International Journal of Architectural, Civil and Construction Sciences
ISSN: 2415-1734
Vol:10, No:6, 2016

Practices in Planning, Design and Construction of Head Race Tunnel of a Hydroelectric Project

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***Abstract*—**A channel/tunnel, which carries the water to the penstock/pressure shaft is called headrace tunnel (HRT). It is necessary to know the general topography, geology of the area, state of stress and other mechanical properties of the strata. For this certain topographical and geological investigations, in-situ and laboratory tests, and observations are required to be done. These investigations play an important role in a tunnel design as these help in deciding the optimum layout, shape and size and support requirements of the tunnel. The paper includes inputs from Nathpa Jhakri Hydeoelectric project which is India's highest capacity (1500 MW) operating hydroelectric project. The paper would help the design engineers with various new concepts and preparedness against geological surprises.

***Keywords*—**Tunnelling, geology, head race tunnel, rockmass.

I. Introduction

PLANNING is very important component for the optimum development of a hydroelectric project. Investigations at site depend upon the nature of the site. Based on the investigations optimum dimensions of different hydroelectric components i.e. dam, reservoir, HRT and powerhouse can be worked out that could give the most economical scheme [3].

Best practices in planning, design and construction of HRT is taken care of at feasibility report stage, detailed project report stage, tender preparation stage, construction design stage and implementation stage. The good practices involved in planning and design at various stages includes complete integration of interdisciplinary knowledge. An organization's past experience, adequate staffing in terms of specialized engineering groups and exposure to new construction and design practices are essential inputs for a good design. Moreover, in planning, design and construction of HRT, following considerations are taken into account:

1. Plan alignment considering the geology to be met and topographical consideration
2. No. of construction faces, length and position of adits with a view to optimize the construction schedule
3. Size of the tunnel decided on the basis of the velocity, head loss and economic consideration

II. Role of Geological Knowledge

Geological investigations for tunnel site are conducted in three stages. In the initial stage, a desk study is undertaken using available maps and aerial photographs to obtain an overall impression of the geological conditions and to plan subsequent investigations. The second stage requires a more detailed investigation and it is geared to the determination of the feasibility of a particular location. At this stage, consideration is given to alternate selected tunnel alignments. Once a tunnel site is selected, then investigation enters the third phase when special additional work is conducted to assist the final design and estimation of tunnel costs. The investigation should produce a geological map of the area and a cross section along the centre line of the tunnel. Wherever possible, the position of the water table should be shown on the section [2]-[4].

The geology along a tunnel alignment plays an important role in many of the major decisions that must be made in planning, designing, and constructing a tunnel. The feasibility, geology along the tunnel alignment and cost of any tunnel depends solely upon the geology. An adequate intensity of site exploration, from which geological and hydrological mappings and ground profiles are derived, is most important for choosing the appropriate tunnel design and excavation method. The rock conditions of the tunnelling location need thorough exploration to study the response of the immediate rocks surrounding the tunnel to disturbance caused by excavation; rock strength, collapse potential and rock pressure in order to satisfy the chosen design and construction method. It has long been recognized that detailed and prior knowledge of the geological character of a tunnelling site is a pre-requisite to successful design, construction and operation for the majority of such projects.

It is to be noted that inspection and mapping of strata should continue during tunnel construction. This information helps to complete the picture of the geological setting as revealed by the site investigation and help the geologist predict any changing condition in advance of the tunnel heading.

III. Design Aspects of a Tunnel

A. Tunnel Layout

Tunnel layout is the route layout of the tunnel in plan [5]. The layout is usually governed by the geology, topography and the type of hydropower scheme adopted. It must be noted that the complicated geological conditions and extraordinary geological occurrences such as intra-thrust zones, very wide shear zones, geothermal zones of high temperature, cold/hot water springs, water charged rock masses, intrusions, fault planes, etc. should preferably be avoided while deciding the layout and sound, homogeneous isotropic and solid rock formations should be selected.

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B. Selection of the Alignment

While selecting the alignment the following points should be considered:

(i) It should be the Shortest Possible: This would ensure minimum losses and shall be economically cheapest.

(ii) It should be Straight as far as Possible: Introduction of bends in the alignment shall involve losses at all such bends and the cost of tunnelling would also increase,

(iii) It should be Easily Accessible: An easy access near the entrance and exit to the tunnel becomes essential for the construction facility,

(iv) Careful selection of entry and exit locations with minimum length and depth of approach cuttings and no weathered, loose fractured layers slope towards portals.

However, it is not always possible to follow a straight alignment because of the topographical and geological obstructions prevailing along the length of the tunnel. This is discussed below with an example.

1) The Alignment of the Tunnel Is Greatly Affected by the Topography and Geology

SJVN Ltd. had two alternatives i.e. to keep the HRT on the left or right bank. The right bank which is comparatively gentle in nature is dominated by overburden, mainly in the form of slope debris with scanty rock exposures and lot of cultivation. The right bank slopes are dissected by 23 nallahs. In contrast the left bank slopes are rocky with less nallahs. Moreover, the left bank consists of massive quartzite for considerable length which would be suitable for butting of the dam and for locating the intake of the HRT. Therefore, the nature of the left bank slopes and rock type influenced the decision to keep the HRT on the left bank.

2) Selection Based on Geology Dominates the Accessibility Criteria

It is not necessary that we have to always choose the location which is easily accessible. As in the above case though a PWD road was available on the right bank from Devraulli to proposed Power house site still right bank was not chosen, as the geological mapping of the right bank alignment of HRT has suggested a unfavourable geological conditions whereas the general steep slopes of the hills and steep side faces in the first few kilometres of HRT (from dam site) indicate the overall favourable topography for placing the HRT on the left bank.

C. Selection of the Adit and Exit Locations

In the choice of the tunnel axis the careful selection of the adit and exit locations is important. It has occurred more than once that an incorrectly selected portal has broken down shortly after the commencement of excavation and has had to be abandoned for another site. Conditions are especially favourable where weathered, loose, fractured layers slope towards the portal. If these are perforated by an approach cutting before a sufficiently resistant tunnel portal structure is built, the entire slope may be mobilized and sliding can no longer be arrested.

Rockfall protection measures comprising of scaling and cleaning of the rock slope and rock fencing will be prudent to install as part of the portal work to protect workers and equipment during portal and tunnel excavation. Moreover, the correct selection of length of approach cutting is also of great significance. For economic reasons its depth should not exceed 20-25 m yet at the same time the adit section should not be sited in a sliding layer as shown below.

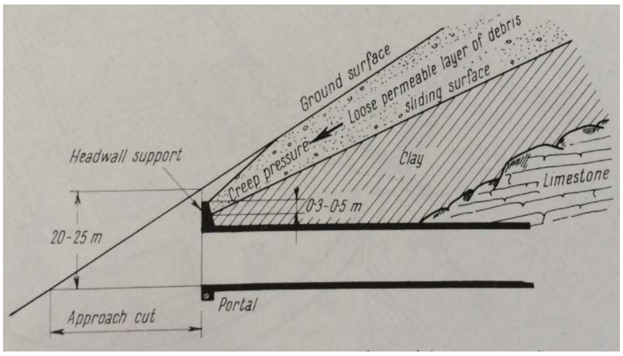

Fig. 1 Design of a Portal [2]

The adit section shall not be sited in a sliding layer and highly resistant structures must be designed in uncertain layers and the coping of the portal (capping or covering of a wall to manage stressful circumstances) should reach above the fractured layers covering the slope.

D. Deciding the Number of Adits

The alignment of the tunnel is so chosen that adequate vertical and horizontal cover for the tunnel is ensured and there are minimum bends on its way commensurate with the maximum distance between two adjoining adits as 5 km. The optimum number of adits is decided based upon the planned construction schedule. The location of the adits has to be chosen based on the existing topographical and geological conditions so that the planned targets are met and the resources are utilized optimally. The size of the adits should be wide enough to cater easy movement of traffic both ways so that the speed of tunnelling is maintained. Keeping in view the length of the tunnel, the provision of access in the adit plugs shall be provided. This will facilitate the future inspection inspection of the HRT after emptying when essential.

E. Tunnel Cross-Section

The aspect requires the determination of the size and shape of the tunnel. HRT is classified on the basis of flow i.e. Pressure flow and open channel flow system and the shape of the tunnel i.e. circular, d-shaped, horse-shoe & egg shaped sections.

1) Influence of Geological Environment on Shape of Cross-Section

A factor of considerable influence on the shape of the cross-section is the type of geological environment in which the tunnel is constructed. The magnitude of external loads i.e. the rock pressure depends on the inherent strength of the rock material, the quality of which is responsible for the ratio between vertical and lateral pressures acting on the tunnel. In loose, soft and weak rock materials large rock pressures and a relatively considerable lateral thrust may be expected. The greater the relative magnitude of the lateral thrust the more advantageous a circular section will be found to be. In solid rocks showing no tendency to weathering, tunnel sections excavated with an arched roof may serve without resort to any lining. In deciding the shape of the cross-section, in addition to structural consideration, earth pressure theory may also serve as a guide i.e. cross section is to be selected in accordance with the relative magnitude of vertical and horizontal loads (Rankine's earth pressure theory). Where the vertical loads are relatively larger, an ellipse with a vertical major axis is preferable, while for relatively large horizontal pressures one with a horizontal major axis will prove more suitable, but in practice a shape composed of circle segments is used instead of an ellipse [2].

2) Influence of Construction Method and Lining Material on the Shape of Cross-Section

The method of construction must be chosen in accordance with the prevailing soil condition but may be influenced to some extent by the availability of equipment, machinery and materials. Conventional mining methods are suitable for driving horseshoe and flat arched sections, and, less economically, for circular sections whereas shield method is restricted almost exclusively to circular section. The free face method can be used for the cross-section of any desired shape. Moreover, the material of tunnel lining also influences the shape of the tunnel. Since materials capable of resisting compressive stresses only (brick or stone masonry, concrete) are limited to structures composed of arches e.g. circular, elliptical and horseshoe sections whereas materials capable of resisting tensile and bending stresses (reinforced concrete, steel) can be used for lining sections of any desired shape.

IV. Hydraulic Design of HRT

While carrying out the hydraulic design of HRT, head losses and hydraulic gradient line for HRT have to be worked out both for steady state and transient conditions. The hydraulic design includes working out the losses occurring during the flow of discharge through the tunnel and the design of lining based upon the losses [5].

A. Economic Diameter

Diameter optimization studies for the design discharge are carried out to determine the diameter of the tunnel. Designers have to select the velocity of flow of water (4-6 m/s) in a HRT. A higher velocity means more head loss and more abrasion of concrete lining. Less velocity, however, leads to a bigger diameter of HRT, which increases cost and the time of construction. The optimization analysis is based on computing the total evaluated cost for the range of tunnel diameters considered and adopting a tunnel size having least evaluated cost. Thus an economical diameter of a HRT is found by the trial-and-error approach and shall be examined for practical consideration such as space required for excavation equipment.

B. Transition Shapes

The transition shapes are required at the tunnel entry and exit in order to reduce the head losses to minimum and avoid cavitation. The length and shape of the tunnel depends upon the velocity and flow conditions prevailing in the tunnel, economics, construction limitation etc. Hydraulic model studies are conducted to determine an efficient and economical transition.

C. Tunnel Entrance and Exits

The entry into or exit from the tunnel has to be carefully designed. Since at these points the water enters or leaves the tunnel, they are prone to hydraulic head loss therefore, a proper transition shape has to be provided to keep the loss minimum and to avoid cavitation. To minimize head losses and to avoid zones where cavitation pressure may develop, the entrance to a pressure tunnel shall be streamlined to provide gradual and smooth changes in flow. To obtain best inlet efficiency the shape of entrance should simulate that of a jet discharging into the air and should guide and support the jet with minimum interference until it is contracted to the tunnel dimensions.

D. Velocity

Average permissible velocity in a concrete lined tunnel may lie between 4 and 5 m/s. For a steel lined tunnel, higher velocity can be adopted and velocity as dictated by economic studies shall be chosen. Permissible velocities in tunnels of different surfaces i.e. unlined, steel lined and concrete lined also depend upon the sediment load carried by the water. Depending upon the nature of sediment load the velocity may be varied accordingly.

V. Choice of Tunnelling Methods

The choice of tunnelling method is influenced by several factors which include the following:

1. Ground conditions: This is the main factor as it may not only influence the choice of method, but may also present major limitations on some particular methods. The following ground conditions are encountered:

(a) Soft ground: The soft ground composition consists of clays, gravels, sands, weathered rocks in various states of decomposition and presents ease of excavation particularly by hand methods.

(b) Rock conditions: Low rock strength may prove advantageous to machine excavation but require special support considerations while high rock strengths may preclude machine excavation but require minimal temporary support.

(c) Mixed face conditions: Tunnelling at bedrock (Solid unweathered rock lying beneath surface deposits of soil) horizon results in the upper part of the face being in the soil or heavily weathered rock whilst the lower part of the face is in the rock; this condition presents problems with machine tunnelling and in connection with the provision of effective temporary support.

2. Tunnel Size: Microtunnels with diameters of less than 0.9 m to full face TBM tunnels of upto or greater than 12 m diameters all require special consideration to be given to a comprehensive assessment of ground conditions; increasing tunnel size introduces changes in significance of particular tunnelling problems.
3. Environmental Aspects: Drill and blast operations can be restricted, if not precluded, in connection with tunnel drivage in urban areas; water table and ground water drainage pattern changes can result from tunnelling activities and affect the surface; other factors include problems encountered with the intermittent or continuous occurrence of water and gases from the rocks in close proximity or even remote to the tunnel.

VI. Structural Design of a HRT

The basic philosophy adopted in design of an underground excavation (tunnelling, surge tanks, power houses etc.) is to utilise the rock mass itself as the principal structural material, creating as little disturbance as possible during the excavation process. The amount of reinforcement required is worked out depending upon the behavior of the rock mass when the opening is made in the rock. The structural design is an important aspect which has to be dealt carefully one the tunnel is designed geometrically and hydraulically [1], [6], [8].

A. Rock Load and Its Estimation

The most important potential loads acting on underground structures are earth pressures, i.e. rock pressure and water pressure. It is general rule that every stress produces a strain and leads to displacement of individual rock particles. For movement a rock particle needs space. In confined state the movement of rock particles is prevented hence the stresses accumulate and reach very high values which are in excess of their yield point. By excavating the cavity, opportunity is given for deformation towards its interior. In order to maintain the cavity, the intrusion of rock masses must be prevented by supporting structures and the loads acting thereon is referred as rock pressure. The magnitude of rock pressure is greatly affected by the magnitude of deformation produced. The modern tunnel designers shall predict the (primary) stress conditions prevailing in the interior of the non-uniform rock mass (depending on the previous tectonic history) and design the tunnel for the same as well as for supporting the rock load which is likely to be developed on account of the opening made in the rock (secondary stresses). The magnitude of secondary pressures developing around the cavity is governed by a variety of factors, such as the size of the cavity, the method of its excavation, support method and bridge action period. There are numerous methods of rock load estimation based on the assumption of the formation of a natural ground arch above the cavity.

The true art in tunnelling lies in the anticipation of the development of large rock pressures, which is far more effective than to find the means of resisting rock pressures which have already developed [2].

The magnitude of earth pressures is in general independent of the strength and the time of installation of the supporting structure and it is only its distribution which is affected by the deformation of the latter however, the magnitude of rock pressure is influenced decisively by the strength and the time of installation of props. This is because deformation following the excavation of the cavity in rock mass surrounding the tunnel is of plastic nature and exceeds over a period of time. The period required for final deformations and thus for pressures to develop, generally increases with the plasticity of the rock and with the depth and dimension of its cross-section. The magnitude of deformation and consequently that of stresses can be limited by installing an efficient designed rock support system at the proper time.

B. Importance of Bridge Action Period while Deciding the Tunnel Support

The necessity of tunnel support arises from the fact that the excavated rock has a tendency to drop out of the roof of the tunnel. The time available to us for installation of the supports depends upon the bridge action period (the time between blasting and the beginning of collapse of the unsupported roof.) which may range from a few hours to few weeks. The bridge action period for cohesionless sand or completely crushed rock is almost zero. Hence, if a tunnel passes abruptly from fairly sound rock into such materials (cohesionless sand or completely crushed rock), excessive over breaks at the point of transition are inevitable. Example: An accident took place during the excavation of Chukha HRT of Chukha hydropower plant at Bhutan where a seam filled with sand and water was encountered. As soon as the rock partition between the rock and the seam was blasted, sand and water flow of the order of 30 to 50 cusecs (cubic feet per sec) was observed. If any such geological problem is encountered, proper investigation of the medium ahead of face by probe holes and thereafter its treatment through a pre-drainage and pre-grouting becomes mandatory. Therefore, it becomes necessary that geological investigations should be continued during construction. The importance of bridge action is explained below. However, if tunnel bridge action period happens to be quite large, one should not misunderstand that the tunnel needs no supports because of the fact that the rock loads go on developing for weeks and months together after the tunnelling operations are over. Example: An accident took place in the Chukha Power House, Bhutan where the cavity (25.5 m span) was left unsupported for 35 days and the accident occurred on the 36th day bringing down about 800 cu m of rock and resulting in the formation of a dome with a maximum height of about 6 m near the crown. This is due to the fact that if the cavity is left unsupported, load at the crown goes on increasing to such an extent that it exceeds the crushing strength of the rock and at that stage the crown caves in resulting in dome formation.

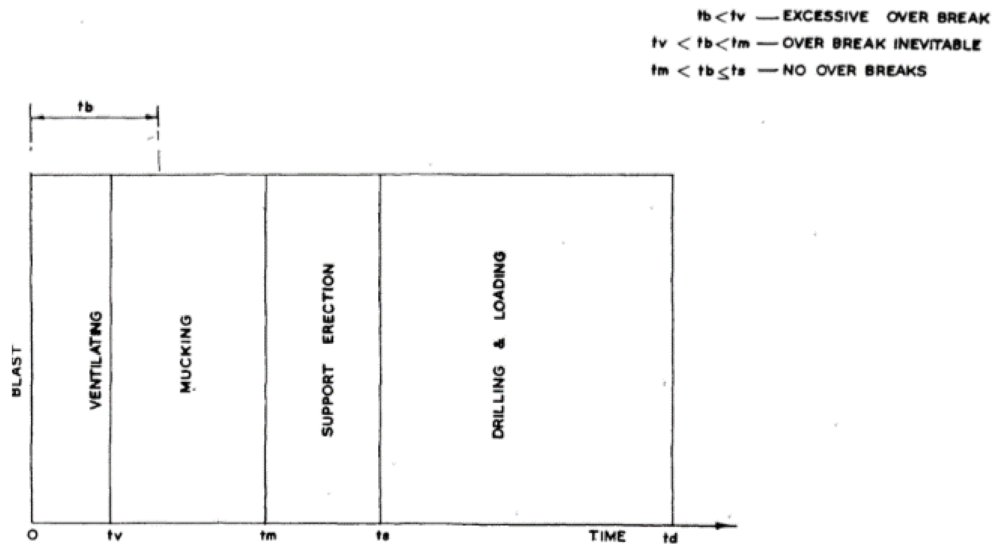

Fig. 2 Importance of Bridge Action Period [2]

C. Excavation and Rock Support System

1) Rock Reinforcement

The concept of rock reinforcement or active support in rock is aimed at limiting rock mass deformation by reinforcing the worn out rock mass in strong and intact rock mass. Such action is intended to improve the mechanical properties of the rock mass by interlocking blocks of ground or applying additional confining stresses, which in both cases aim to limit the magnitude of the rock mass deformations. This is in direct contrast to the concept of using free standings or passive supports such as steel sets where in general the support only applies an opposing stress to ground movement as a result of its own deformation by the movement it is designed to prevent. Such movement can lead to significant deterioration of the rock mass by extensive rock failure or the opening up of rock joints which can result in creating inherently unstable tunnel conditions.

D. Mechanisms of Support

There are four basic and relatively independent mechanisms by which rock reinforcement can improve the stability of a rock mass.

1. In relatively competent and well-jointed rocks, instability may be the result of individual blocks detaching from the roof when the gravitational loading exceeds their retaining load provided by shear resistance and cohesion along joints. Such bolts can be stabilised by rock bolts with an anchorage force capacity greater than the weight of the block in question.
2. In weaker blocky rock conditions, tunnel closure can occur either through blocks sliding or rotating or through rock yielding and these can be opposed by tensioned or untensioned bolts designed to maintain the shear stress along the discontinuities.
3. In laminated or stratified rocks, instability may be as a result of slip along bedding planes in addition to bed separations. Fully grouted, untensioned rock bolts can be used to preserve the inner strata shear strength and thus prevent such actions and give rise to forming a significant load sustaining reinforced beam.
4. Rock failure in overstressed rocks can be minimised by increasing the degree of confinement or the minor principal stress. In tunnelling, the minor principal stress vector acts in general perpendicular to the wall. Thus tensioned rock bolts, installed relatively quickly after excavation, can provide additional confinement and limit the extent of rock failure by increasing the allowable major principal stress.

E. Concrete Lining

Lining in tunnels is a very important component and makes up for 30 to 40^% of the total cost of tunnel [1]. Tunnels forming part of water conductor system have to be invariably lined with cement concrete-plain or reinforced or steel lined, However, in cases where a tunnel is meant for operation for short periods and where it has to be abandoned after it has served the purpose e.g., diversion tunnels the lining could be avoided.

Structural design of tunnel lining requires a thorough study of the geology of rock mass, the effective rock cover, results of in situ tests and other mechanical characteristics of the rock. The designer should assess the rock load acting on the lining and the internal pressure which is to be transmitted to the rock mass based on the result of these investigations.

The function of concrete lining is one or more of the following:

- To reduce head losses in the system;
- To protect steel ribs from deteriorating;
- To prevent leakage of water;
- To protect the turbines by preventing loose rock particles falling into the water and being carried to the turbines;

- To take that part of the internal pressure not taken up by the rock.

VII. Geological Problems during Tunnelling and Remedial Measures

Tunnelling may be associated with many problems like problems of stress, ground movement, squeezing and swelling ground conditions, heavy ingress of water, high geothermal conditions and sometimes gases apart from varied lithological and structural geological features.

A. Tunnelling Problems

Various problems are encountered during tunnelling. The various problems quoting examples from Nathpa Jhakri Hydroelectric Project (H.P.) and practically adopted solutions will be discussed [7], [9].

1) Occurrence of Geothermic (Hot Water) Zones in the HRT

During excavation of the 27.4 km long HRT of Nathpa Jhakri HE project many geological problems associated with underground excavation were encountered. One of the most difficult problems was the occurrence of geothermic zone of about 3.5 km length. A number of hot water springs occurred at places where deep acquifers have been tapped by tectonic dislocations like faults, shears and joints. Hot water with a temperature ranging from 35 °C to 55 °C was seeping and gradually increasing through the rocks and the rock temperatures were also in the vicinity of 55 °C, this started from RD 16950 onwards. The dewatering arrangements provided at that time could not tackle the quantity resulting in accumulation of water. The total discharge of water was around 100 lit/sec.

2) Consequences and Preventive Measure

a. Excavation Problem

There was high temperature and humidity inside the tunnel due to heavy seepage of hot water. It was extremely difficult to work under such conditions. The problem was further compounded by dust generated due to blasting and exhaust of construction machinery.

Preventive measures taken to reduce the effect of hot water:

1. Improving the ventilation by installing additional fan of 200KW, 180000 cum/hour capacity at the adit portal thereby increasing the amount of fresh air through the booster fans.
2. Spraying of cold water on rock inside the tunnel.
3. Providing cold water bath arrangements for workers.
4. Very short duration of work shifts i.e. 2 to 4 hours.

Moreover, before concrete lining was started, the hot water inflows were channelized by providing half PVC pipes with mortar to the rock surface and diverting it outside through the shutter with the help of a pipe, in order to achieve a rock surface as dry as possible. On completion of concreting these pipes were grouted with cement mortar.

b. Durability of Tunnel Concrete Lining

The entire HRT was to be concrete lined using RCC/PCC. It was suspected that the concrete may face long term durability problems under such high temperature of seepage water. The CSMRS (Central Soil and Material Research Station) were entrusted to access the problem and to suggest some suitable measures against it.

CSMRS during their studies found that the seepage water was of chemically aggressive nature [11]. Besides this some of the aggregates proposed to be used were suspected against alkali silica reaction. Taking all these factors into account, CSMRS/ CWC suggested the use of blended cements with low water cement ratio for concreting of entire HRT lining. Both Portland puzzolana cement with the ash content of 25% and Portland slag cement with slag content of 50% have been used for tunnel lining.

c. Precautions Taken during Initial Filling

Initial filling of water conductor system of the project at a rate of 1.0 m/hour ensure slow and uniform saturation of rock mass, thereby avoiding overstressing of rock mass due to high gradient of seepage flow. Further, the filling was carried out with suitable pressure steps with pause of 24 hours between each pressure step. As soon as the water in the HRT reached the geothermal zone, the filling rate was reduced to 0.3 m/hour. This slow rate of filling in hot water zone prevented cooling shock of the lining and also ensured that favourable cracking of the lining as anticipated by the provision of reinforcement really takes place.

3) Highly Sheared Rock with Heavy Seepage

Very poor ground conditions were encountered in the reach of tunnel between Rattanpur downstream and 6th face adit. The rocks, in the shear zone encountered, were quartz mica schist, sericite schist, biotite and amphibolite band. It is mostly rich in sericite and biotite schists with multiple small shears filled with clay and squeezing in nature. The heavy ingress and seepage of water encountered have made the shear zone material in flowing conditions. Further advancing of excavation was not possible with the available equipment. After detailed study of the ground conditions and various other solutions, the excavation of tunnel was done with DRESS (Drainage-Reinforcement-Excavation-Support-Solution).

DRESS methodology is completely flexible and effective in responding to extremely heterogeneous rock mass with large rock loads and squeezing behaviour under high water head with risk of huge inflow in unstable excavation profile and reliable of time and cost estimates. But this method is a costly alternative and all other possibilities for excavation like multi-drift method, etc. should be explored before adopting this methodology.

DRESS involved pre-drainage of ground ahead of face with 24m long drainage holes and stabilization of the crown ahead of tunnel face by steel pipe umbrella arch, up to a predetermined length, followed by excavation in small steps by mechanical means and support thereof.

Sequence of Activities Involved

The details of various steps carried out with dress methodology are described as under:

(a) Draining of the rock ahead of the face: Before opening of the face, advance drainage was done all around and ahead of the face to eliminate the detrimental influence of water pressure on face stability. Six to eight drainage holes of 77 mm diameter up to 24 m length depending on the site strata, in an upward inclination of 15 degree were drilled with a hydraulic drilling rig using DTH hammer. M.S. Pipe of 50 mm diameter 12 m grouted and 12 m perforated protected with geotextile (impervious felt) were provided in the drilled drainage holes to avoid the blockade of drainage system. These drainage holes have been provided in alternate forepoling blocks.

(b) Face improvement: After providing the drainage system, stability of the face and ahead was improved by cement grouting with W.C. ratio 1:1. Sometimes when grouting was not possible due to encountering of gougy sheared material mixed with clay the face was stabilised by shotcreting and grouted anchor bars of 25 mm diameter 8 m long.

(c) Umbrella fore poling arch: Fore poling (casing) of steel pipes was provided ahead of the face before excavation of the face using the hydraulic drilling rig. In this the crown of the tunnel above springing level was supported with 12 m long steel pipe forepoles (casing) of 114.3 mm outer diameter with 6mm thick wall and in an upward direction of 6 degree over rib R1 of the block. The forepoles were spaced @ 400 mm C/C spacing. After drilling and installing of the forepoles, cement grout in W.C ratio of 0.75 to 0.45 was placed at a maximum pressure of 5 kg/cm^2.

(d) Face advance in heading excavation: After stabilising the crown and the zone ahead of the tunnel face by forepoles, drainage holes, shotcreting and grouting, the tunnel advance in one forepoling block of 12 m length was carried out up to 8.75 m length in a variable diameter of excavation from 11.65 m to 13.45 before the next block of forepoling. In this 8.75 m length of the tunnel advance, a total number of 12 sets of ribs of ISMB 300 × 140 @ 750 mm c/c spacing were provided in a sequential advance of 0.75 m to 1.50 m depending upon the stand up time of rock strata. Rib R1 was installed at the minimum excavation line and subsequent ribs R2 to R12 were provided in a variable minimum excavation line radius to facilitate the forepoling machine working in the next block of forepoling. After protecting the crown with shotcrete/wiremesh, the excavated section was supported with ribs and the space between the rock surface and the rib intradose have been filled with shotcrete. In the second round of excavation, the central portion was first excavated up to the previous round advance and again the excavation for this round was carried out in an advance of 1.5 m in the same way as explained above. After excavating and supporting the forepoling block up to rib R12 (last rib of the block), the excavation of Rib R1 of next block was done and the rib is installed and supported with wiremesh and shotcrete. The ribs were anchored at springing level with 25 mm dia 6 m long cement grouted anchor bolts with ISMC 150 × 75 runners joining three to four sets of ribs. The face was then sealed off with shotcrete for improvement of face and ahead before start of excavation of next forepoling. The excavated reach was further supported with radial rock reinforcement in the form of 32 mm dia 6 m long hollow core self drilling cement grouted rock bolts. In such type of bolting the rock bolts are installed simultaneously with drilling having the steel bit attachment to the rock bolts at the drilling end. Grout was then pumped through the bolts itself forcing out water, debris etc. and filling of all fissures voids and complete grouting of the bolts was ensured. A temporary invert arch of 350 mm thick shotcrete was also provided to prevent heave of an unsupported invert and punching of steel ribs from the arch support into soft rock.

(e) Importance of logistics in long tunnel: The logistic supports in the tunnel such as, proper ventilation, dewatering and lighting, etc. are the lifeline for the speedy tunneling [10]. These supports become much more important when the tunnel is excavated through water charged mediums and long tunnelling from one face. As the human body cannot function properly in the absence of any organ, similarly speedy tunnelling cannot be executed in the absence of any logistic. Example: Tunnelling (10.6 km HRT) in Dulhasti H.E. project, on river Chenab, in J&K, remained a bottleneck due to poor logistics like ineffective ventilation and managing huge discharge of water inside the tunnel and frequent occurrence of geological problems like encounter with the shear zones gradually slowed down the progress of the tunnel which could have been easily tackled with proper planning of the logistics. Seeing such problems following recommendations are made:

1. Long tunnelling (more than 2.5 km) from one face is to be avoided. If unavoidable, then another alternative such as ventilation through intermediate shaft, at a suitable location, should be planned.
2. Systematic monitoring of pollutants level in the air, air flow (m^3/sec) and air speed (m/s) are mandatory to improve the system.
3. In water charged medium, sufficient gradient of tunnel should be provided for under gravity drainage. Simultaneously, a side drain is necessary for effective drainage and to save the invert.
4. If any geological problem is encountered, proper investigation of the medium ahead of the face by probe holes and thereafter its treatment through pre-drainage and pre-grouting becomes mandatory.
5. As far as possible minimum pumping stations are to be made and rehandling of water is to be avoided.

VIII. Conclusion

It has been observed that tunnels bring the project on critical path. Designers have to give proper consideration to the design standards and come to a final design only after detailed study of the topographical, geological conditions prevailing on the site. Further, geological exploration prior to the planning, detailed geological investigation parallel to planning should be done and to be continued during the construction in order to ascertain whether the driving method adopted is correct or needs to be modified based on the prevailing conditions.

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