Significance of Instrumentation in Tunnels and Large Underground Openings

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Abstract

Most of the hydroelectric projects in India are located in Himalayas which are characterized by a variety of rock types having varied strength and are often subjected to structural deformation. The Himalayas are also traversed by many thrust and faults which are considered as zones of high stress concentration and influence the tunneling condition.

Rock mass behavior plays a vital role in determining the stability of the underground structures. During excavation, the natural state of stress of rock mass is disturbed. Consequently, the rock mass around the opening begins to deform and causes inward radial movement. This displacement or movement continues till the rock mass reaches an equilibrium with the new state of stress condition. The extent of such deformation depends primarily on the in-situ strength of the rock mass and stress condition before and after the excavation. Implementation of Instrumentation arrays concurrent with the tunneling plays a significant role in understanding the rock mass behavior and its influence on tunnel excavation and large underground openings during and after the excavation. It also helps the designer to plan suitable support system for safety and stability of the structure in the given regime of stress condition around the openings.

Various rock mass properties such as deformation behavior, rock loads, in-situ state of stress, mechanical and shear strength parameters of the rock mass can be monitored by installing instruments during construction and post- construction stage. Instruments also play a key role in optimizing the support system, particularly when the tunneling is done under challenging geological environments such as 'High Stress Condition' or 'squeezing ground' condition.

The present paper gives an overview on the need of instrumentation in tunnels and underground excavation with experiences gained at some of the NHPC projects.

Introduction

Rock mass behavior plays a vital role in determining the state of stability during excavation of tunnels and large underground openings. During excavation, the rock mass gets disturbed and undergo changes in the existing state of stress. Consequently new stresses are developed which tends the rock around the opening to deform resulting into an inward movement of rock mass or closure of the opening. The re-adjustment of stress is reflected in the form of deformation, displacement or loads and on surface it is manifested by cracks, bulging of walls or floors, irregular shape of tunnels, loosening of rock support, buckling of ribs etc. The extent of such deformation depends mainly on the in-situ strength of the rock mass and the amount of pre and post excavation stresses. When the pre-excavation stresses are low and the rockmass is fairly competent, the deformation is less and the rock is said to be in 'elastic state'. However, on the other hand the incompetent rock mass under high pre excavation stresses undergoes large deformation and the rock is said to be under 'plastic stage' or squeezing ground condition. Any type of deformation during excavation of tunnel/underground opening is contained by installing adequate support. Normally, the supports are designed on the basis of rock properties collected during investigations by performing in-situ and laboratory rock mechanic tests. The stresses in the

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rockmass are determined by performing hydro fracturing tests in the exploratory drifts. However, some times, the rock properties varies to a large extent with depth and moreover, the stress regime of many underground structures particularly long head race tunnels is not available during investigation stage and only a broad estimation of rock behaviour is done on the basis of available surface and subsurface data. This sometimes leads to over-design or under-design of the support systems. Both the situation are undesirable, while an over supported tunnel may prove a costly affair, an under supported tunnel may be disastrous. Instrumentation is a means to understand and monitor the behavior of rock mass during and after the excavation of under ground structure. In fact, instrumentation acts as a warning signal to detect and measure any type of distressing that a rock mass is subjected to. Timely augmentation of support may save the structure from further damage and can thus prevent the time and cost overrun.

Rock Movements and Instrumentation

Generally, during excavation, due to redistribution of stresses the rock mass undergoes following changes:

i) **Closure of the tunnel/opening**: Consequent upon the excavation the surrounding rock tries to move gradually towards the centre of the opening which is known as tunnel closure. This happens when the induced stress exceeds the uni-axial compressive strength of the surrounding rock. The rate of such tunnel closure can be measured by Tape Extensometers. A typical tape extensometer arrangement is shown in Fig. 1.

ii) **Radial displacement**: Due to blasting, the natural stresses of rockmass gets disturbed. This leads to loosening and fracturing of rock around the opening. It is important to measure the depth of this zone to optimize the length of the rock

Fig. 1 : Tape Extensometer

Fig. 2 : Multipoint Borehole Extensometer

different depth. These bore hole extensometer can be of wire type or rod type and either mechanical or electrical type. A typical multipoint borehole extensometer is shown in Fig.2.

The location, orientation and length of the extensometer depend upon the geotechnical features of the project and construction methodology for determining the direction, depth and amount of anticipated rock movement. The documentation of geological condition and construction activity in the vicinity of measurement is essential for proper interpretation of field data.

iii) **Rock Load**: Rock load and their mode of manifestation influence the design of tunnel supports. If the rock movements are low the resultant rock loads may also be low and the support will be least stressed and sustain, however, if the stresses are high the support may fail. The magnitude of rock load on the support can be measured by the aid of load calls. These load cells can be either hydraulic, mechanical or strain gauge type. A typical load cell is shown in Fig.3.

3. **Phases of Instrumentation:** Any instrumentation programme has mainly

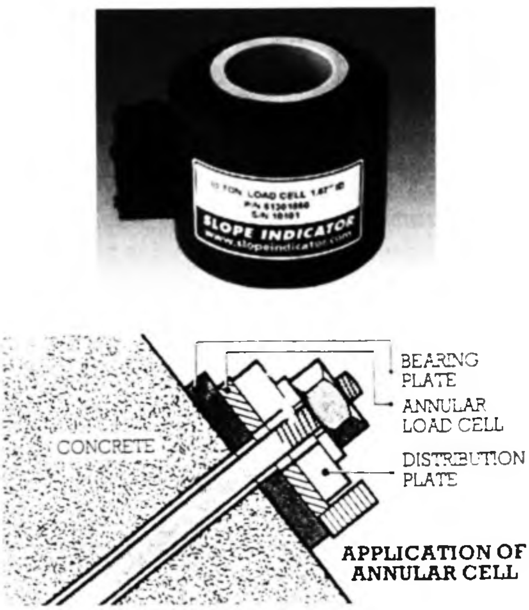


Fig. 3 : Load Cell

three phases:

i) Planning and Design

ii) Implementation.

iii) Data interpretation and its utilization.

Specific aim of instrumentation programme needs to be decided during the planning stage. This aims becomes the guideline for selecting the type of instrumentation that can be used to achieve these aims.

Objectives of Instrumentation

The initial support designing is done on the basis of rock characteristics and the geotechnical parameters collected during the field investigations. However, sometimes, due to site limitations only scanty data on rockmass characteristic is available. In such cases, the design is based on certain assumptions which may lead to over design or under design. An economic design is possible by adopting systematic instrumentation. Instrumentation data is also helpful in taking decision on tunnel lining and reinforcement and consolidation grouting.

In some circumstances, it is possible to install the instruments before the commencement of actual excavation from adjacent exploration drifts etc. Instrumentation is also helpful in analyzing the response of the under ground structure to the excavation carried out in the neighbourhood. The objectives of instrumentation can be classified as follows:

i) **Before Construction**: To collect information on rock behaviour required for design/support of the excavation.

ii) **During Construction**: To confirm the adequacy of the design support and to provide a data base for changes in design, if required. Monitoring of displacement can be used to improve the stability and safety of underground structures.

iii) **After Construction**: To monitor the response of adjacent excavation upon the structure and overall behaviour of the rock mass during construction.

Inadequacies in Instrumentation Programme

It is often thought that instrumentation is a time consuming, costly process and hampers the construction schedule. Another misconception about instrumentation is that the results may be of academic interest but may not be useful to the project. These misgivings often results in inadequacies in implementing, the instrumentation programme in a holistic way. Generally, following inadequacies in instrumentation programme are observed.

- **Initial rock mass behaviour not well established**. The instrumentations should be concurrent with the excavation; however, sometimes the instruments are installed much later after the excavation that the initial behaviour pattern of rock mass goes unnoticed.
- **Inexperience of crew in installation and observation of the instrument:** The inadequate installation of instrument often results in damage of the costly equipments. The casual approach in collecting instrumentation data often results in recording of wrong data and the crew sometimes fails to detect the instrument behaviour or warning signals for instability.
- **Delay in Interpretation:** The instrumentation data needs to be analyzed immediately for fruitful results. Any delay may prove harmful to the structures at a later stage.
- **Instrument damage due to construction activity:** Proper handling of the instruments is necessary. Sometime the instruments are damaged due to blasting activity, vehicular movement inside the tunnels, carelessness of the labours etc.
- **Contractor Responsibilities not well defined**: The instrument programme should be given due cognizance in the contract. In tunnel construction the excavation is done by one contractor while the instrumentation is awarded to another contractor. This result in conflict, while the contractor responsible for instrumentation argues for safety and stability of tunnel, the other contractor is concerned for its progress. Clear responsibilities of the contractors should be defined in the contract as progress as well as safety and stability of the tunnels in important.
- **Inadequacy of the instruments:** The instruments are installed for monitoring the long term stability of structure therefore it should be both sensitive as well as robust to withstand the adverse condition during under ground excavation such as seepage, vehicular movements, etc. Inferior quality of instruments may lead to erratic readings and wrong interpretation of data.

The instruments should be installed concurrently with the excavation of under ground openings. It is observed that the cycle time of construction increases by few hours when the instruments are installed. There is also some time loss when the periodical observation of instruments is taken. However, this time is very meager when compared with the total construction period of the project.

The instrumentation process is also not very costly and money invested in it is paid back manifolds in the form of stable tunnels and under ground openings. Below are some of the examples of large under ground opening and tunnels excavated by NHPC, where instrument has played a vital role for optimization of support and long term stability of the structure.

Instrumentation for Large Under Ground Openings:

Chamera Stage-I Underground Powerhouse:

Chamera Hydroelectric Project is located on river Ravi in Himachal Pradesh. The project comprises of a 121m high concrete gravity arch dam, a 6.4 km long head race tunnel and an underground power house having 3 units of 180MW each for generating 540MW

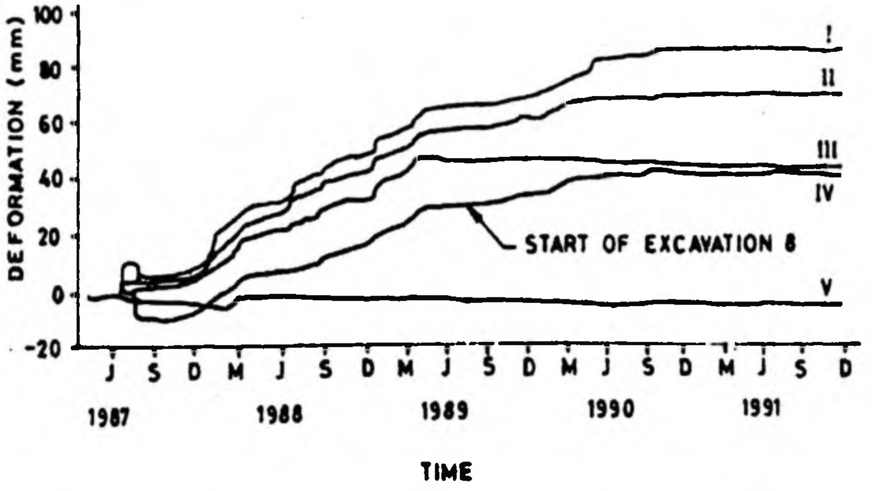
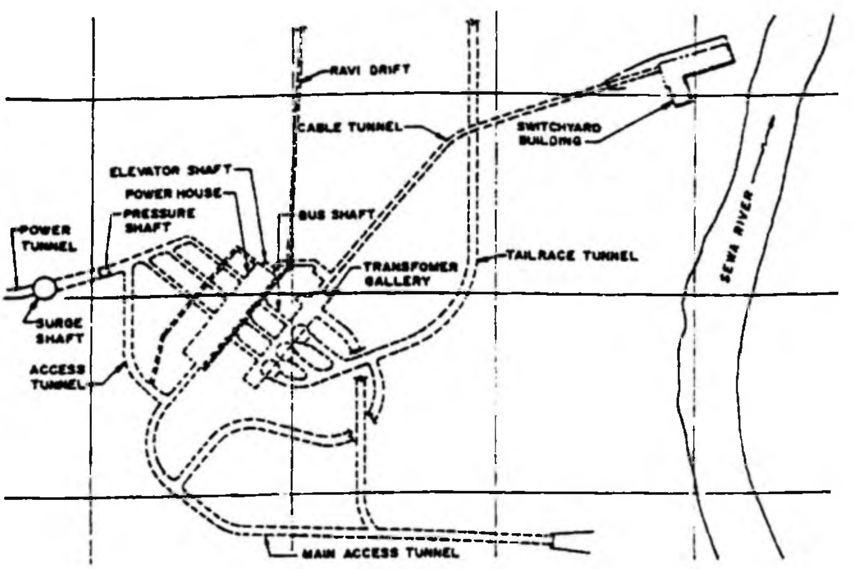


Fig. 5. Typical MPBX Data, Chamera Powerhouse

of power. The power house discharge is routed back into Ravi river through a 2.4 km long tail race tunnel. The project was commissioned in 1994.

The underground power house cavern having dimensions 112m long, 24m wide and 37 m high is located on the right bank of Ravi river, inside the hill in blocky to foliated meta volcanics of Panjal traps with overall rock cover of 300m above the cavern.

Effective and systematic instrumentation campaign was carried out for the large under ground opening of powerhouse by installing an array of instruments to monitor the rockmass behaviour concurrent with the excavation. Various instruments such as mechanical borehole extensometer, electrical bore hole extensometer, tape convergence meter and load calls were installed to monitor the deformation in the rock mass during excavation. The observation recorded from each instruments and their implication on the cavern excavation is discussed in the



Layout of Chamera Underground Powerhouse following paragraphs.

Multipoint borehole extensometer (Mechanical): During the investigation stage, a 530m long drift (2m x 1.5m) namely Ravi adit was made above the power house cavern to ascertain the geological condition of the power house and for performing in-situ rock mechanic tests. This drift was about 32m above the cavern and was running parallel to the downstream wall of the cavern. Two cross cut running transversely across the power house cavern, corresponding to RD 57m and RD 87m of the power house cavern were made in the drift. For the purpose of measuring crown deformation of the power house cavern an array of three multiple point extensometers were installed in each cross cut. The depth of deepest anchor varies from 30m to 32.5m. A typical arrangement of these extensometers is shown in Fig-4.

The crown deformation was observed from June'87 till the completion of entire power house excavation in March'91. One such plot showing deformation vs time at ch 87m is given as Fig-5. It was seen that more deformation were noticed at ch.87m as compared to ch.56m due to the presence of 3m to 4m thick crushed and sheared very closely foliated zone between ch.85m to 95m of the power house cavern whereas the metabasic rock around ch.56m was more competent. During the top heading (El.569.5 to 561m) and first bench excavation (El.561m-558m) more deformation of the order of 13mm to 23mm was noticed on the upstream of the crown whereas downstream of the crown it was about 8mm. However, during the excavation of 2nd bench from El.558m to 553m and central gullet from El.553m to 547m, the crown movements were more pronounced in downstream, mainly due to the opening of bus ducts and draft tubes on the some wall. A cumulative deformation of 15mm was noticed, whereas a deformation of 8mm was observed in upstream of the crown during the same period. The deformation of the downstream wall was also manifested in the form of cracks in shotcrete between El.563m and el.553m. Further, a big cavity also

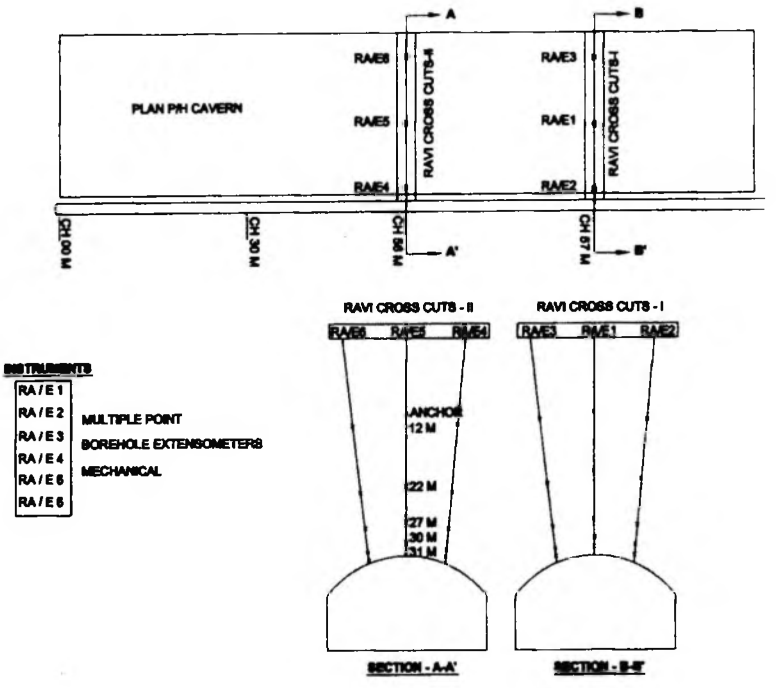


Fig. 4 : Arrangement of MPBX from Ravi Drift

developed in the draft tube. Due to this, the bench excavation from El.547 to 544m remained suspended. The finite element analysis carried out indicated an overstressed zone of 8m periphery in the downstream wall.

The support system was further augmented by installing 10m to 12m long rock bolts. Support in upstream wall was also upgraded.

Tape Extensometer: Detailed tape convergence measurement between upstream and downstream walls was undertaken during various stages of power house excavation. In all, 18 stations were established between ch.30m, 56m, 75m and 95m at various elevations. The arrangement of these stations is shown in Fig 6.

A maximum cumulative convergence of 138mm was recorded between the upstream and downstream wall of the power house cavern during the excavation of the cavern. A typical tape extensometer plot is shown in Fig-7. As can be seen from the plot, significant jump in the deformation were noticed due to release of stress buildup over a period of time, particularly in the start of 1989 till 1990 when the excavation of powerhouse was in full swing. The stresses began to stabilize during late 90's and early 91', when the cavern excavation was almost complete. A comparison of tape extensometer and borehole extensometer reading shows marked similarity in the deformation pattern which is further supported by load cell data also.

Load Cells: Mainly two type of load cells, the hydraulic type annular load cells of 250KN capacity and Electrical strain gauge load cells were installed to measure the load on the rock bolts. Observation of various load cells reading revealed that loads on the support increased gradually with the excavation of cavern, particularly on the downstream wall which can be attributed to the excavation of bus ducts and draft tubes. These observations were further supported by tape convergence measurements which also showed a similar trend.

The above instrumentation programme was very useful in understanding the rock mass behaviour and had necessitated the need of increase in support system for stability of the structure. It was also established from the

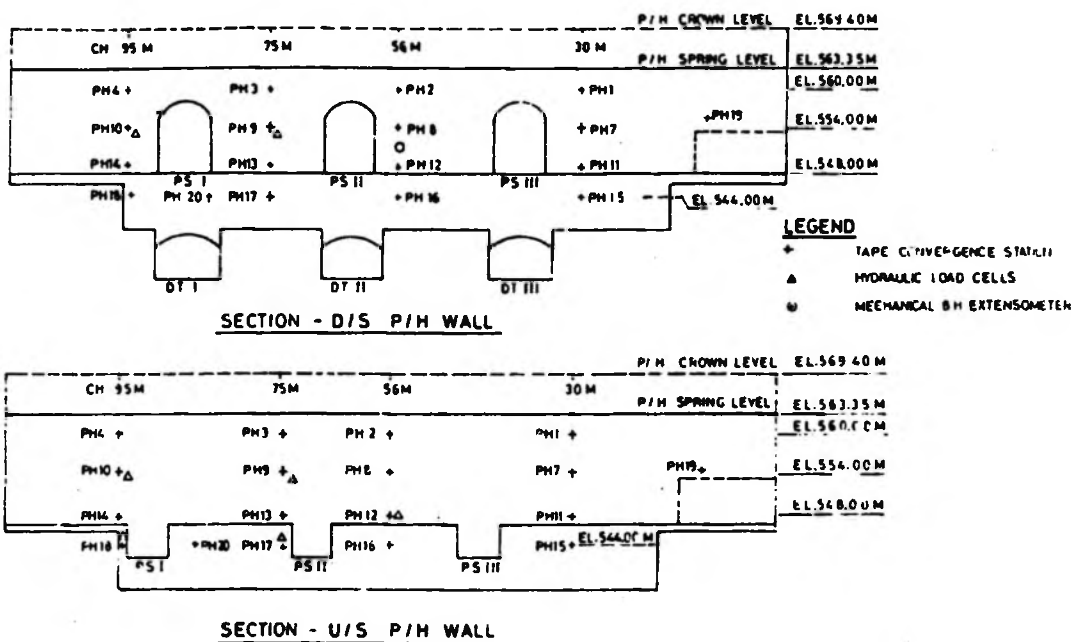


Fig. 6 : Arrangement of Various Instruments in Powerhouse Cavern

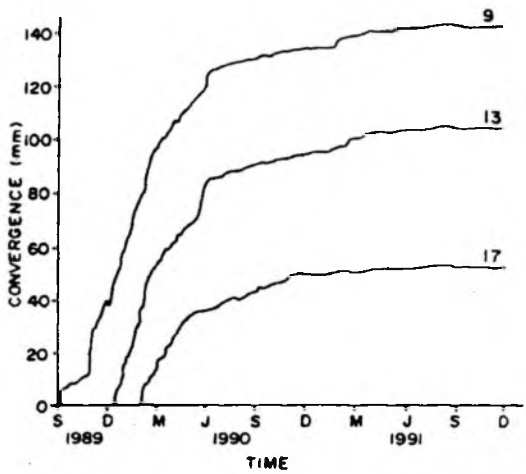


Fig 7. Plot of tape extensometer

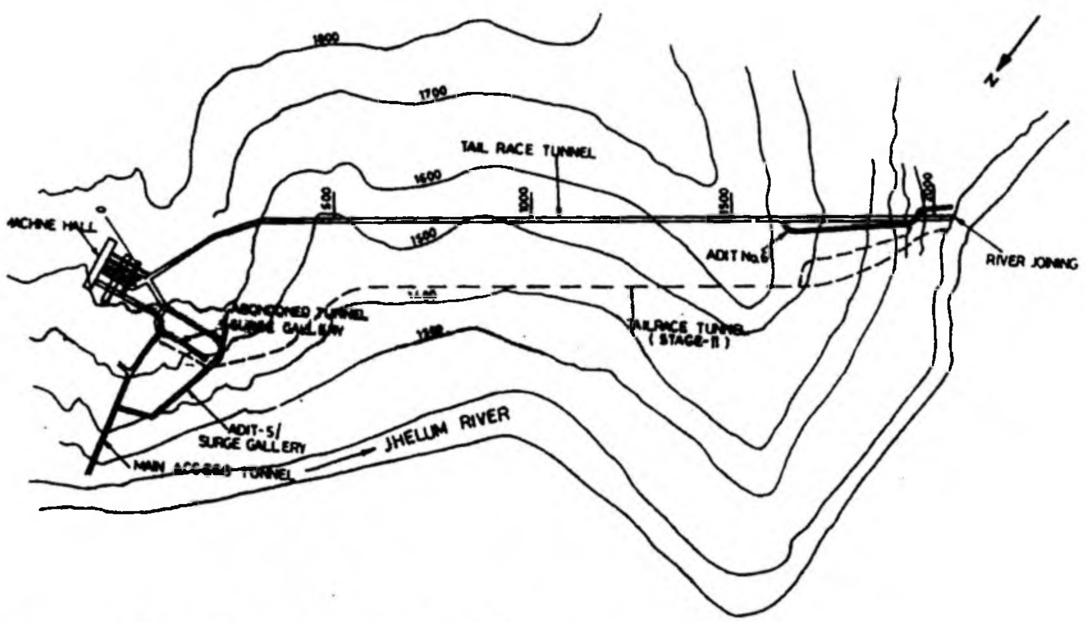
above instrumentation that the deformation were more pronounced on the downstream wail due to simultaneous excavation of many opening viz bus duct, draft tubes and transformer cavern in the neighbourhood.

Another example exhibiting the importance of instrumentation in squeezing ground condition encountered in Tail race tunnel of Uri Project located in Western Himalayas.

Instrumentation in Squeezing Ground Condition Uri Project

The 480MW Uri HE Project, Stage-I is located on river Jhelum in Baramulla district of J&K state. The project consist of a barrage, a10.5mkm long head race tunnel, a 22.5m dia, 91m high surge shaft, an underground power house and a 2.05km long tail race tunnel. The project was commissioned in 1997.

The tail race tunnel area of the project exhibits a complex geology. The alignment passes through two rock units namely Panjal volcanics and Eocene shales. The contact of the two units is marked by the Panjal thrust which was folded also. Due to the presence of this thrust zone, the Eocene rocks comprising of shales with pockets of limestone and gypsum and meta volcanics comprising of carbonaceous phyllite, graphitic schist, chlorite schist were highly disturbed, sheared and crushed and constituted a poor tunneling media. Ground water seepage was also copious in some reaches. The rock cover over a substantial



Layout of Tail Race Tunnel: URI H.E. Project, Stage-I portion of the tunnel was more than 300m which was also bound to trigger some deformation, particularly in weak rock formation. In view of the poor/very poor rock condition expected in nearly 50% stretch of the tunnel, the excavation was planned through heading and benching in poor rock condition and multi drifting in very poor rock condition. Advance probe drilling was carried out to ascertain the rock condition ahead of face.

The excavation of TRT was taken up through three faces. From Adit-5, the tunnel excavation was done for a length of 329m in downstream direction.Adit-6 met the TRT at RD 1627m and from here the tunnel excavation progressed both in upstream direction upto RD 329m and in downstream direction upto the outlet. The rock strata between RD 38m to RD 329m from Face-5 and RD 1319 to 1560m were most critical due to the presence of weak carbonaceous phyllite and graphitic schist in the former stretch and poor quality shales with pockets of gypsum and carbonaceous material in the latter reach, rendering to highly squeezing ground condition.

The rock support elements in TRT consist of 3m long dowels and 30-60mm fibre shotcrete for Class I & II rock, for poor to very poor rock, the dowel length were 4mc/c at 2m spacing and 100-150 mm fibre shotcrete.In highly squeezing ground condition grouted dowels of 6m length @ 1.5m c/c spacing with five skins of wiremesh and shotcrete were provided.

The excavation of the tunnel was duly backed by tape extensometers to record the extent of squeezing. In class-III & IV rock, the spacing of tape extensometer stations was at 5 to 10m interval whereas in other sections the tape extensometers were installed at 20m to 100m interval. It was seen that the stretch between RD 170 to 255m were the most critical where the deformation rate of 0.6mm/day and a cumulative deformation of 170 to 225mm were noticed in a period of 18 months. In addition to above instrumentation, two test line sections were also casted between RD1380-1385m and RD 1558-1563m in variegated shales and metavolcanics respectively. These sections were continuously monitored for deformation and rock loads. In these tests lining, the rock load were determined to be of the order of 0.6MPa to 1.4MPa. It was ascertained that a deformation value of 0.015mm/day is acceptable to cast the final lining of the tunnel. In the original design proposal lining thickness for class-I & II rock was stipulated as 250mm whereas for class-III & IV, the same was 450mm. However, a thickness of 350mm was considered adequate irrespective of rock class. In November 1995, after 3 months of the excavation, the deformation rates in the TRT, between chainage 0-500m were in the range of 0.02mm/day to 0.07mm/day whereas in rest of the reaches it was less than 0.015mm/day. While the lining work between ch.500m to 2000m was taken up, pouring of concrete between RD 0-500m was delayed to allow the deformation to cease to acceptable limits. In this section, the rock support was further augmented by providing 20mm bars to form an outer ring and 10mm bar for inner ring between ch.170-325m while for Chainage 30m-170m and 325-360m only inner reinforcement of 10mm bar was provided. It was seen that even after augmenting the support, the deformation rate, even after six months of excavation, were of the order of 0.023mm/day 0.025mm/day between ch.290-330m, which were still higher than the prescribed limit for 350mm lining. In this section, a 450mm thick lining was provided as sufficient space for the same was available. This reach remained stable and did not pose any problem later on.

It is pertinent to mention here that inspite of excavation in poor to very poor rock strata where highly squeezing with some flowing ground condition was encountered, no cavity or rock collapses were observed. Also, no conventional rib support was installed and the tunnel, even in problematic reaches was supported only by rock dowels, wiremesh and fibre reinforced shotcrete only, thus making huge savings on cost and time of the project. This could be made possible due to meticulous planning, advance horizontal probe holes, superior blasting and construction methodology and use of fibre reinforced shotcrete supported by effective instrumentation programme.

Conclusion

Instrumentation plays a vital role in determining the rock movement during excavation and is helpful in determining the adequacy of the support provided for large underground openings. In today's context when most of the under ground works are being undertaken in complex geological setup, relevance of instrumentation has increased manifold. The examples of Chamera underground power house and tail race tunnel of Uri Stage-I Project aptly demonstrate that instrumentation is not only useful in augmenting the design support but also helpful in deciding the construction methodology and lining thickness of the tunnel and are helpful in preventing the time and cost overrun of the project.

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