, H was considered in the design of steel rib support system.

Thus, the short-term support pressure at the cavern roof works out to be

$$P_{roof} = 1.64 \text{ to } 2.30\ kg/cm^2$$

8.3 Classification System of Barton et al (1974, 1975)

Barton, Lien and Lunde (1974) developed the Q-system of rockmass classification in Norway, which is considered as a major contribution to the subject of rockmass classification for many reasons. The first and foremost reason was that the system was proposed on the basis of an analysis of some 200 tunnel case histories from Scandinavia. Further it is a quantitative classification system, and it is an engineering system of enabling the design of supports for tunnels and large chambers.

The long-term and the short- term support pressures for cavern roof and cavern wall are obtained as below :

Short-term support pressure

P_{roof} = 0.39 - 0.44 kg/cm² (cavern roof)

P_{wall} = 0.30 - 0.32 kg/cm² (cavern wall)

Long-term support pressure

P_{roof} = 0.71 - 0.76 kg/cm² (cavern roof)

P_{wall} = 0.52 - 0.56 kg/cm² (cavern wall)

8.4 Underground Wedge Analysis

In an underground excavation, the failure of rockmass may be attributed to the high insitu stress conditions and/or to the geological discontinuities forming wedges around the opening. The pre-knowledge of such possible failures during excavation is essential for which analysis are required to be carried out before the actual excavation.

Finite element and/or Boundary element methods are generally used to study the stress-induced failure while graphical or analytical methods of wedge analysis are employed to study the failure mechanism of rock due to formation of unstable wedge at the roof or at the wall of an excavation.

The support pressures evaluated from Hoek & Bray (1981) wedge analysis are given in Table 1 for dry and water charged conditions.

Table 1

Roof Support pressure from underground wedge analysis (Hoek and Bray 1981) for different seepage water pressures (from Mitra, 1991)

Sl. No.	Wedge	Support pressure when p =0 kg/cm²	when p =1 kg/cm²	when p =2.5kg/cm²
1.	123	0.19	0.77	1.69
2.	124	0.17	1.17	2.67
3.	125	0.12	1.12	2.62
4.	134	0.42	1.42	2.92
5.	135	0.24	1.24	2.74
6.	145	0.20	0.46	0.94
7.	234	0.43	0.77	1.49
8.	235	-	-	-
9.	245	0.32	0.62	1.16
10.	345	0.41	1.02	2.05
	Average	0.28 kg/cm²	0.95 kg/cm²	2.03 kg/cm²

8.5 *A New Hypothesis on the Mechanism of Dynamic Support Pressure*

An empirical support pressure theory has been developed to account for accumulated strains in rockmass due to recurring earthquake in seismic region. Barton's support pressure theory (1984) is considered as a basis or the development of proposed hypothesis on dynamic rock pressure. The empirical equation relating seismic support pressure near thick shear zone due to recurring earthquakes with rockmass quality of the cavern wall (Q wall) is found to be as follows :

$$\Delta Peq = \frac{2 \times 10^{-4}}{Jr} \cdot Neq\,(Q_{wall})^{-1/3} \qquad \textbf{(Mitra, 1991)}$$

where,

ΔPeq	=	Seismic support pressure due to recurring earthquakes near thick shear zone in kg/cm²
Jr	=	Joint roughness number
Q wall	=	Rockmass quality in wall (2.5Q)
Neq	=	Total number of earthquakes (2<M<5) within 100 km range within the life time of an opening.

Accordingly, an approximate estimate of increase in support pressure due to recurring earthquakes after 100 years is estimated to be of the order of 0.38 kg/cm^2 for wall and 0.52 kg/cm^2 for roof.The above equation does appear to confirm that dynamic increment of support pressure (0.52 kg/cm^2) due to earthquake is likely to be small compared to long-term support pressure (2.47 kg/cm^2). It may be noted that the proposed estimate is nearly 25 percent of long-term support pressure as suggested by Barton (1984).

However, above equation does not account for predominant period of shocks, which is found to have some effect on seismic support pressure (Okamoto et.al 1973). It also does not account for increase in seepage through opened fractures. However, the same could be considered by a reduced value of Jw in proposed equation on the basis of past case historries.

Table 2 and 3 gives the observed support pressure in the cavern roof and the ratio of long-term to short-term support pressures for observed period of 8 to 10 years and 100 years after extrapolation. 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. The support pressures from various methods are summarized in Table 3.

9.0 POST EARTHQUAKE OBSERVATIONS

An earthquake of 6.3 magnitude occured about 100 kms. from powerhouse area on Oct. 21, 1991 on account of which minor cracks developed at different locations of the complex (Mitra & Singh, 1995b). This appears to confirm the proposed theory on dynamic support pressure near thick plastic shear zone. The minor cracks developed in the complex are outlined as follows :

a) Out of 8 extensometers installed on the sidewailils of the cavity, two extensometers on the d/s wall adjacent to control room (near underlying shearzone) showed significant rock deformations which are of the order of 1 to 4 mm. Besides, a deep crack (2 to 4mm wide) formed diagonally up to a length of 3.5 metre between these two extensometers.

b) Horizontal hair cracks at a height of 0.5 to 2.5 metre are observed on each column of the control room and d/s side wall.

c) In the portal at the main entrance of the powerhouse adit, two horizontal cracks are observed, the length of which are 5 metre and 4.5 metre having thickness of 1 mm. Inside the adit, two vertical cracks at an interal of 80 metre are observed at a height of 1 metre having thickness of 0.5 mm and 4 mm. These cracks appear to have formed in shotcrete.

d) Few anchor plates in expansion chamber adit, which have been used to support prestressed rock anchors appeared to have stretched slightly during earthquake. This has resulted in formation of cracks in the shotcrete.

The computer program (UWEDGE) inicated a large number of critical wedges on the d/s side wall of the powerhouse cavity, which explains more anchor load observed on the d/s wall than the u/s wall of the cavern. The observed cracks due to 1991 earthquake are also on the downstream wall and near thick shear zone.

10. CONCLUSION

There is some time-lag between the development of an analytical theory and its verification through field measurements. The objective of the present study was to procure the wealth of field data and analyse it in detail so that field data may be used in the verification of theories and development of new concepts. Without suitable verification through field measurements, the value of any theory is limited. Moreover, necessary modifications may also be made in the existing theories in view of the observed effect of several factors on the long - term behaviour of a power-house cavern. The results of the present study may be more useful in view of safety consideration rather than optimization.

This study is an attempt to bridge the gap between purely theoretical and empirical approach. The analysis of field data has led to the following conclusions :

a) The reinforced rockmass around underground opening is found to behave as continuum except in the water charged region and in the neighbourhood of thick shear zone.

b) The support pressure in the powerhouse cavity has been observed to increase temporarily after charging of the water conductor system and during heavy rainfall periods. Further, the stresses in the steel rib supports are found to vary with the quantity of water drawn for power generation. Therefore, the support pressure assessment should account for seepage effect also where adverse hydrogeological conditions may prevail in future.

c) Thus, the assessment of support pressure for powerhouse cavern from Barton's theory has been found inadequate in view of the observed effect of hydrogeological conditions.Barton's Q system does not take into account the soluble/erodible nature of joint fillings in 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 pressures at a later date.

d) The support pressures on steel ribs in roof of the power- house cavity, do not appear to be affected considerably by earthquake vibrations. The effect could be instantaneous but the same has not been recorded. However, the wall support pressure has been observed to increase temporarily during rainy seasons and during earthquakes. The percentage increase in static anchor loads due to a single small earthquake (Magnitude ≤ 5) is less than 2%.

e) There may be some residual strains in the rockmass due to the effect of a nearby thick shear zone. The wall support pressure near the shear zone seems to increase significantly with time due to strains accumulated after each earthquake shock. This problem may not occur if shear zone is far away, i.e., 1.5 times the span of the opening.

f) A new hypothesis has been given on the mechanism of seismic support pressure if the cavern is located near a thick shear zone. Consequently, an empirical support pressure theory (Article 8.5) which is an extension of Barton's (1984) theory, has been proposed to account for the accumulated strains in the rockmass due to recurring earthquakes in a seismic region. The proposed theory takes into account Barton's rockmass quality (Q), joint roughness number (Jr) and number of small earthquake shocks (Neq) with epicentres within 110 km range.

g) *No significant time-dependent behaviour has been noticed where rockmass is dry, i.e., on the roof of surge tank (located in slates) and the walls of powerhouse cavity. Time-dependent effect has been noticed only where there is seepage problem, i.e., roof of thepowerhouse cavity and also near thick shear zone. The analysis of data further suggests that the ratio between long-term and short-term support pressure may not be a constant equal to 1.7 as commonly assumed but would vary from nearly 1 to 6 depending seepage conditions and soluble/erodible nature of joint fillings.*

h) The long term effect of seepage/saturation should be considered properly in design of underground openings for river valley development projects particularly in the rockmass whose modulus of deformation will reduce considerably after post construction saturation.

i) Underground wedge analysis by Hoek and Bray (1980) may give a realistic range of support pressures when the seepage water pressures after commissioning of the oproject are taken in to account according to joint water reduction factor in Q-system.

j) The observed stresses on the roof arch of surge tank have indicated a periodic type of rock movement. There is no increase in support pressure with time in this case for 8 years.

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

l) A careful study of monitored data suggests that the powerhouse complex would remain stable for its life time i.e. about 100 years. as :

   i) the effect of time - dependent rock deformations on support pressures is within safe limit of steel ribs/cable anchors.

   ii) major plastic shear zone (25m thick) will not enhance the effect of earthquake vibrations on support system considerably, and

   iii) the seepage through limestone band towards crown of the powerhouse cavity is being drained out effectively through a built-in drainage system provided in between the steel rib supports at a regular interval. However it may be suggested that the Almunium sheets or PVC sheets may be used inisted of G.P. sheets in built in drainage support system to avoid rusting and corrosion.

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12. REFERENCES

Agrawal, P.N. (1970), "Evaluation of Earthquake Parameters in Koyna and Dakpathar Regions", Ph.D. Thesis, Deptt. of Geology and Geophysics, University of Roorkee, Roorkee, India.

Arya, A.S., Agrawal, P.N. and Srivastava, L.S. (1978), "Seismic Instrumentation for the Water Resources Deveopment Projects in Ganga and Yamuna Valley", All India Symp. on the Economic and Civil Engg. Aspects of Hydro-Electric Schemes, University of Roorkee, Vol. II, PP.VIII-33-35.

Auden, J.B. (1934), "Geology of the Krol Belt", Rec. Geol. Surv. of India, Vol. 76, Pt. IV.

Barton, N. (1984), "Effects of RockMass Deformation on Tunnel Performance in Seismic Regions", Tunnel Technology and Subsurface Use, Vol. 4, No. 3, pp. 89-99.

Barton, N.R. (1986), "Deformation Phenomena in Jointed Rock", Geotechnique, Vol. 36, No.2, pp. 147-167.

Barton, N., Lien, R. and Lunde, J. (1974), "Engineering Classification of Rock Masses for the Design of Tunnel Supports", Rock Mechanics, Vol. 6, No.4, pp 183-236.

Barton, N., Lien, R, and Lunde, J. (1975), "Estimation of Support Requirements for Underground Excavations", Proc. Sixteenth Symp. on Rock Mechanics, University of Minnesota, Minneapolis, U.S.A., pp 163-177.

Bieniawski, Z.T. (1984), "Rock Mechanics Design in Mining and Tunnelling", A.A. Balkema, Rotterdam, Boston.

Deere, D.U., Peck, R.B., Monsees, J.E. and Schmidt, B. (1969), "Design of Tunnel Liners and Support System", Highway Research Record No. 339, U.S. Department of Transportation, Washington, D.C.

Dutro, H.B. and Perry, D.J.(1987), "Measuring up for Tunnel Construction", Tunnels and Tunneling, June, 87 pp. 47-51.

Goyal, K.G. and Rajvanshi, U.S. (1978), "Instrumentation on Works of Yamuna Hydel Scheme Stage-II, Part-I and their performance and results", Proc. All India Symp. on the Economic and Civil Engineering Aspects of Hydroelectric Scheme, Vol. II, pp. VIII-65-74.

Gupta, J.P., Jain, V.K., Singhal, M. Mande Badarinath, H.S. (1985), "Chhibro Underground Powerhouse-roof and wall supporting Arrangement", Third National Symp. Rock Mechanics, Roorkee, pp. II-92-104.

Hoek, E. and Bray, J. (1981), "Rock Slope Engineering", The Institution of Mining and Metallurgy, London.

Jethwa, J.L., (1981), "Evaluation of Rock Pressures in Tunnels Through Squeezing Ground in Lower Himalayas", Ph.D. Thesis, Deptt. of Civil Engg., University of Roorkee, Roorkee.

Mitra, S., Singh, B. and Rajvanshi, U.S. (1988a), "Performance of Chhibro Underground Powerhouse Cavities after Charging Water Conductor System - A Case Study", Proc. Int. Symp. Underground Engg., New Delhi, Vol. 1, pp. 425 - 430.

Mitra, S., Singh, B. and Rajvanshi, U.S. (1988b), "Performance of Chhibro Underground Powerhouse Cavities after Charging Water Conductor System - A Case Study", Proc. Int. Symp. Underground Engg., New Delhi, Vol. 2, pp. 33 - 34.

Mitra, S. and Singh, B. (1989), "Performance Observations on the Pressure Shaft for the Chhibro Underground Power House Complex", Proc. Shaft Engineering Conference, Harrogate, U.K. pp. 317-323.

Mitra, S. and Singh, B. (1991), "Long - term Observations on Underground Power House Opening in Complex Geology", Journal of Engg. Geology, (India) Vol. XX, No.1 to 4 pp. 267-272.

Mitra, S. (1991), "Study on Long-term Behaviour of Underground Powerhouse Cavities in Soft Rocks", Ph. D. thesis, University of Roorkee, India : 194.

Mitra, S. & Singh, B. (1992), "Performance Study of a Large Cavern for Chhibro Underground Powerhouse Complex", Proc. of the International Symposium on Rock Support, Sudbury, Ontario Canada, : 237-244.

Mitra, S. & Singh, B. (1995a), "Support Pressures in Powerhouse Cavern in Weak Rocks - A Case Study", Rock Mechanics Proc., 35th U.S. Symposium on Rock Machanics, University of Nevada, Reno, U.S.A. : 437-443.

Mitra, S. & Singh, B. (1995b), "Long-term behaviour of a large cavern in seismically active region of lesser Himalaya", Proc. 8th Interrnational Congress on Rock Machanics, Tokyo, Japan.

Okamoto, S. Tamura, C., Kato, K. and Hamada, M. (1973), "Behaviour of Submerged Tunnels during Earthquakes", Proc. Fifth World Conference on Earthquake Engineering, Rome Vol. 1, pp. 544 - 553.

Terzaghi, K. (1943), "Theoretical Soil Mechanics", John Wiley and Sons, Inc., New York, pp. 202-215.

Terzaghi, K. (1946), "Rock Defects and Load on Tunnel ıpports", Introduction to Rock Tunnelling with Steel Supports by Proctor, R.V. and White, T.L., Commercial Shearing and Stamping Co., Youngstown, Ohio, U.S.A.

Table 2

Observed support pressures in Roof of the Power House Cavity (from Mitra, 1991)

Location	'Observed stresses' in steel rib (kg/cm^2)	Equivalent increase in support pressure (observed), (kg/cm^2)	Barton's short term support pressure (reference), (kg/cm^2)	'Total support pressure (kg/cm^2)	Stresses extrapolated for 100 years (kg/cm^2)
Ch. 108.5 m (far away from thick shear zone)	250 *(after 8 years)*	0.96	0.42	1.38	530
Ch. 11.0 m (near thick shear zone)	350 *(after 10 years)*	1.35	0.42	1.77	675

Table 3

Ratio of short term to long term support pressures in power house cavity (from Mitra, 1991)

Location	Observed support pressure (kg/cm^2)	Long - term support pressure extra-polated for 100 yrs. (kg/cm^2)	Observed support pressure / Barton's short-term support pressure support pressure	Long - term support pressure / Barton's short-term
Ch. 108.5 m (far away from thick shear zone)	1.38	2.47	3.28 *(after 8 years)*	5.9 *(extrapolated for 100 years)*
Ch. 11.0 m (near thick shear zone)	1.77	3.03	4.20 *(after 10 years)*	7.2 *(extrapolated for 100 years)*

Table 4

Comparison of Support Pressures for the Chhibro Powerhouse Cavern (from Mitra, 1991)

Rock Mass Description	Predicted support pressure in (kg/cm²) Classification system						Underground wedge Analysis (kg/cm²)		Observed support pressure extrapolated for 100 years (kg/cm²)		Adopted support pressure (kg/cm²)	
	Terzaghi	Deere et al.	Barton et al. Short-term		Barton et al. Long term							
	Pvo	Pvo	Pvo	Pho	P_{vult}	P_{hult}	Pvo	Pho	Pvo	Pho	Pvo	Pho
Dolomitic lime-stone interlayered with slates, γ = 2.74 gm/cc, RQD = 50-60% Q = 1.03 - 1.23 Cj = 1 t/m², ϕj = 30° Width of cavern 18.20 m height of cavern 32.5 m Rise/span ratio : 0.27	1.64 to 2.30 (1.97)	2.30	0.39 to 0.44 (0.42)	0.30 to 0.32 (0.31)	0.71 to 0.76 (0.73)	0.52 to 0.56 (0.54)		0.28 (dry condion) 0.95-2.03 (water charged)	2.47 and 3.03 (near thick shear zone)	0 0.38	3.8	0.87

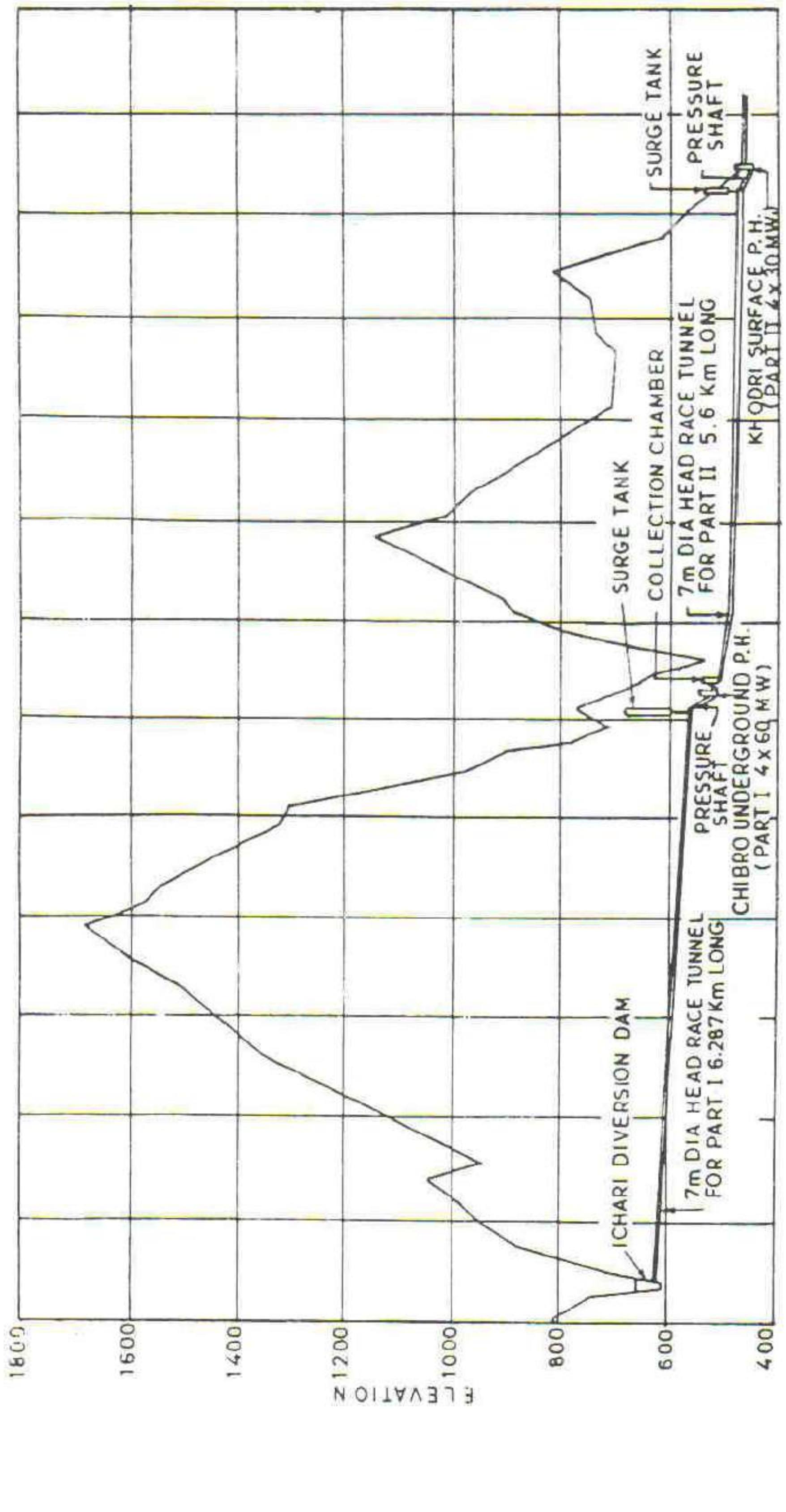


FIG. 1 - SCHEMATIC PROFILE OF YAMUNA HYDEL SCHEME, STAGE-II

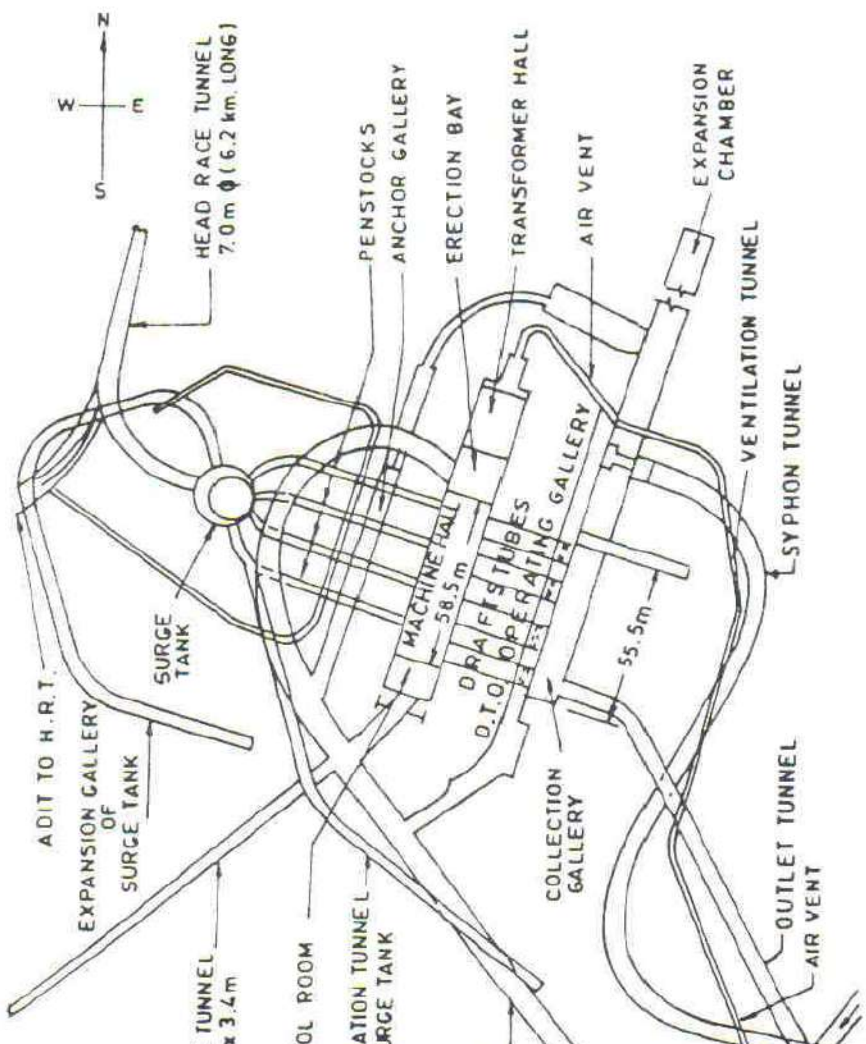
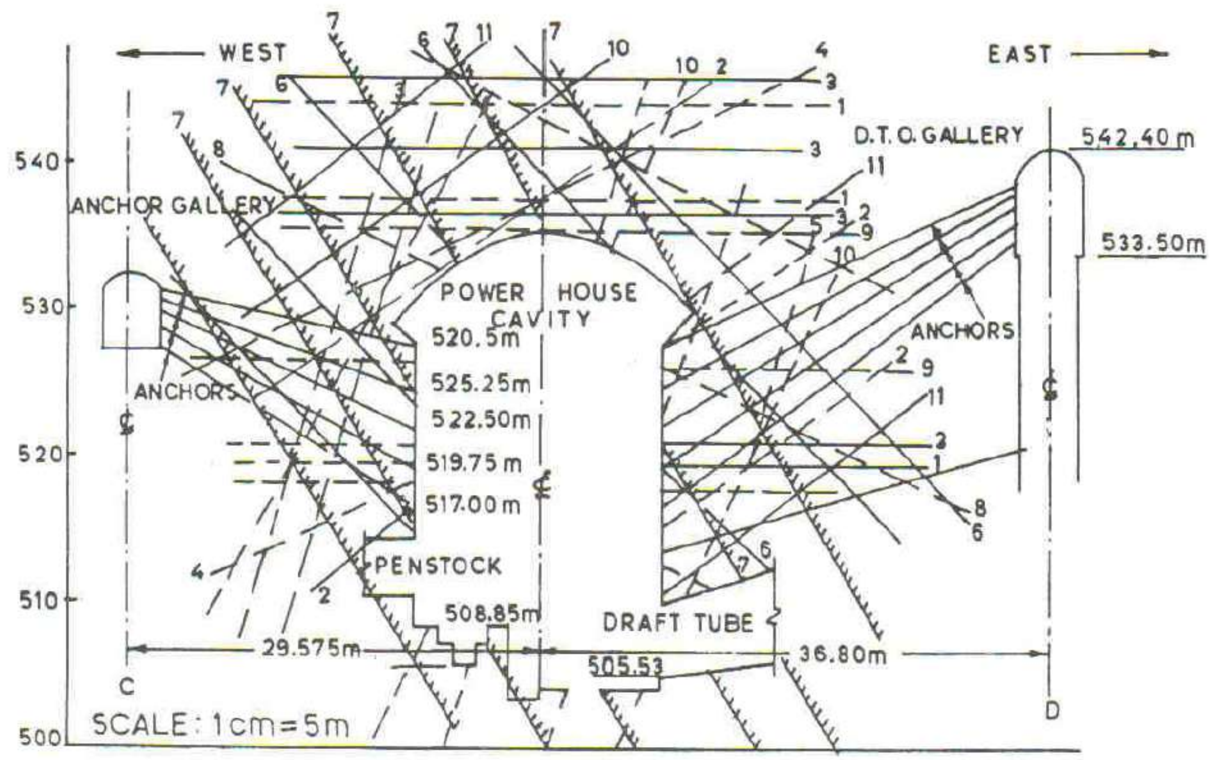


FIG. 2 - GENERAL LAYOUT OF CHHIBRO UNDERGROUND POWER HOUSE COMPLEX



THE PROMINENT SHEAR ZONES, SLIP PLANES & JOINTS

SL NO	AMOUNT OF DIP	DIRECTION OF DIP	REMARKS
1	35° - 50°	N 10° W TO N 15° E	JOINTS PARALLEL TO BEDDING @ 2-10 cm INTERVAL
2	30° - 86°	N 30° W TO N 50° E	5-10 cm THICK PLASTIC SEAMS SLIP PLANES & JOINTS CONTINUITY MORE THAN 3m
3	5° - 15°	DUE N TO N 10° E	SLIP PLANES CONTINUTY OBSERVED MORE THAN 3m
4	20° - 30°	DUE WEST	JOINTS
5	40° - 88°	N 70° W TO N 80° W	JOINTS
6	35° - 65°	N 50° E TO N 70° W	5-10 cm THICK PLASTIC SEAMS SLIP PLANES & JOINTS CONTINUTY MORE THAN 3 m
7	40° - 75°	S 70° E TO DUE E	PROMINENT SHEAR ZONE 10 cm TO 50 cm THICK WITH GOUGE
8	30° - 53°	S 25° E TO S 40° E	JOINTS
9	35° - 60°	DUE S TO S 10° W	JOINTS
10	45° - 80°	S 70° W TO S 80° W	JOINTS
11	50° - 65°	S 20° W TO S 40° W	5-10 cm THICK SHEAR ZONE SLIP PLANES & JOINTS

INDEX:

PROMINENT SHEAR ZONE

5-10 cm THICK PLASTIC SEAMS, SLIP PLANE AND JOINTS

— — — JOINTS & OCCASIONAL SLIP PLANES OBSERVED USUALLY FOR SHORT DISTANCES (LESS THAN 1 m)

REMARKS

1 THE OVER ALL PATTERN OF SHEAR ZONE, JOINTS & SLIP PLANES IS ILLUSTRATED IN THE DRAWING

2 THE SPACING & CONTINUITY OF INDIVIDUAL JOINTS IS ILLUSTRATIVE

.3 - GEOLOGICAL SECTION ALONG SHORT AXIS OF CHHIBRO POWER HOUSE SHOWING ANCHORS AND STRUCTURAL DISCONTINUITIES

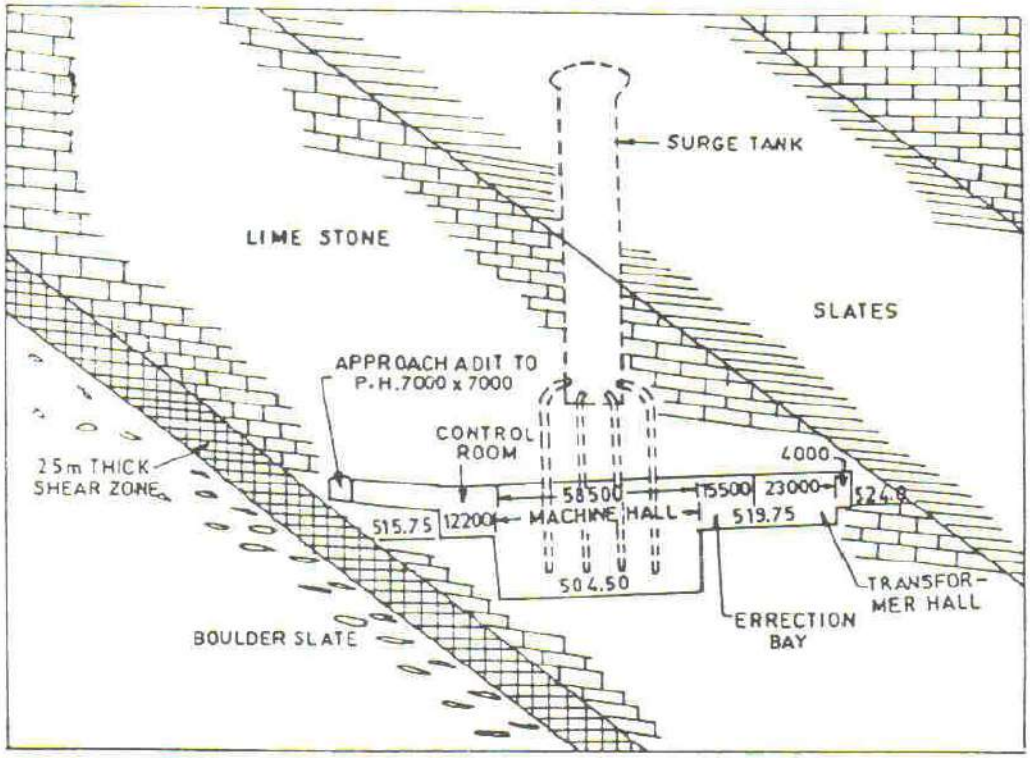


FIG. 4 - GEOLOGICAL CROSS SECTION ALONG LONG AXIS OF POWER HOUSE COMPLEX SHOWING SURGE TANK AND PRESSURE SHAFT

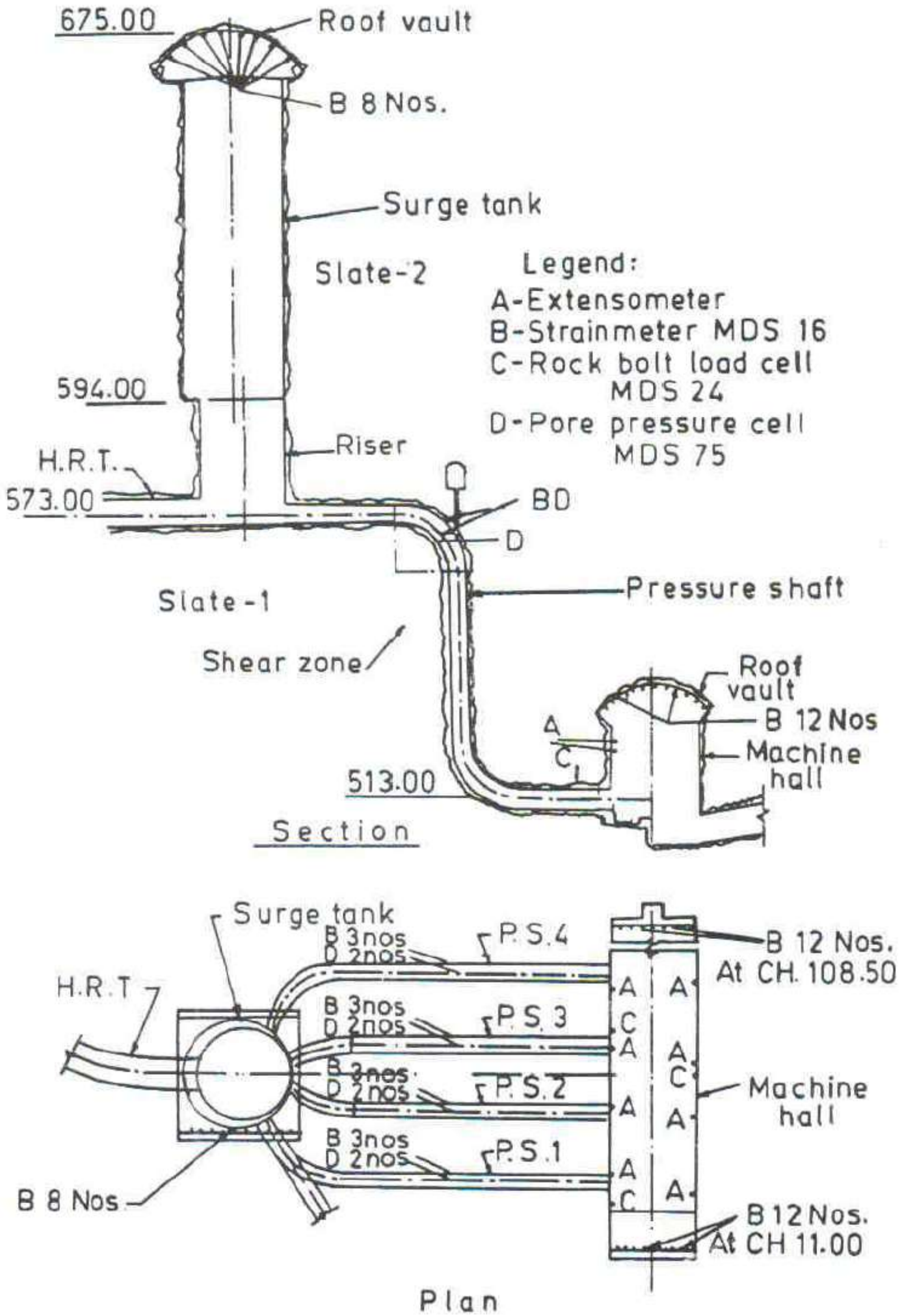
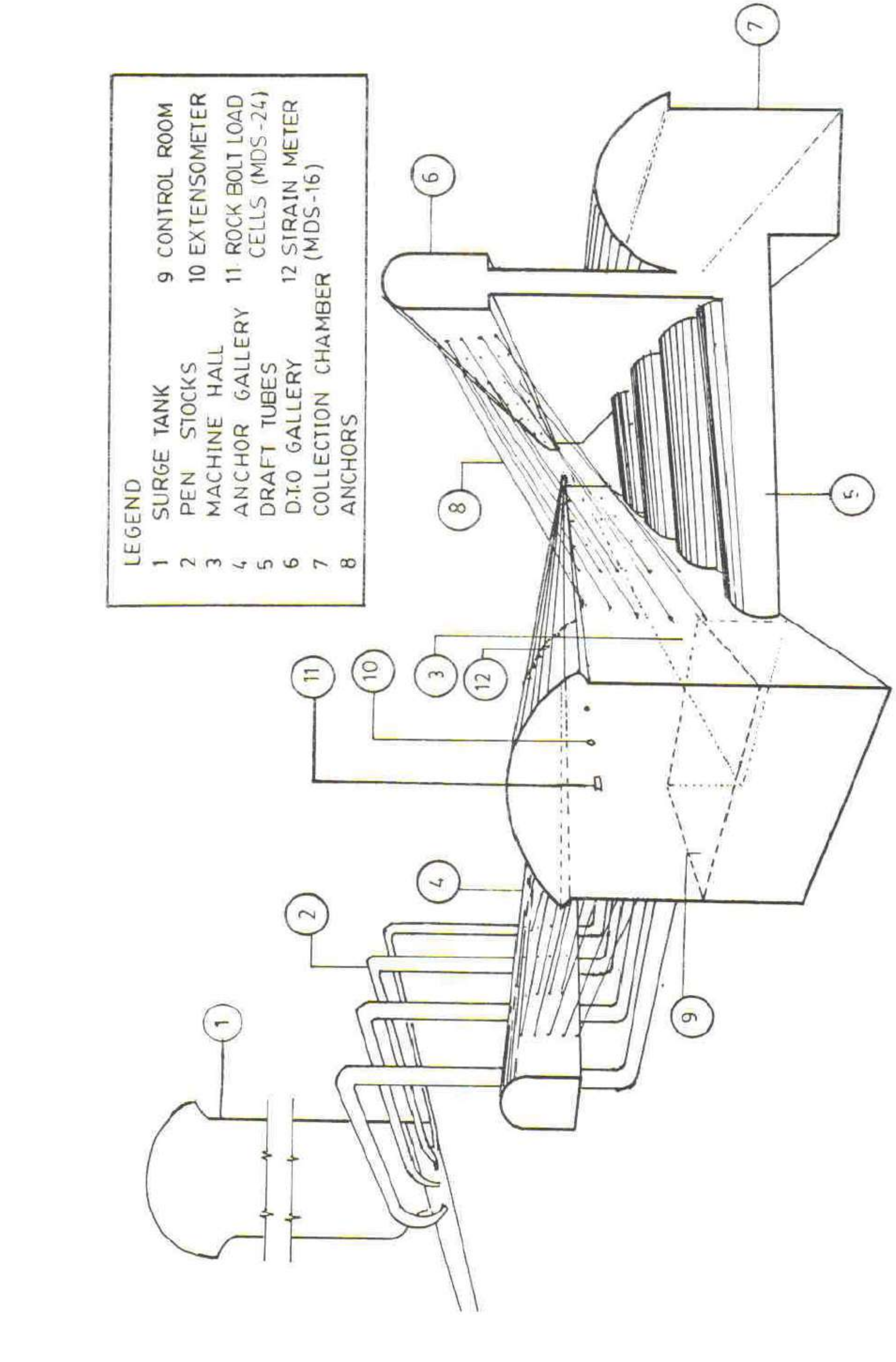


FIG. 5 – CROSS SECTION AND PLAN OF THE UNDERGROUND POWER HOUSE COMPLEX WITH INSTRUMENTATION LAYOUT



PERSPECTIVE VIEW

FIG. 6 CHIBRO UNDERGROUND POWER HOUSE COMPLEX

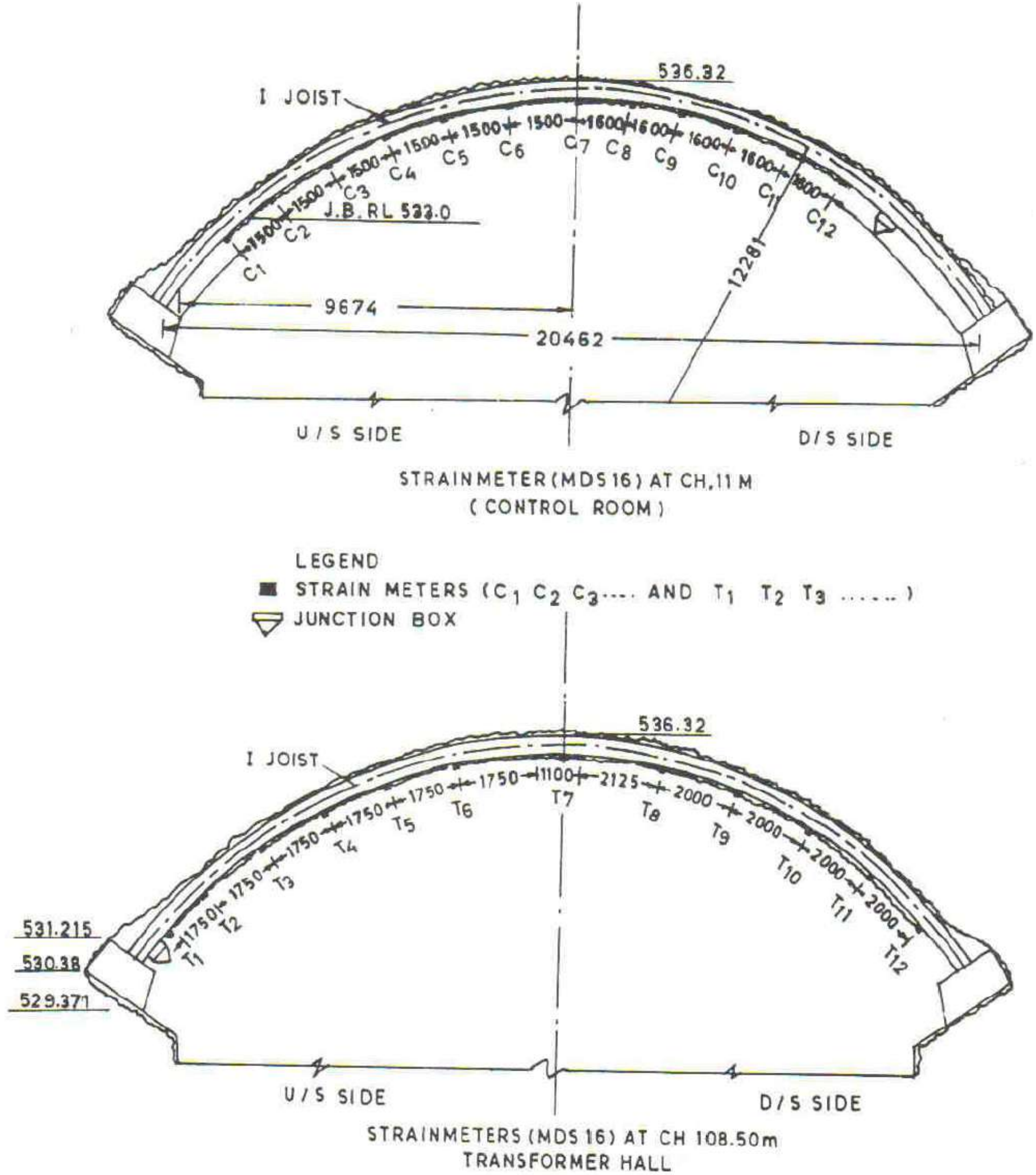


FIG. 7 - INSTALLATION DETAILS OF STRAIN METERES IN THE STEEL RIBS OF POWER HOUSE CAVITY

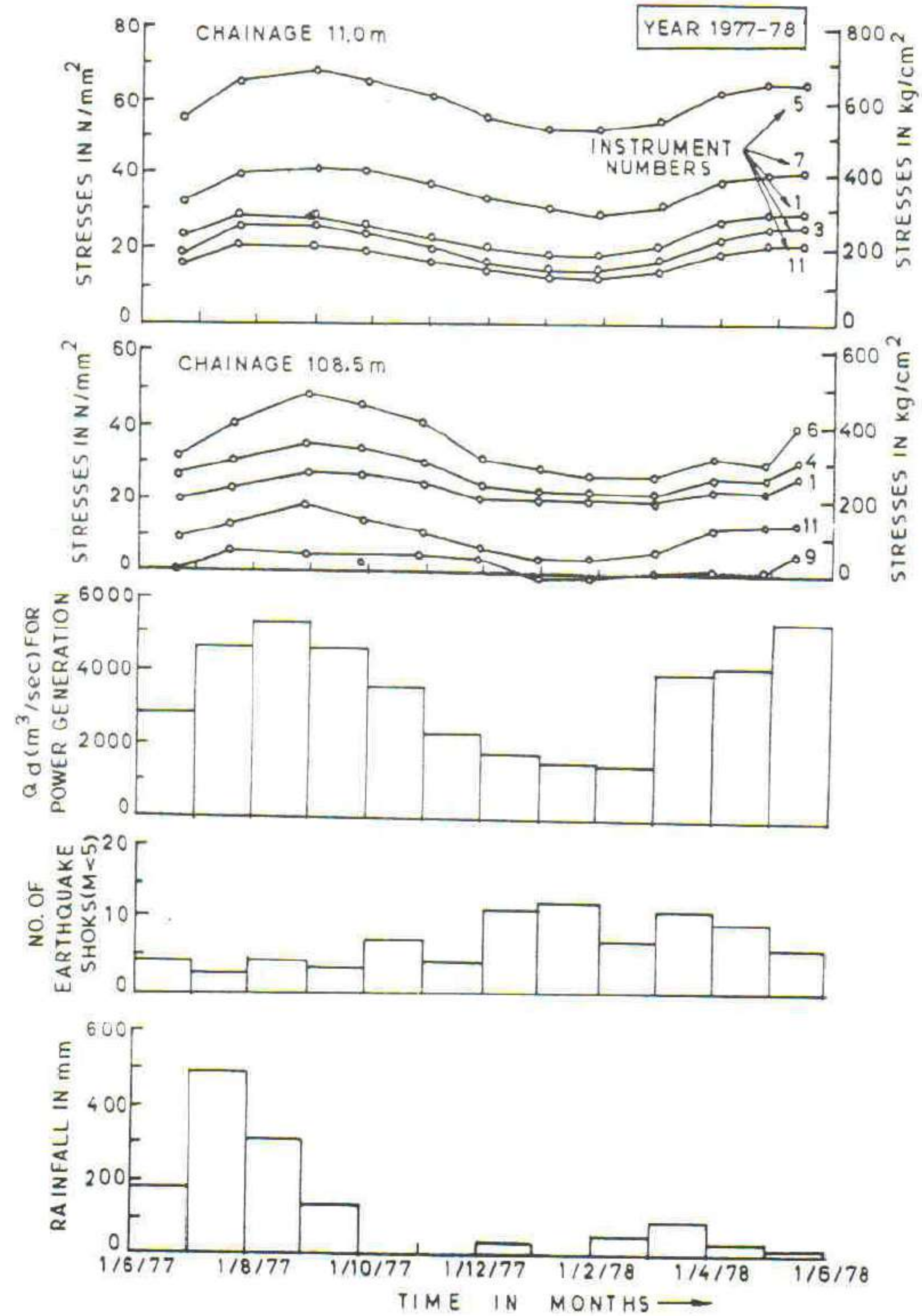


FIG. 8 -VARIATION OF STRESSES IN STEEL RIB SUPPORT, QUANTITY (Q_d) OF WATER DRAWN FOR POWER GENERATION, EARTHQUAKE SHOCKS WITHIN 110 km RANGE AND RAINFALL WITH TIME

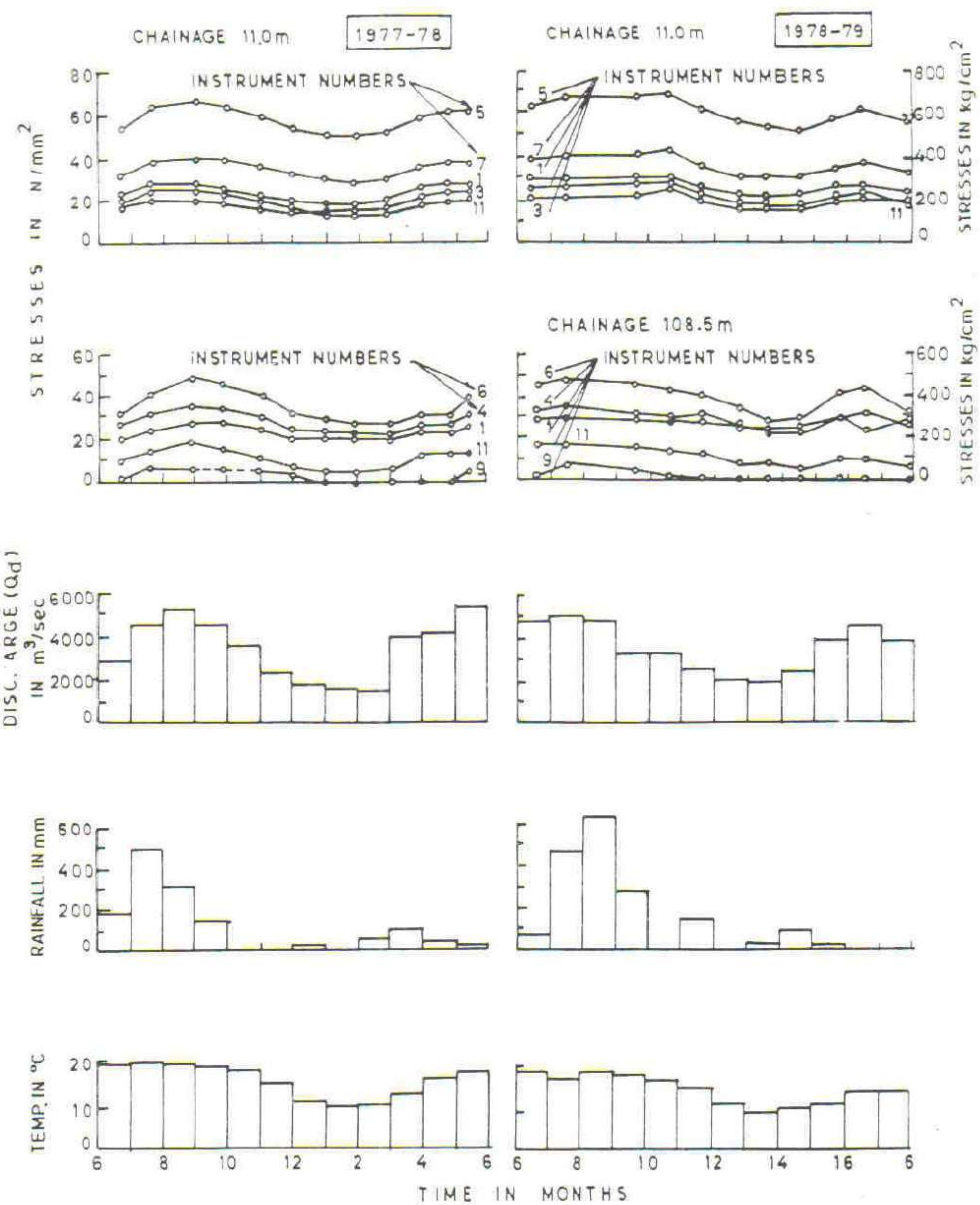


FIG. 9 – VARIATION OF STRESSES IN RIB, QUANTITY (Q_d) OF WATER DRAWN FOR POWER GENERATION, RAINFALL AND TEMPERATURE OF WATER IN °C WITH TIME

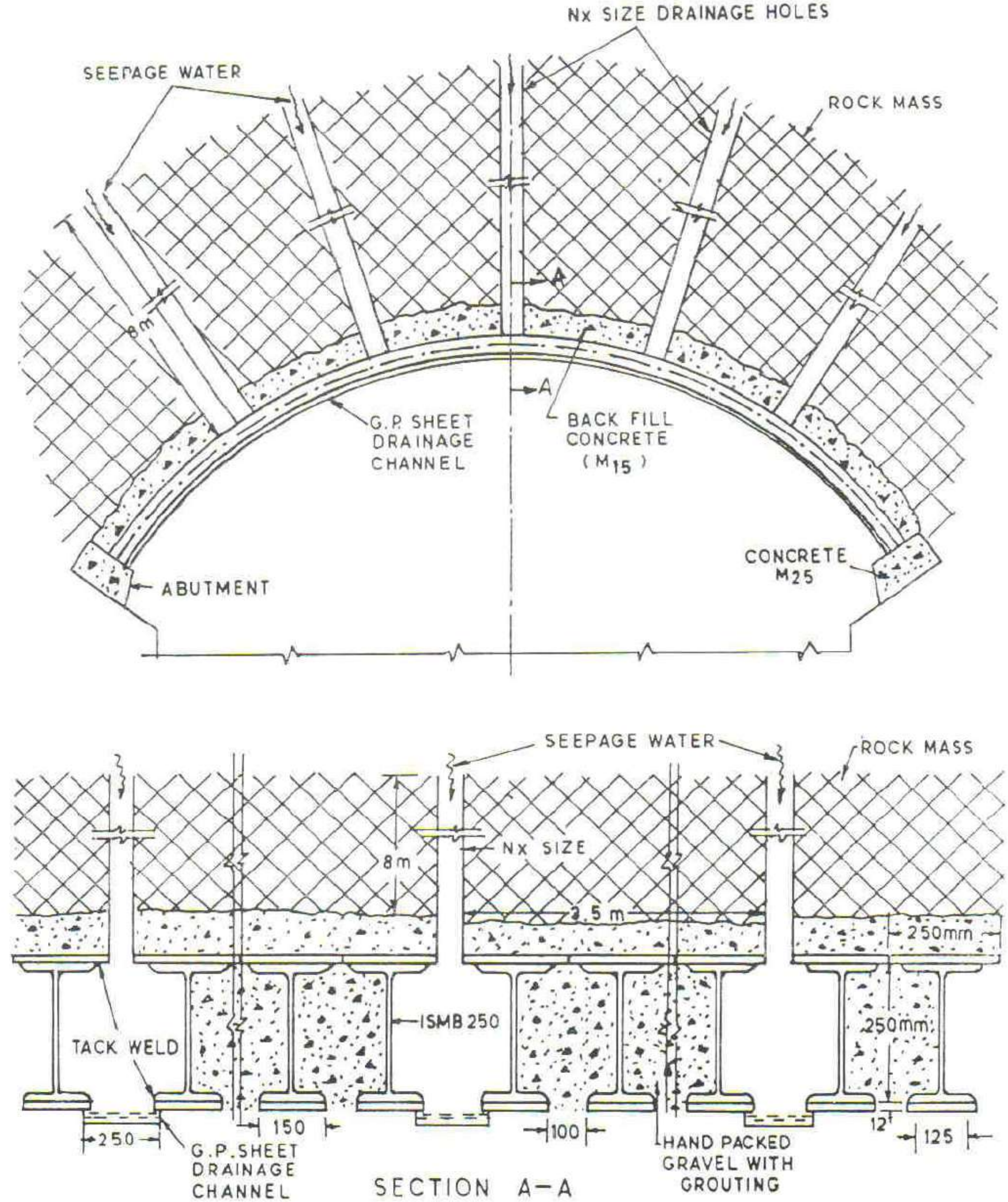
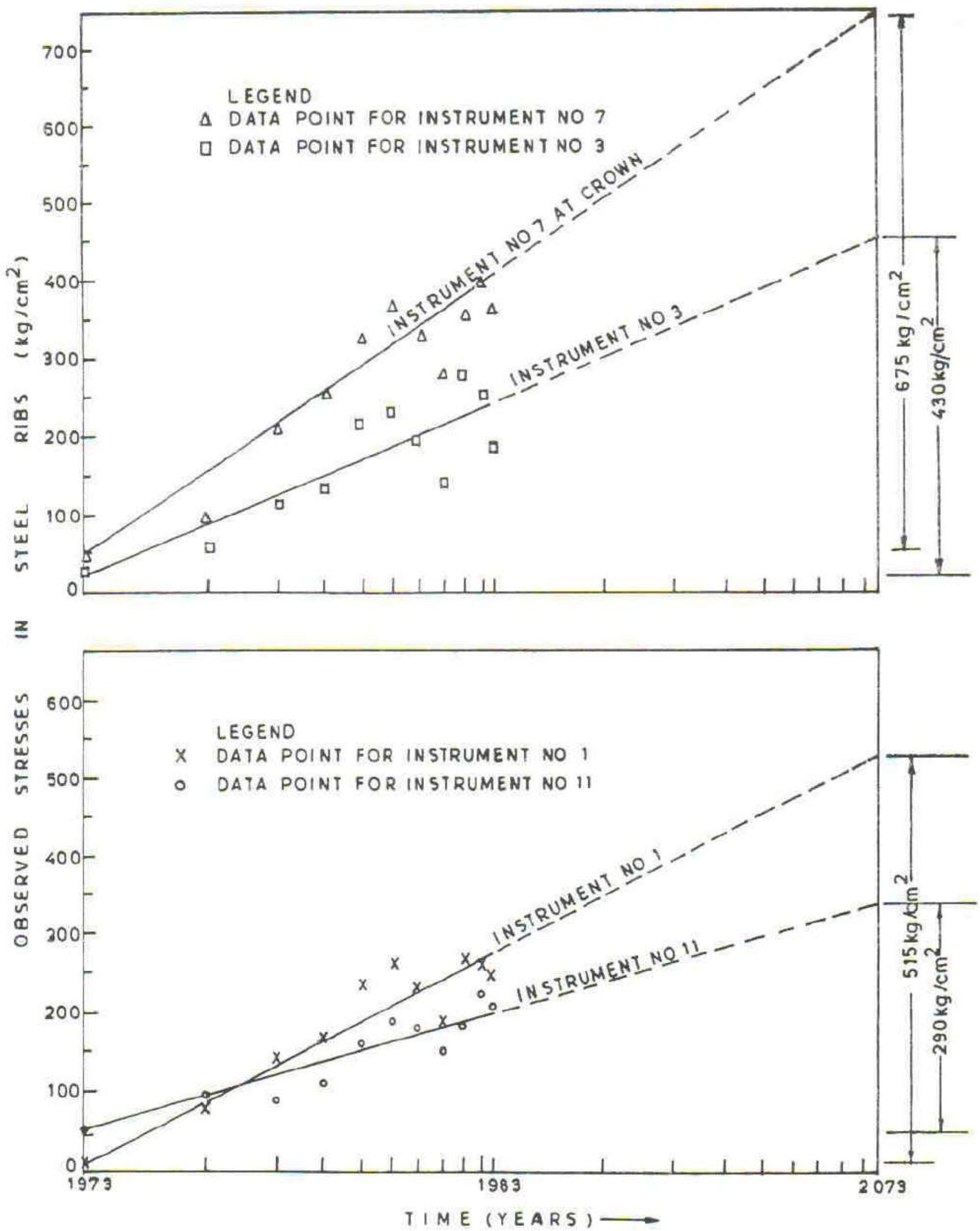
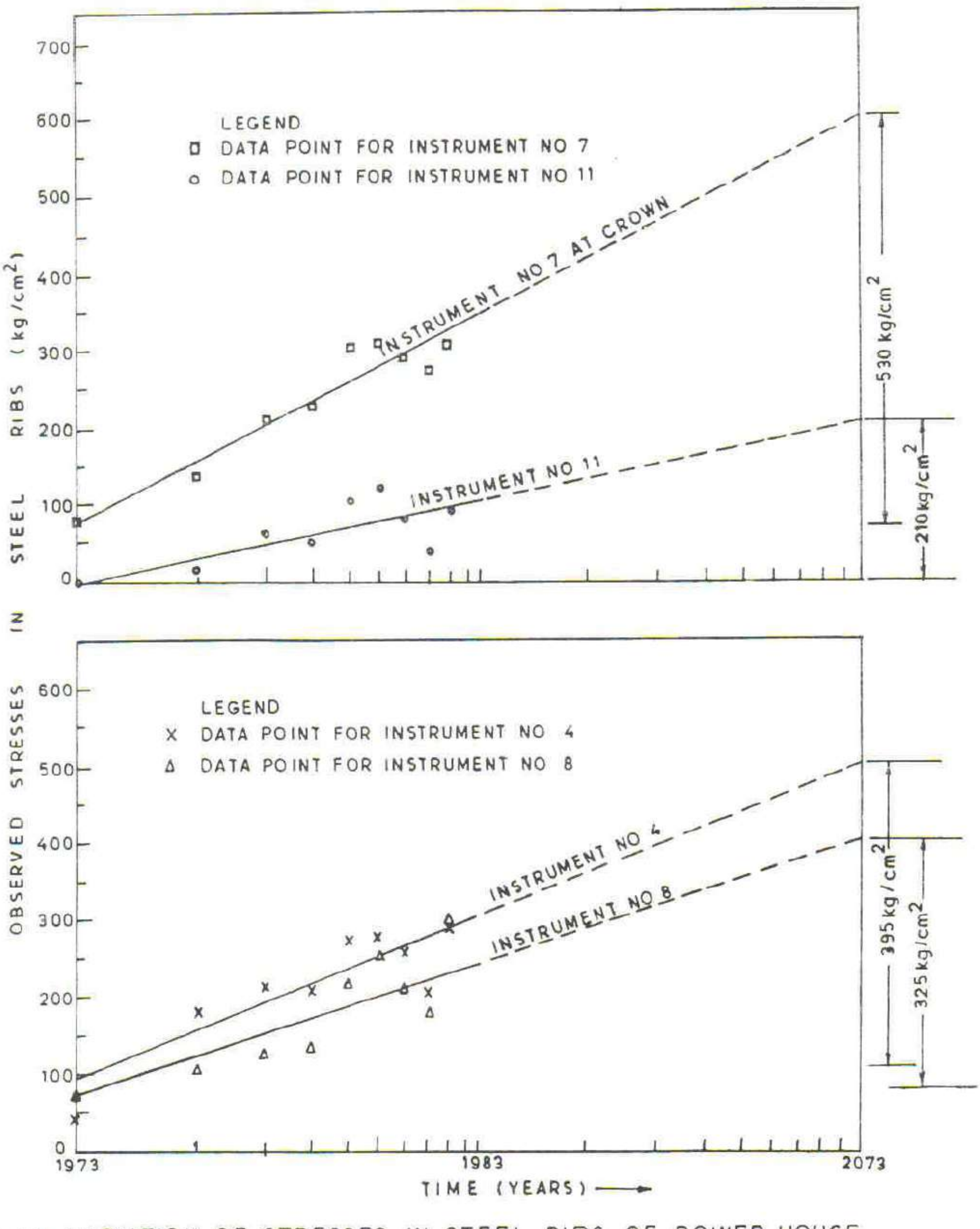


FIG. 10 -SCHEMATIC DIAGRAM SHOWING SELF DRAINAGE SUPPORT SYSTEM BETWEEN THE RIBS



G. 11a-VARIATION OF STRESSES IN STEEL RIBS OF POWER HOUSE CAVITY WITH LOG-TIME AT CHAINAGE 11.0 m



5. 11b- VARIATION OF STRESSES IN STEEL RIBS OF POWER HOUSE CAVITY WITH LOG-TIME AT CHAINAGE 108.5 m

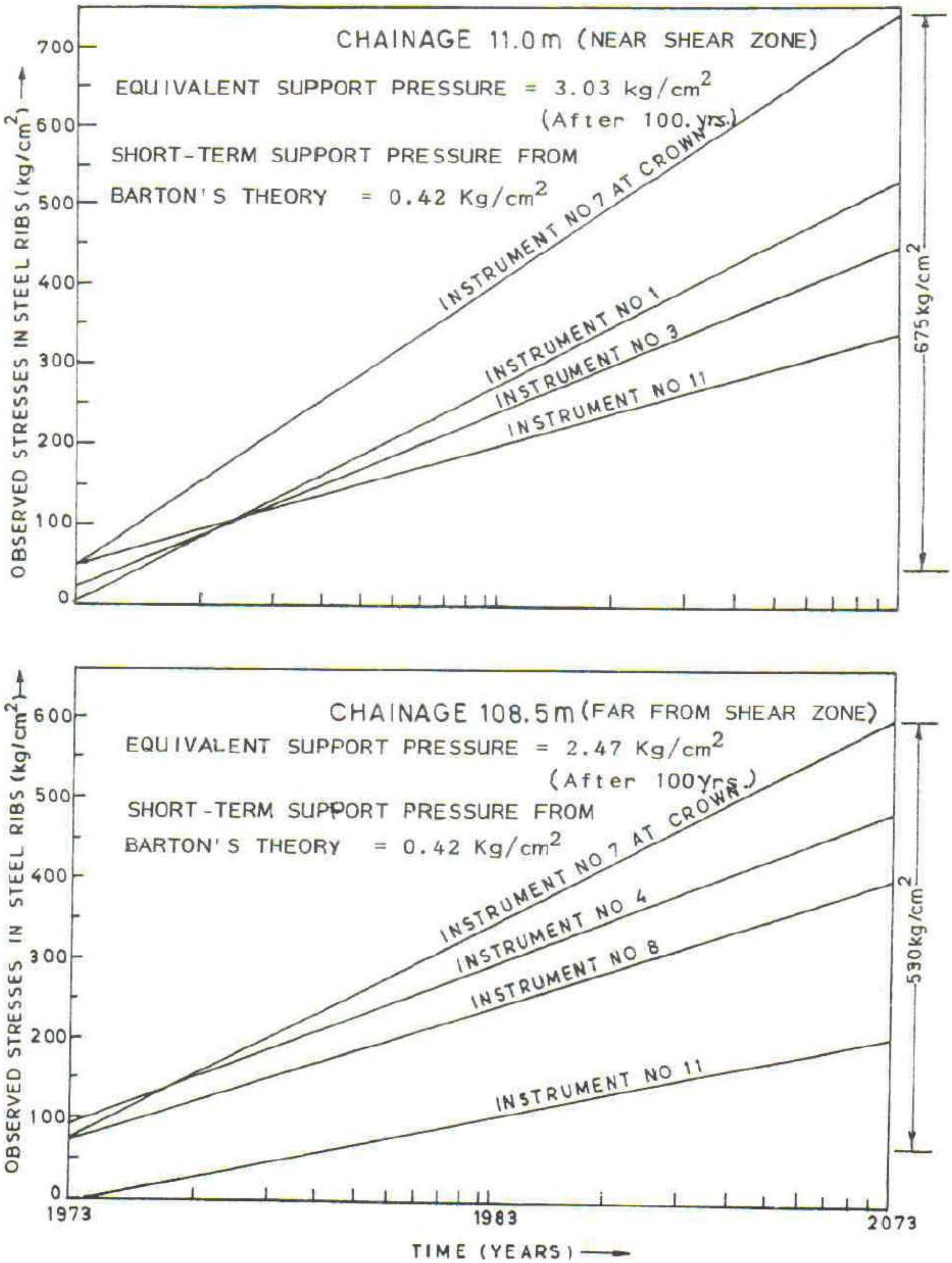
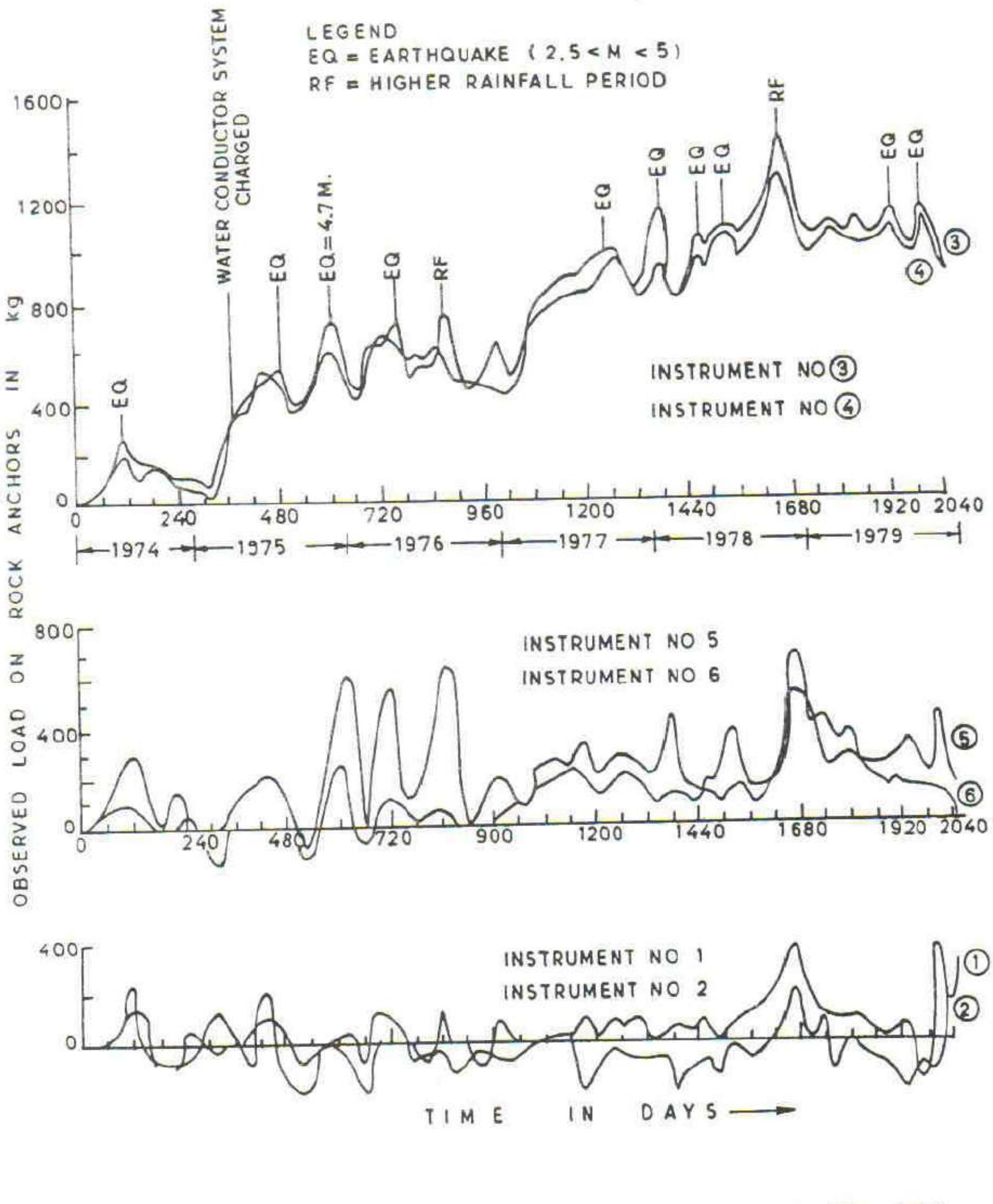


FIG.12 – VARIATION OF STRESSES IN STEEL RIBS OF POWER HOUSE CAVITY WITH LOG TIME (EXTRAPOLATED FOR 100 yrs.)



13 – VARIATION OF ANCHOR LOAD WITH TIME TOGETHER WITH EARTHQUAKE MAGNITUDE AND HIGHER RAINFALL PERIOD (1974-79)

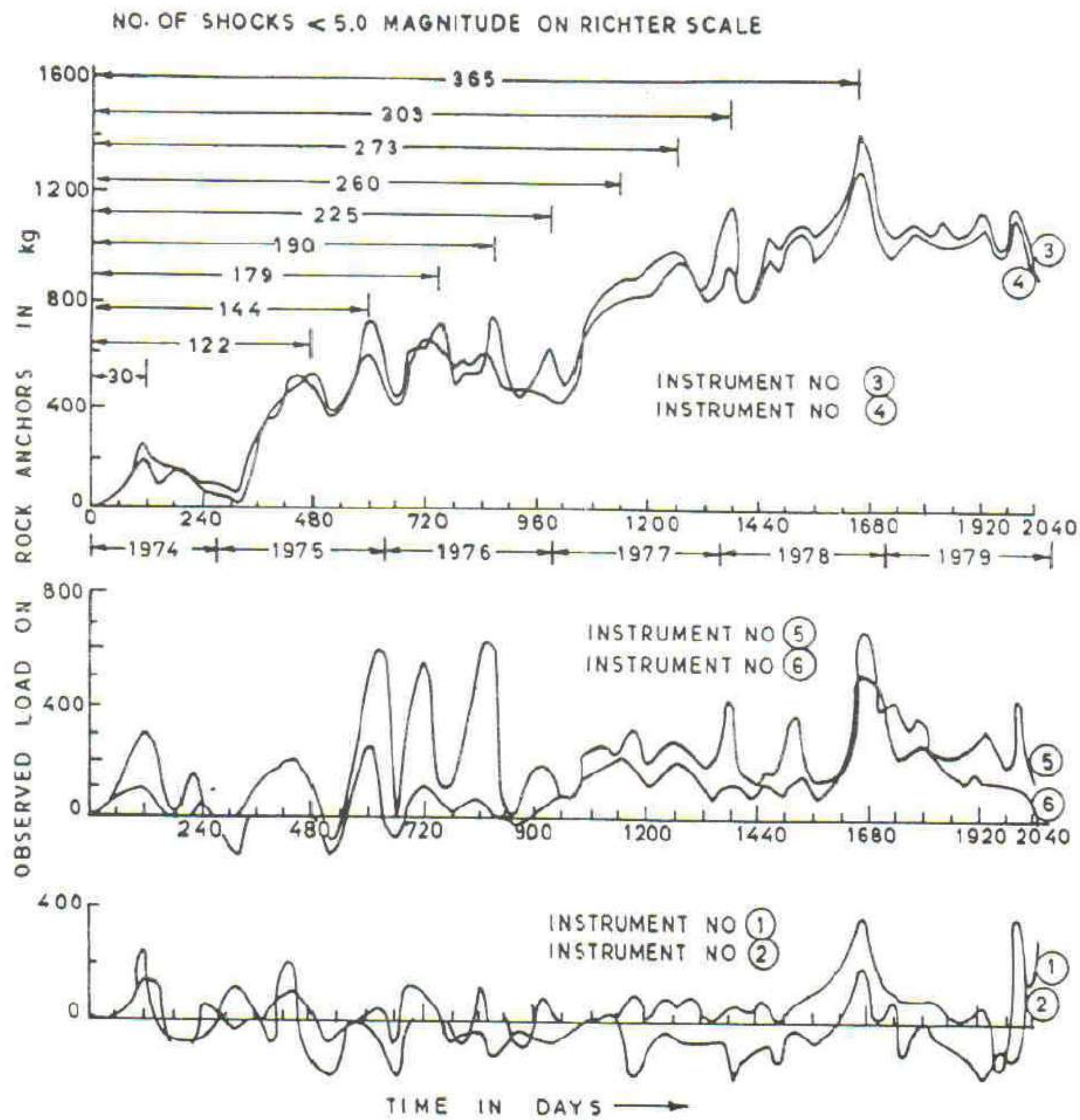
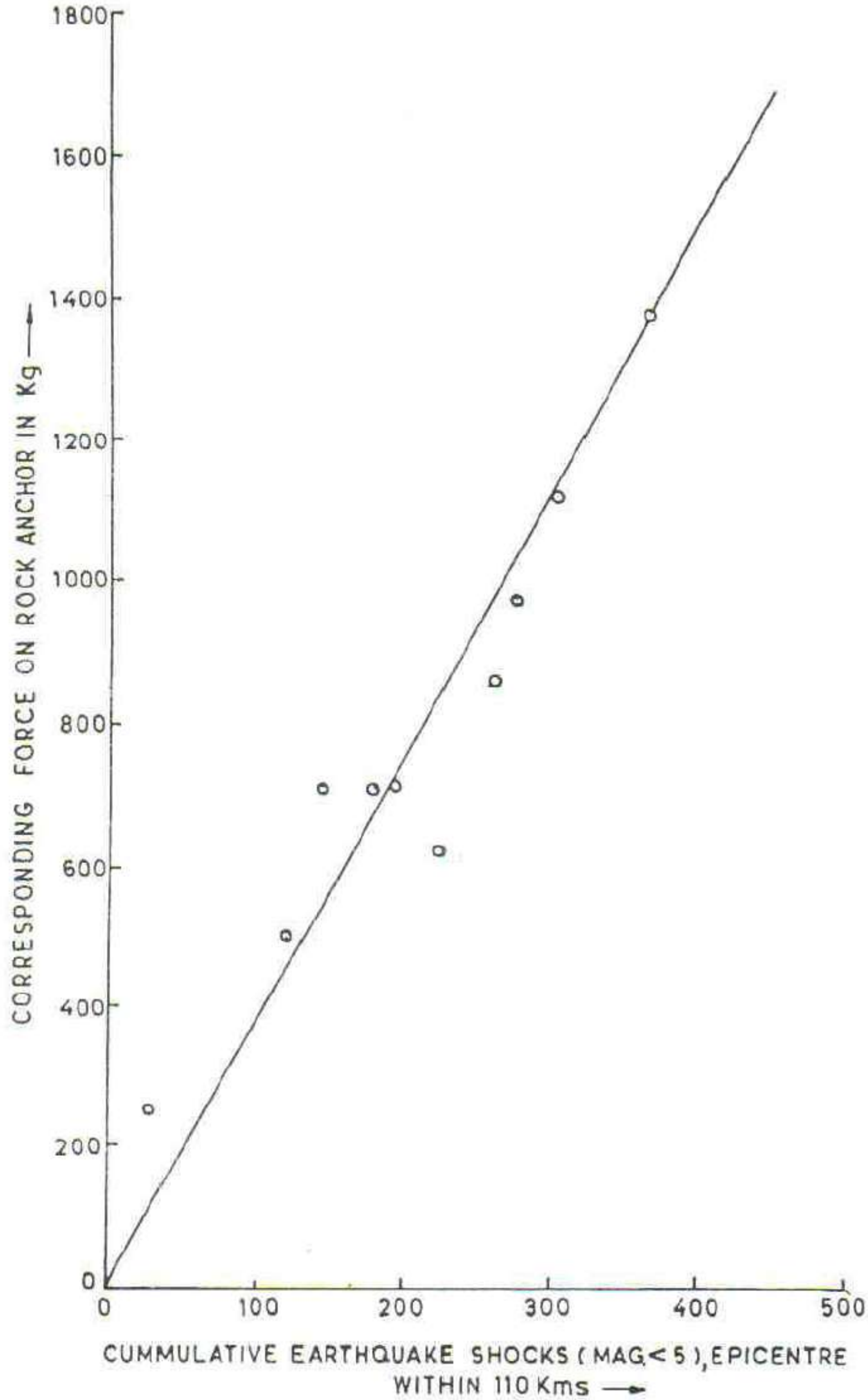


FIG. 14 - VARIATION OF ANCHOR LOAD WITH TIME TOGETHER WITH NUMBER OF EARTHQUAKE SHOCKS



IG. 15 – RELATIONSHIP BETWEEN NUMBER OF EARTHQUAKE SHOCKS (CUMMULATIVE) AND CORRESPONDING ANCHOR LOAD (NEAR THICK SHEAR ZONE)

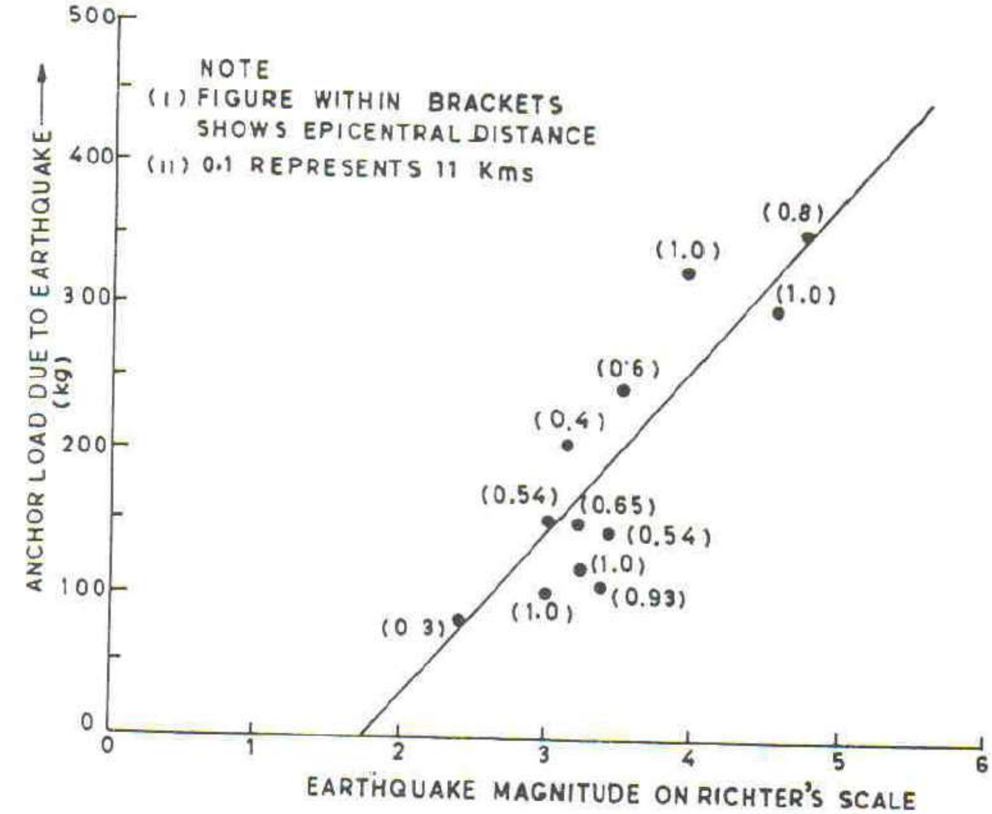


FIG. 16 - RELATIONSHIP BETWEEN EARTHQUAKE MAGNITUDE AND ANCHOR LOAD

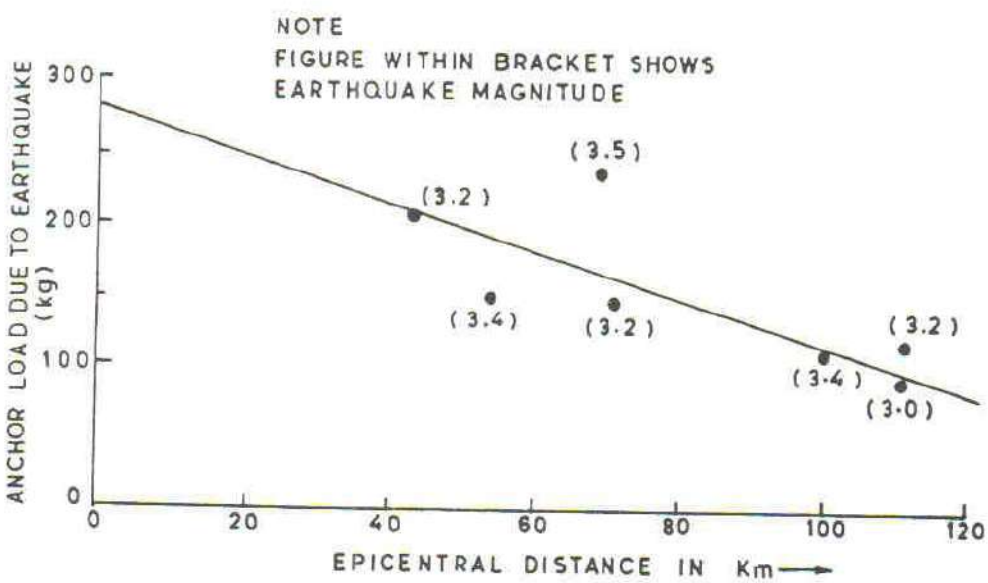


FIG. 17 - RELATIONSHIP BETWEEN EPICENTRAL DISTANCE AND ANCHOR LOAD

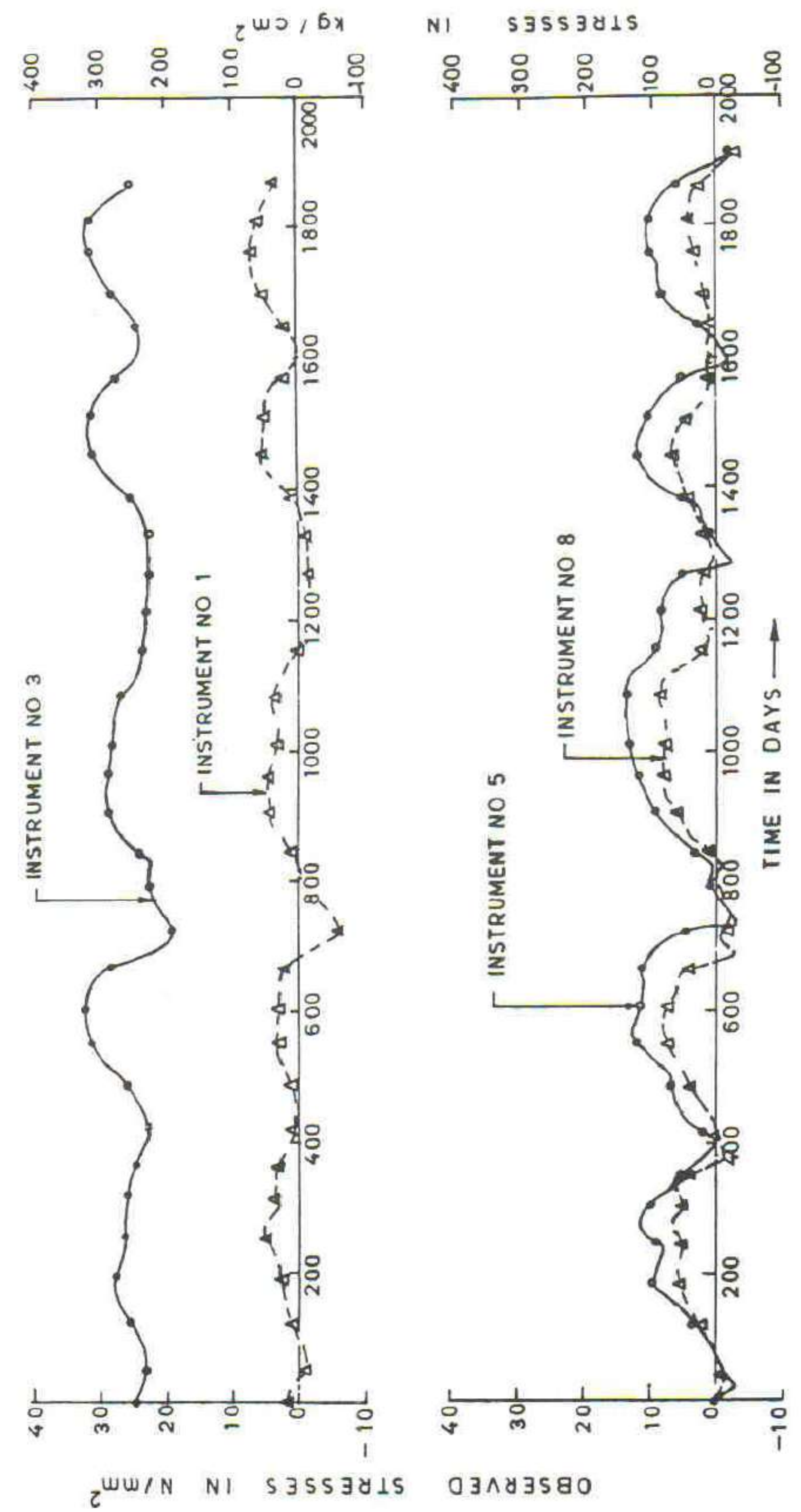


FIG. 1.8 VARIATION OF STRESSES IN STEEL RIBS OF SURGE TANK WITH TIME

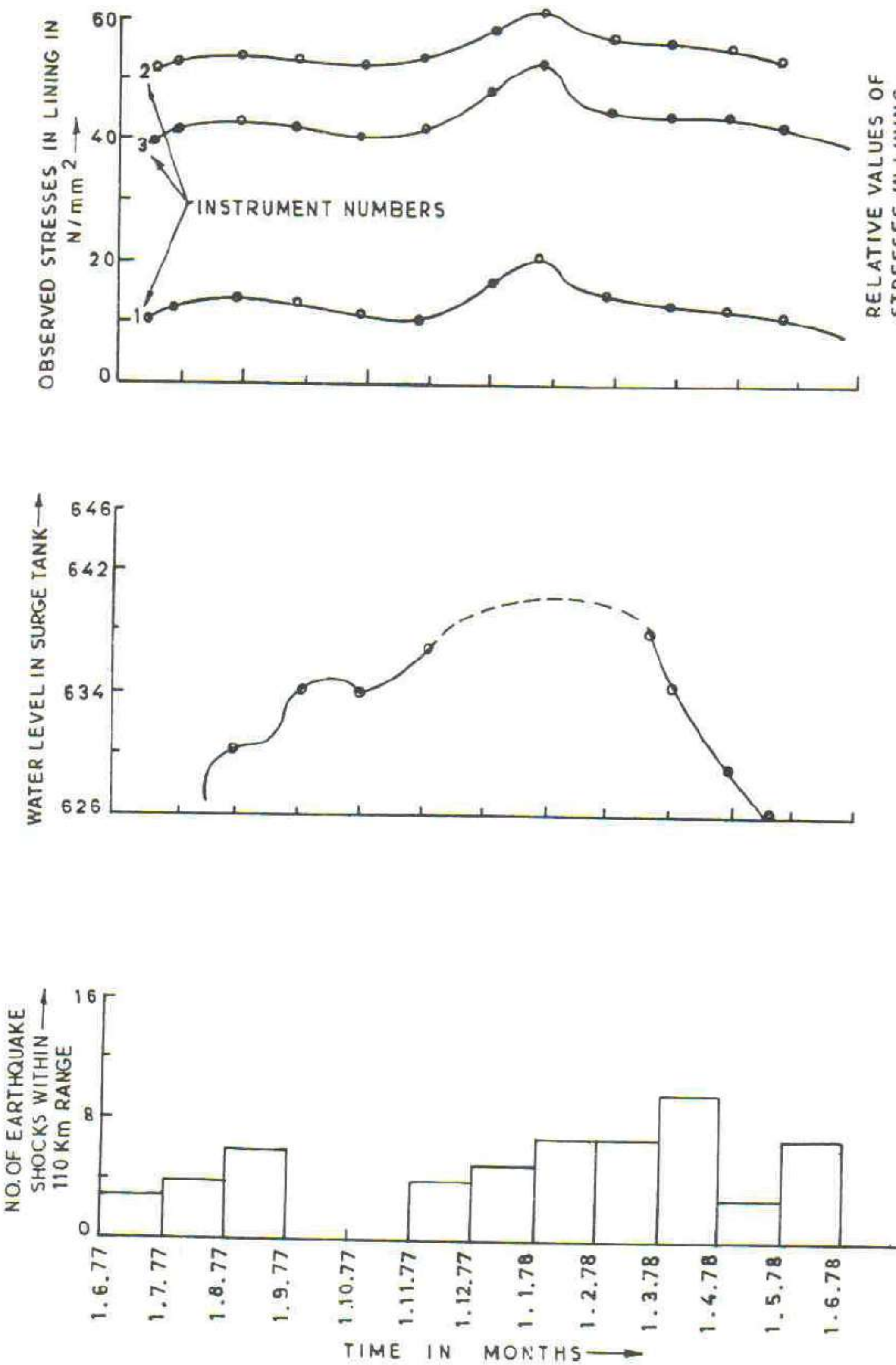


FIG. 19 VARIATION OF STRESSES IN STEEL LINER, LEVEL OF WATER IN SURGE TANK AND EARTHQUAKE SHOCKS WITH TIME (1977-78)