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# ENGINEERING OF HEADRACE TUNNEL FOR NEELUM JHELUM HYDROPOWER PROJECT

ENGR. ABDUL KHALIQ KHAN

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By

**Engr. Abdul Khaliq Khan**[1]

## Abstract

*Neelum Jhelum Hydropower Project is located in Azad Jammu and Kashmir with diversion dam on Neelum river near Nauseri and the power station near Chatter Kalas on lower limb of Jhelum river providing a gross head of 420 m on the turbines. The headrace tunnel is 28.5 km long and the installed capacity of the project is 969 MW.*

*A Norwegian firm completed the feasibility study and detailed design of the project in 1997. The construction of the project commenced in January 2008.*

*During the process of review of earlier design, it was established that major design changes were needed mainly due to the occurrence of historic high earthquake in October 2005 near Muzaffarabad and consequent up-gradation of seismic design parameters.*

*Delays in the scheduled progress of works caused by various site problems prompted WAPDA to consider deployment of Tunnel Boring Machine (TBM) for excavation of headrace tunnel so that the project could be completed within the contemplated schedule. Use of TBM has resulted in some further design changes. This paper discusses the engineering of design changes for the power tunnel of the Project from its planning phase to the present stage of construction.*

## 1. INTRODUCTION

Neelum river originating from glaciers in the vicinity of Kaobal Gali flows westward and joins Jhelum river at Domel near Muzaffarabad. Downstream of Domel the Jhelum river takes a sharp southward turn and then continues south upto Mangla reservoir.

Neelum river has steep gradient such that a tunnel from Nauseri on Neelum river upto Thotha on Jhelum river (upper limb) could provide a head of over 200 meters for generation of electric power. A second tunnel from upper limb of Jhelum river to its lower limb near Chattar Kalas could provide another head of about 200 m for a second Power Station. It was this concept under which a feasibility study was carried out by WAPDA during 1985-87.

The 1987 Feasibility Report had envisaged three interlinked projects. Firstly the flows of Neelum river were to be diverted to Jehlum river near Thotha through a 19.50 km tunnel to generate 500 MW power in phase-1. The combined flows of Neelum and Jhelum rivers were then to be diverted through 9 KM tunnel from Thotha to Kohala to generate 1000 MW power in phase-II. In third phase a dam at Dudhnial on Neelum river was proposed which would regulate Neelum river supplies and enhance power generation.

[1] Advisor to WAPDA on Mega Projects

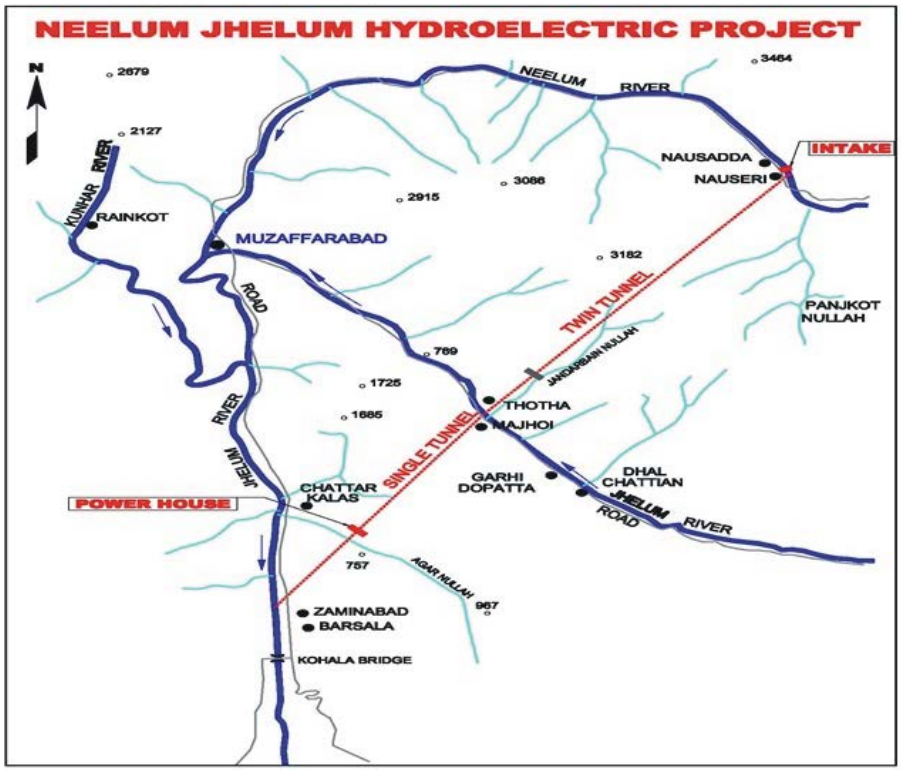



*Figure 1: Project Layout showing Neelum and Jhelum Rivers*

## 2. BRIEF HISTORY

The PC-I Proforma for the project was approved by the Federal Government in December 1989.The project could not be started for several years due to non-availability of funds. In November 1994, WAPDA deputed a Norwegian consortium of consulting engineers M/s Norconsult and Norplan for up-dation of feasibility report and preparation of detailed designs for the project. They issued upgraded feasibility in 1996 and detailed design report in 1997.

In the very early part of their assignment the subsurface investigations at Thotha revealed highly problematic foundation conditions for the powerhouse. The powerhouse at Thotta was located in the impact zone of Hazara Thrust Fault and there was no better location in the near vicinity. The feasibility scheme was therefore modified into a single dam and intake, a continuous 28.5 km long headrace tunnel and a single powerhouse. This scheme involved longer construction period but promised more power. A few years later on October 5, 2005, a major devastating earthquake occurred near Muzaffarabad. It had a magnitude (Mw) of 7.6 and was located on the Balakot-Bagh Thrust Fault (BBT) commonly known as Hazara Thrust Fault and the epicenter was about 13 km from Nauseri and 33 km from the site of the powerhouse. WAPDA therefore asked the project consultant, M/s Neelum Jhelum Consultants (NJC) to revisit the seismic hazard.

## 3. SEISMIC HAZARD

The Norwegian Consultants had determined that the potential source of severe earthquake is Murree Fault / Main Boundary Thrust (MBT) for the diversion dam and intake works, and the Jhelum Fault for the powerhouse area. They reported that neo-tectonic fieldwork by that time

had not revealed any evidence of recent fault breaks in the dam site area. Therefore they adopted the following PGA values for an Operating Basis Earthquake (OBE).

- Surface installations 0.25g (for rock sites)
- Underground works 0.17g

The Norwegian consultants in their 1997 report state that they commonly use a return period of 200 years for design earthquake. The ICOLD Bulletin 72 suggests that the ground motion with a 50% probability of exceedance in 100 years should be used for determination of seismic design factors.

With this scenario M/s NJC got a fresh study made for seismic hazard and the PGA values for OBE were upgraded as follows:

- Surface installations 0.34g
- Underground works 0.25g

This change had large bearing on previous designs. Consequently the following major design modifications were carried out:

a) Type of dam changed to a composite body with under-sluices and spillway on left side of MBT and earth-core rockfill embankment over the Fault and on its right side.

b) Deeper foundation to place the dam on less weathered better rock.

c) Stilling basin added below the Spillway to dissipate energy.

d) Outlet of Diversion Tunnel placed farther downstream clear of the stilling basin exit.

Review of the 1997 design also led to the following additional modifications:

e) Upstream coffer dam body changed from earthfill to concrete which will save annual rebuilding after washout in high floods.

f) The upstream part of headrace tunnel had been provided a dip to create storage volume for peaking power. Daily filling and emptying of this part of tunnel involved risk of tunnel collapses. The dip was therefore eliminated and the dam height raised by 2.5 m to provide the required storage.

## 4. HEADRACE TUNNEL LAYOUT

### Horizontal Alignment

Shortest alignment of headrace tunnel is the straight line route from the Intake structure at Nauseri to the Powerhouse cavern on right side of Agar Nala near Chattar Kalas (see figure 2). The Intake location is fixed by the dam axis about 300 m upstream of the confluence of Nausadda Nala with Neelum river where valley topography and abutment rocks are both favourable.

Location of Powerhouse cavern is decided by sound rock environment, strike direction of major joints, appropriate depth of rock cover and suitable tailrace tunnel or channel back to river. The length of headrace tunnel on this alignment is 28.45 KM.

The headrace tunnel starts from point T0 at Nauseri ending at point T6 at its junction with penstocks near Agar Nullah. The upstream part for 11.45 KM was designed as twin tunnels

each with a cross-section of 46 $m^2$. The remaining headrace down to the penstocks was designed as single tunnel with a cross-section of 80 $m^2$. The design report of 1997 had proposed that tunnel should be excavated by drill and blast (D&B) method and that shotcrete lining would be adequate. The tunnels will have concrete invert and the parts of the tunnel in weak zones will be fully concrete lined. This basic scheme was generally retained in the construction project with some changes as discussed in later paragraphs.

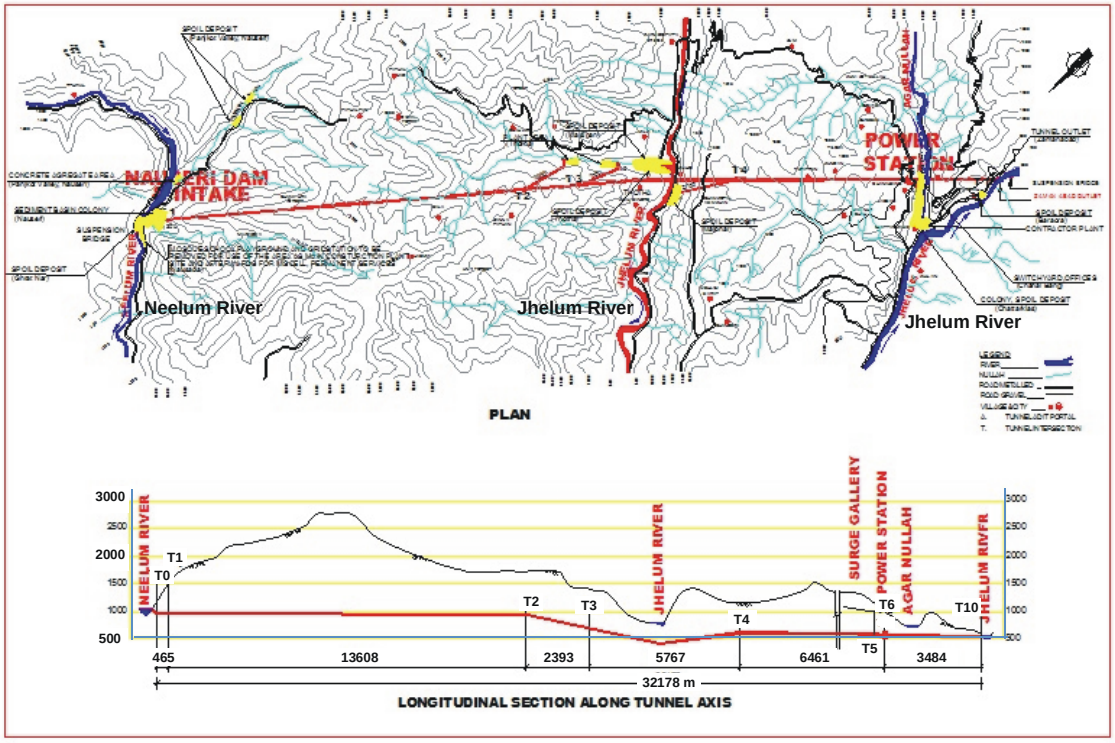



*Figure 2: Plan and Section of Headrace Tunnel as given in the Feasibility Study Report*

## Vertical Alignment

In vertical alignment the tunnel was provided a low point in the upstream part by making inclined shaft down from El.989.5 to E. 965 and then taking tunnel gradually upslope to El 995. It was proposed that this dip would provide additional storage for meeting daily needs of peak time power. Another dip was provided at Jhelum river crossing for a different purpose i.e. to make it safe against risk of hydrojacking.

## Layout Details

The headrace tunnel starts at point T0 at the downstream end of the sedimentation basins. Adit A1 provides construction access to headrace tunnel from Nausadda Nala. The point T1 where access adit meets headrace is set at 450m downstream of T0 at EL 995.0m. The location and elevation of T1 was set to minimize the access adit length.

Subsequently the adit A1 portal was relocated, but the T1 junction was kept unchanged. The changes include moving the portal to a lower elevation and changing the angle of the T1 junction for easier access to the downstream part of the headrace tunnel excavated from A1.The movement of the portal to a lower elevation reduced the inclination of the access tunnel.

The section T3 – T4 alignment is mainly governed by the Jhelum River crossing. The headrace tunnel crosses under the river at EL 400.0m, allowing approximately 380m cover to the tunnel. On each side of the crossing the tunnel ascends at a slope of 1:10. After subsequent reviews and detailed discussions, as described in para 12, this dip was eliminated. The point T6 on the downstream side of headrace tunnel at entry into Penstocks was fixed at El.594. The total horizontal length of T3-T31-T32-T4 is 5.74 km (see Fig. 3).

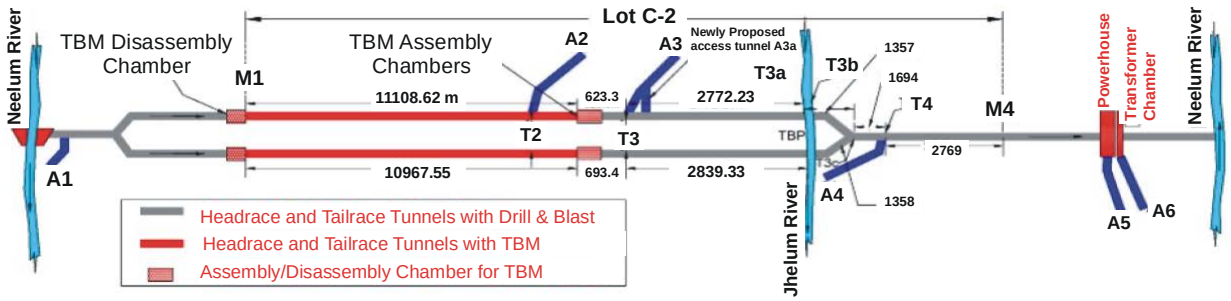



*Figure 3: Modified Plan of Headrace Tunnel*

The T31 to T32 section, of length about 1500m, was proposed to be excavated as twin tunnels. Twin tunnels were chosen because the tunnel will cross the Muzaffarabad fault (Bagh - Balakot Thrust) and shorter diameter of the twin tunnels will be safer construction. The presence of Jandarbain nullah was utilized to provide additional access for construction through Adits A2 and A3. Another access was provided through Adit A-4 from left bank of river Jhelum.

The A3 and A4 adits were planned to be sealed off, with a bulk-head plug at T4 and a concrete plug at T3. This plan was later modified to address the requirements of TBM induction in the upstream reach.

The Jhelum river crossing was designed (1997 report) as a deeply submerged tunnel so that it may function as a pressure tunnel with concrete lining. The alternative to a submerged tunnel was considered a shallow tunnel, steel lined in a stretch of 1200-1500m. A cost/benefit comparison between the two alternatives was performed and the deep submerged tunnel was found economical. In the construction stage the two options were rigorously discussed among the Client, the Engineer and the Contractor resulting in the adoption of shallow crossing with steel liner.

## Critical Stretch of Tunnel for timely completion

The length between T1-T2 is 13.5 km and represents the longest tunnel stretch between any two working faces in the Project. Despite its excavation from two fronts the construction time would still be on the critical path for the completion of the Project. A further consideration was that the tunnel between T1 and T2 will have the highest overburden and will therefore experience the highest rock stresses. An alternative alignment to bend the tunnel around the high peaks was also studied but only a slight reduction in rock overburden could be achieved. The designed straight tunnel alignment was therefore considered optimal solution.

## Alternatives for Jhelum River Crossing

When designing a cost effective headrace tunnel two main objectives should normally be met:

- In order to enable emptying of the tunnel by self-draining, the tunnel should be given a continuous downward slope to tailrace.
- For unlined tunnels, the overburden must be sufficient to safeguard against hydraulic splitting. The rock stresses built up around the tunnel must therefore be higher than the water pressure in the tunnel.

The topographic conditions for river crossing at Thotha are such that both these objectives cannot be met at the same time. A fully self-drained tunnel has low overburden and must be partly steel lined. Steel lining can, however, be avoided for a deeply located tunnel which, on the other hand creates a water lock.

The alternative with the deep crossing was selected by the Norwegian Consultants, for of saving cost on steel liner required for shallow crossing. The construction challenges and operation problems were not duly evaluated. Further review is discussed under para 12.

## 5. GEOLOGY

The surface cover of the proposed headrace tunnel is largely concealed by thin to thick layers of Quaternary deposits. Rocks exposed at various outcrops over and around the proposed headrace tunnel exclusively belong to Murree formation. The two geologic materials exposed in the project area the quaternary deposits and bedrock, are briefly described below:

### Quaternary Deposits

The Quaternary deposits contain upper Pleistocene to Recent alluvial, colluvial and terrace deposits. These are tectonically less disturbed and generally form only a thin cover above the headrace tunnel alignment. None of these materials are deep enough to reach tunnel vicinity.

### Bedrock

The tunnel alignment passes through Murree formation which comprises alternate beds of sandstone, siltstone, mudstone and shale.

### Sandstone

Sandstone can be subdivided into SS1 and SS2. The SS1 sandstone is generally strong, well cemented, and fine to medium grained. The sandstone is thinly to thickly bedded, at places massive and blocky, moderately to closely jointed and at places fractured. In general, the thickness of sandstone outcrop between Nauseri and Thotha ranges from 5 to 20m. The SS2 sandstone is usually reddish brown and appears to be gradually changing to finer materials which include thin siltstone and mudstone beds.

### Siltstone

Siltstone is grayish brown to reddish brown, sandy at places, strong to medium strong and intermixed with mudstone and shale. In general, the thickness of siltstone between Nauseri and Thotha ranges from 3 to 6 m while between Thotha and Agar Nullah it ranges from 0.3 to 2 m.

### Mudstone

Mudstone is reddish brown, weak to medium strong, and slightly to moderately weathered. In general, the thickness of mudstone between Nauseri and Thotha ranges from 1.5 to 4 m. In downstream parts of the tunnel the beds are slightly thinner.

### Shale

Shale is dark to reddish maroon, weak sheared and fissile in nature. It occurs in thin beds at places. In general, the thickness of shale is 0.5 to 1 m.

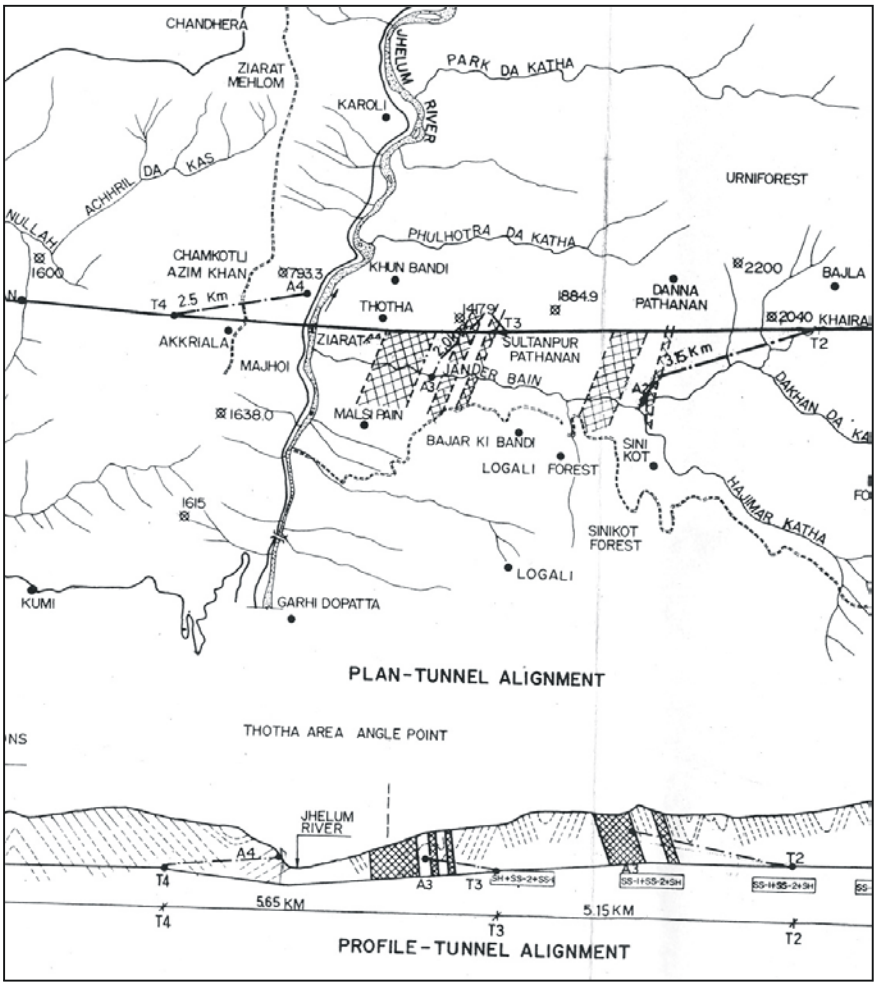



*Figure 4: Geological Plan & Cross Section between T2 and T4*

## 6. ROCK PROPERTIES

### Rock Mechanics Parameters

An unlined pressure tunnel was proposed for the project in the 1997 report. The design is based on the principle that the final rock support design is done concurrent with the excavation as the in-situ rock mass is revealed. The global stability of the pressure tunnels is taken care of by locating the tunnels in rock masses where the minimum stress is higher than the internal water pressure.

It was assessed in the 1997 report that the global water control, i.e. control of water losses from the tunnels towards the surface during operation, could be achieved by the inherent low permeability of the rock mass and by the high phreatic level and by grouting as required in areas with high permeability. Control of significant water ingress during excavations will be achieved by detection and treatment of water leakage ahead of the face by exploratory drilling and grouting.

For some early evaluation of rock mass characteristics a few tests were conducted on rock samples taken at surface. The majority of tests, however, were later carried out for representative material obtained as cores from deep drill holes.

The UCS values, based on point load test results, indicated that low range of elastic modulus is in the range of 5 to 10 GPa for intact rock. Based on Dwarshak Dam experience it was estimated that there will be a certain increase in low modulus due to the effect of the alternating layers of strong sandstone, SS-I. The rock mass modulus for SS-II/SH was therefore assumed to be 3.0 GPa and the modulus for SS-I as 10 GPa. In order to evaluate the effect of the rock mass properties on a circular tunnel with lining, the standard charts for circular tunnels could be used. NATM charts were used for maximum effective overburden. Maximum effective overburden was assumed to be in the order of 1200 meters. This is about 2/3 of the topographical overburden. The estimated reduction is based on the experience that sharp peaks and ridges will reduce overburden pressure at depth.

### Rock Stresses

Rock stresses in the rock mass at depth were measured for the Feasibility study by use of hydraulic testing in exploratory boreholes in the Thotha and Agar Nullah areas. The report from SINTEF describes that the tests did not create any new fractures and caused only jacking of existing fractures or bedding planes. Most of the tests showed a “shut-in-pressure” higher than theoretical vertical stress. All values were found to lie between 150% and 85% of theoretical gravitational stress.

The objective of the stress measurement is to optimize the tunnel lining design. Hydraulic fracture testing yields reliable data on the minimum principal stress which is an essential input for pressure tunnel design. The method was considered reliable in deep drill holes. The holes were located in selected areas considering the risk of hydraulic splitting of the rock mass.

Further verification of in-situ stresses was recommended to be carried out at each critical location during construction. This is now practiced. The objective is to determine the exact location of the tunnel alignment in critical areas (Fault Zones) as well as the location of bulkheads in access tunnels and the transition between unlined and steel lined water tunnels.

## 7. DRILL AND BLAST TUNNELING

The excavation procedure should always be adapted to the local rock mass conditions. Excavated rock is supported by rock bolts as a general practice. When traversing weakness zones or other areas of heavily jointed or otherwise incompetent rock, immediate shotcreting, pressure grouting spiling or other support measures may be required.

In low quality rock mass short rounds and subdivision of rounds are used. The objective is to obtain an opening with “stand-up time” long enough to allow the installation of necessary support.

Smooth blasting was specified to reduce overbreaks and control surface undulations. This method is based on closely spaced contour holes, with reduced charges to be fired.

### Design Principles

The tunnels should be designed with an “economic” shape aiming to satisfy the combined requirement for tunnel usage, rock support, friction loss reduction and convenient construction. For drill and blast tunnels a horseshoe shape and tunnel width approximately equal to tunnel height was adopted.

The tunnel size should be determined by considerations of tunneling costs and loss of head due to friction. With the prevailing rock conditions in Neelum Jhelum project the major aspect in designing the tunnels was to apply adequate means of tunnel support. The major part of the

tunnel approximately 90% will be supported primarily with bolts and shotcrete. The remaining part, approximately 10% where adverse rock conditions are met (Q5, and partly Q4) will be heavily supported with composition of bolts, thick layers of shotcrete, rib girders and concreting.

In the feasibility study fully concrete lined tunnel was compared with tunnels being partly lined with shotcrete and concrete lining. The study concluded to adopt the combined shotcrete/concrete lined solution.

### Rock Supports

Rock was classified Q1 for strongest to Q5 for weakest. Q3 and Q4 classes are the most dominant classes along the waterway, representing around 85 % while class Q1 rock masses are not expected to occur. For each Q-class corresponding rock support resources (RS) were defined.

The tunneling costs and time consumption calculations were made for various rock and rock support systems to optimize the tunnel sections.

In order to reduce to the risk of damage to the shotcrete due to erosion and wear, the maximum water velocity was set to approximately 3.5 m/s. Accordingly the minimum excavated cross-section for single tunnel was set to 82 $m^2$ and for twin tunnel, it was 43 $m^2$.

## 8. HYDRAULIC DESIGN OF TUNNEL

The gross head at the turbines was taken as 417 m and tunnels designed to pass a discharge of 280 $m^3$/sec with average water velocity of 3.5 m/s.

All waterway tunnels were designed with 0.2 m concrete invert, which provides lesser loss of head and ease in maintenance.

The walls and roof of the tunnels are either supported by concrete lining or by a minimum of 100 mm steel fibre reinforced shotcrete.

Concrete lining was to be applied only where adverse rock conditions are met, and estimated at about 10% or 15% of the tunnel length.

### Head Loss Due to Friction

The head loss was calculated using the Manning formula. The Manning's 'n' values used in the 1997 report were as follows:

n = 0.0133 for concrete face.

n = 0.0185 for shotcrete face.

For shotcreted sections, where walls and roof are shotcreted while the invert has concrete, an average Manning figure was calculated for the actual tunnel shape.

Head loss due to friction was calculated with the actual net waterfilled cross-section, where the following assumption was made:

- Lining structure 0.5 m and 0.6 m inside theoretical excavated contour for twin tunnel and single tunnel respectively.
- Shotcreted walls and roof 0.15 m outside theoretical excavated contour and concrete invert 0.2 m inside.

Head loss was calculated for concrete lined and shotcreted sections and added up with the occurrence of concrete lined and shotcreted sections along the tunnel portions as follows:

| | |
|---|---|
| T0 - T2 = 21.9 m | T3 – T4 = 7.2 m |
| T2 - T3 = 2.7 m | T4 – T6 = 7.1 m |

Total head loss due to friction in the headrace tunnel was calculated to be 38.9 m. Adding 3.9 m loss in tailrace and 0.9 m at transitions junctions and bends the net total head loss was determined 43.7 m.

M/s NJC considered that the roughness factor n = 0.0185 for the tunnel shotcreted surface was optimistic and equates to values generally used for Tunnel Boring Machine (TBM) excavated tunnels, which are much smoother than the drill and blast construction called for in the construction contract. It was apprehended that the design dimensions of the tunnels adopted in the tender design would result in an estimated 27% reduction in available energy. To ensure the planned generation capacity it was proposed that either the excavation size be enlarged or the entire tunnel be concrete lined to reduce head losses.



*Figure 5: Shotcreted surface inside access Adit A5*
*Note the irregularity in the pattern of over-cuts at crown*

## 9. SINGLE OR TWIN TUNNELS

As a general rule, a single tunnel is normally chosen since both the excavation volume and the loss of head due to friction increase with multiple tunnel system conveying the same flow. The advantage of having more than one tunnel is the reduction in construction time and lesser geological risks from smaller excavated cross-section.

- The first part of the waterway represents the crossing of the Murree formations between Nauseri intake and Jandarbain Valley. Due to topographical constraints, about 11 km of tunnels are to be excavated between adit A2 in Jandarbain Valley and the adit AI at the Nauseri intake. The reduced construction time with a twin tunnel configuration would be beneficial for this portion of the waterway.
- For the remaining parts of waterway (approximately 21 km between upper Jandarbain (T2) and the outlet) the topographical conditions facilitate

establishment of construction adits and hence six (6) excavation fronts. The average length of tunnel excavation fronts is slightly more than 3 km. From an economical point of view, a single tunnel waterway was the right selection for this part.

The twin tunnel configuration for the T1-T2 portion reveals slightly higher total costs but it was recommended for two main reasons:

- The mountain formations between Nauseri and Jandarbain have high overburden and will yield high rock stresses. The choice of two tunnels instead of one gives a reduced tunnel size which represents a lower geological risk when crossing these formations.
- The single tunnel has 2.5-3 years longer construction time.

It was noted that even after selection of a twin tunnel configuration, the T1 – T2 section still remains on the critical path in the overall construction schedule.

For twin tunnels to progress faster than a single tunnel it was necessary to deploy two complete sets of excavation and lining equipment. Alternatively provide cross-passages at about 200 m – 250 m centers so that drilling and excavating plant could move quickly from one face to the other. At a later stage of construction in a bid to accelerate works the tunnel boring machine (TBM) was deployed which changed the Drill and Blast (D&B) requirements.

## 10. SHOTCRETED VERSUS CONCRETE-LINED TUNNELS

The thickness of overburden above the tunnel in section T0-T2 is very large. The ground water table would be located very high above the tunnel. The pore water pressure in the rock mass around the tunnel can be expected to be higher than the water pressure in the tunnel itself. This means that a significant hydraulic gradient towards the tunnel should be expected. The magnitude of this gradient depends on the permeability conditions in the rock mass around the tunnel.

The change in water pressure in the tunnel due to power peaking and reservoir level changes will not be very abrupt. The pore pressure in the rock mass around the tunnel will adjust accordingly at a slow or moderate rate. In the Detailed Engineering Design report 1997 the draw down rate was limited to 20 m per hour, to safeguard the tunnel and its support structure against unacceptable external water pressure.

With the specified thickness and flexural and tensional strength of steel fibre reinforced shotcrete, cracking and disintegration due to water pressure fluctuations and water flow was not to be expected. If conditions indicating risk for erosion in clay/silt zones in the rock mass are encountered during tunnel excavation, proper sealing and possibly pressure grouting should be considered in order to limit the amount of water flow into/out from the tunnel. Experience about actual convergence and the corresponding effect on the shotcrete, will form the basis for deciding on the best design and scheduling of the shotcrete.

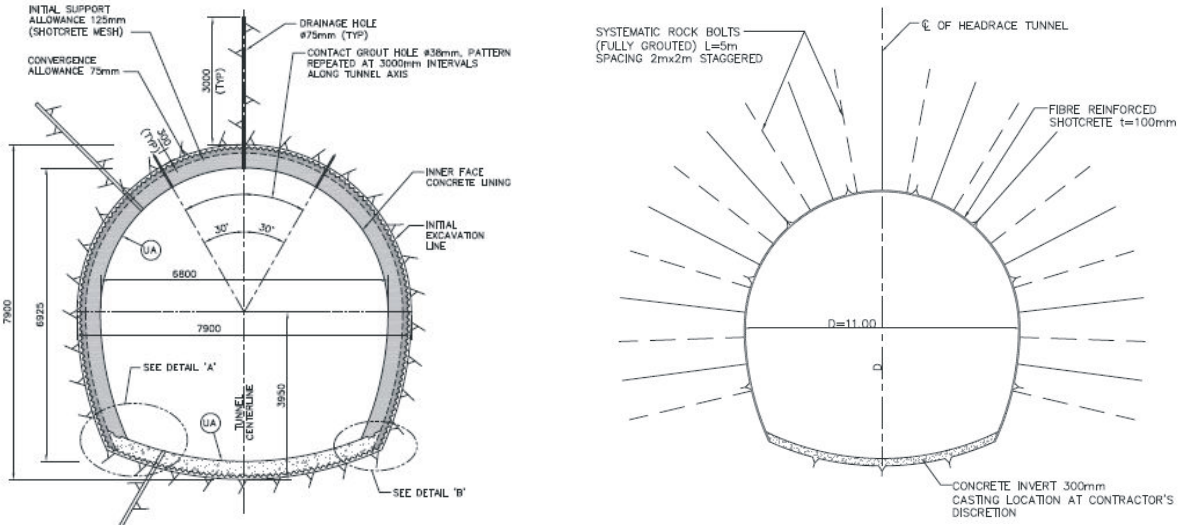



*Figure 6: Headrace Tunnel Typical Cross Sections - Concrete (left) & Shotcrete (right)*

## 11. TUNNEL BORING MACHINE (TBM)

### Feasibility Report Assessment

The advantages of using a TBM include the following;

a) Continuous operation and higher advance rates.

b) Less rock damage and less support needs.

c) Greater worker safety and cleaner environment.

Disadvantages of a TBM are a fixed circular section, longer mobilization time and higher capital costs.

The use of a TBM was considered in the 1997 Report and found attractive on parts of the tunnel between Thotha and Nauseri. The main reason behind this assessment was higher anticipated excavation rate, the possibility of omitting access Adit A-2 and some road construction to the Upper Jandarbain valley. Preliminary cost estimate indicated that partial use of a TBM on the project might be a viable alternative. The preliminary estimate was based on double shield type machine. It was presupposed that the TBM tunnel will be lined with a segmental concrete lining over its entire length.



*Figure 7 : Gripper Type Tunnel Boring Machine*

The critical parameter for a TBM-tunnel is the available stand-up time when crossing rock mass of low quality. If the rock mass deforms fast enough to exert significant pressure on the shield the TBM may get stuck. The rock stand-up time has an exponential relation between unsupported span at the face and rock mass quality. A large machine is slower than a smaller one and hence prone to get jammed.

The net progress rate of an 8 m machine was estimated to about 425 m per month.

The TBM scheme was planned with the following tunnel arrangement:

- From T3 two TBM tunnels will be driven upstream, following the same vertical alignment as for the base case.
- From T1 two tunnels will be driven by drill and blast methods.

This scheme offered the advantage of reduction of construction time for the upper part of the headrace tunnel.

A thickness of 0.5 m was assumed for lining segment and rock support, giving following cross-section:

- Excavated diameter: 7.8 m $A = 47.8\ m^2$
- Inner diameter: 6.8 m $A = 36.3\ m^2$

The computations given in the 1997 Report showed that the TBM was more costly than the drill and blast tunnel therefore, for the feasibility design only the drill and blast solution was recommended.

The project implementation scenario subsequent to the project start indicated about 12 months delay during the first 24 months of construction period. The Engineer estimated project completion by end 2017 instead of the contractual date of 31 October 2015. Studies were made to accelerate the works and it was generally accepted that a part of the Headrace Tunnel should be excavated by TBM and the balance tunnel works as well as other project works at Weir site and Powerhouse caverns should be accelerated and their activities be rescheduled to adhere to the original project completion date.

The11 km of TBM excavation from T3 at Thotha will balance the 5.2 km of drill and blast excavation from T1 at Nauseri. This will result in 5.0 years total tunneling construction time from adit to adit (A3-T3-T1-A1). Compared with the base case approximately 1.5 years tunnel construction time could be saved.

## Contractor's Proposal

In September 2009 the contractor submitted a report proposing use of tunnel boring machine (TBM) to speed up the tunnel excavation work.

Two important factors that determine whether the TBM is economical and reasonable are the compressive strength and the quartz content of the rock. If these two factors are too high, the cutter will be worn much faster and the time for changing the cutter will increase, which will not only increase the cost but also affect the advance rate.
At Neelum Jhelum project we have medium-strength rock strata and low quartz content. The compressive strength of SS-I at the project is around 150 MPa, that of SS-II is around 70 MPa and shale is less than 15 MPa, which is feeble and easily weakened and disintegrated. The quartz content is anticipated to be lower than 20%. Therefore, it is suitable to use TBM for construction of headrace tunnel in this project.

The tunnel construction of about 4 km with the conventional drill and blast method had amply shown that with this pace and methodology the project will certainly be delayed. The project had already suffered delay of about one year on account of issues relating to land acquisition, slow mobilization of contractor, law and order situation and unreliable power supply. The TBM technology has greatly advanced since the project design in 1997. Keeping in view the urgency and importance of the project it was decided to deploy two boring machines and accelerate the construction works.

## TBM Design Aspects

Main types of TBM are single shield TBM, double shield TBM and main beam TBM, among them, double shield TBM and main beam TBM are commonly used on large size tunnels. The geologic conditions at section T1-T2 indicate that the machine will have to pass through zones of large convergence and high gush of waters. A main beam TBM with front shield and back side grippers was therefore selected. The (selected) Herrenknecht TBM has been designed with special equipments and tools to deal with extreme advance conditions. Success will depend on skilled management of operations giving due time and importance to geologic monitoring and taking intelligent decisions.

A series of Design Meetings were held with M/s Herrenknecht the manufacturer of TBM, during April to November 2011 and following major decisions were taken;

- TBM shall have Main Drive, Cutterhead, Grippers and front shield.
- Size of the assembly chambers for the two TBMs shall be 100 m x 22 m x 20 m and 60 m x 22 m x 20 m.
- Boring diameter shall be 8.53 m.
- Total power demand of two TBM's is 18 MW.
- Drilling depth of roof bolts shall be > 3.9 m.
- Ring beams of adjustable type TH 44 @ spacing 1.0 m.
- Grouting pump capacity 1.5 $m^3$/h @ 100 bar pressure.

The length of tunnel excavation by TBM between T2 – T1 was adjusted to 11.2 km. It was also decided to use precast invert segments for the tunnel invert.

Some foreseen construction problems were examined by the Contractor as follows.

The middle parts of tunnel section T2 – T1 pass under high mountain with buried depth reaching maximum about 1800 m. Consequently there could be very high stress field around the excavated tunnel. Freshly cut areas near tunnel are particularly prone to suffer from rock bursts in SS-1 rocks and large squeezing in SS-II/Shale zones. Sandstone rocks acting as aquifer can develop gush of high pressure seepage water along their contacts with shales placing heavy demands on drainage pumps.

### Rock Bursts

Rock burst is related to the stress and the lithology of surrounding rock. Controlling the stress condition of the surrounding rock and enhancing the ability of plastic deformation, can achieve the aim of controlling the rock burst. Reducing the speed of cutter head, enhancing the Cutter torque and adjusting the tunnel speed help to prevent the rock burst or reduce the scale of the rock burst. Following measures can be taken;

- Release the terrestrial stress by pre-drilling, or injecting water into the hole.
- After excavation, sprinkle high-pressure water to the working face and the nearby rock mass of the tunnel wall immediately to reduce the strength of the rock mass, improve the plasticity, and reduce the intensity of the rock burst.

### Very Weak and Squeezing Ground

The cutterhead can allow 10 mm radial convergence. For conditions of larger convergence, replace the gauge cutter and extend the reamer cutter by hydraulic structure mechanism. Cut the tunnel in an enlarged diameter of 250 mm and then retract the cutterhead 20 cm backwards. During 6 hour maintenance shift keep the cutterhead rotating for several minutes every two (2) hours to cut the converged rock mass and thus avoid TBM getting stuck.

### TBM Support against Weak Tunnel Wall

On the overcut/collapsed area place arch steel plate with rib filling concrete in the cavity. Set the gripper pads over the steel plates.

### High Pressure Water Gush

For the tunnel section passing through zone of high external water pressure, the TBM tunneling will adopt the “reverse advancing method”. For the gushed water ample drainage and plug can be combined together but drainage should be given the priority. If gushing water appears, the submersible pumps installed behind the TBM head should be immediately started. When there is too much gushed water where TBM cannot advance, the drilling/pre-grouting would be adopted.

To handle the above problems, the most efficient method is to enhance the geological predictions.

### Measures for Geological Predictions

The best treating measure against poor geological area of TBM tunneling is to perform advance prediction and forecast. The short-range forecast is to analyze and forecast the condition of frontal surrounding rock based on the conditions of the exposed surrounding rock and the rock pieces on the belt, the changing of the tunneling parameter and the vibration of TBM etc. Exploratory drilling probe is used to obtain information on geological conditions ahead.

The medium range forecast can apply BEAM method (Bore-Tunneling Electrical Ahead Monitoring).The theory of measuring is to test the apparent resistively of the rock strata to get the knowledge of rock mass quality as well as the cavities and the water in the rock. The system can forecast the geological condition and the characteristic of the rock of the tunnel face within 30-50 m accurately (see Figure 8).

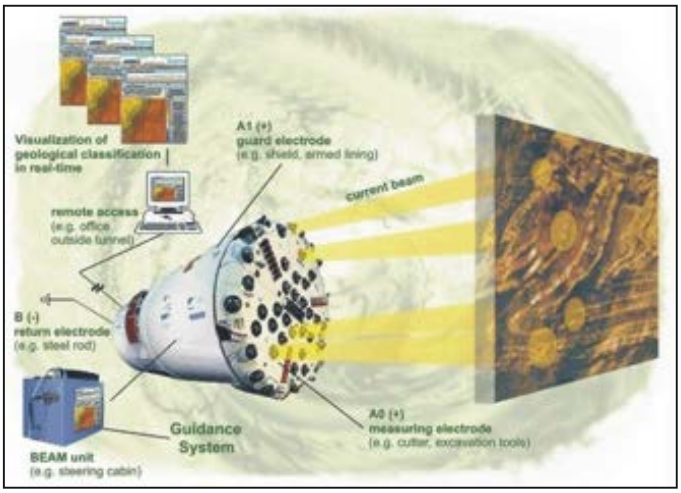



*Figure 8: BEAM Method of Monitoring*

For the long-range prediction and forecast of the geological conditions, geological radar method and Tunnel Seismic Prediction (TSP) method can be used. These two methods both can accurately forecast the geological conditions and changing of the rock character around 150 m ahead of working face. The theory is to receive and treat the geological forecast by reflecting the signal of seismic wave (see Figure 9).

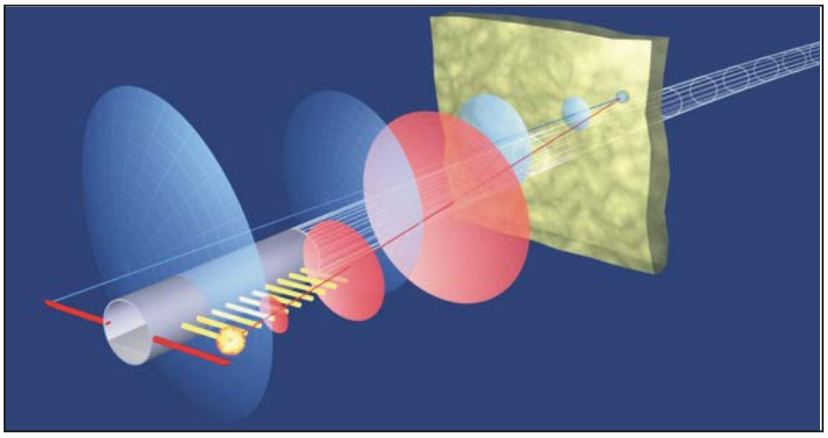


*Figure 9: TSP Method of Monitoring*

## 12. JHELUM RIVER CROSSING

The feasibility study had proposed that Headrace Tunnel should cross the Jhelum River at a minimum depth which provides safety against hydrojacking. For this purpose the tunnel alignment was given 1 in 10 downward slope from both banks. This layout created a midway low point at El.400 that would in turn establish a water lock of about 400,000 $m^3$. During tunnel inspections in the future the tunnel could be dewatered under gravity except the tunnel dip area. It was proposed in the 1997 report that the trapped water volume be pumped out before every such inspection. Dewatering scheme proposed use of two mobile sets of submersible pumps mounted on trailer and tractor assembly. Review of this proposal brought out following concerns;

- Drill and blast tunnel going down slope under a river involves geological risks. Dewatering would be colossal task.
- With the potential back pressure on the shotcrete lining, the lining may fail along with rock spalling and accumulation of debris.
- The scheme involves extended outages of power station.

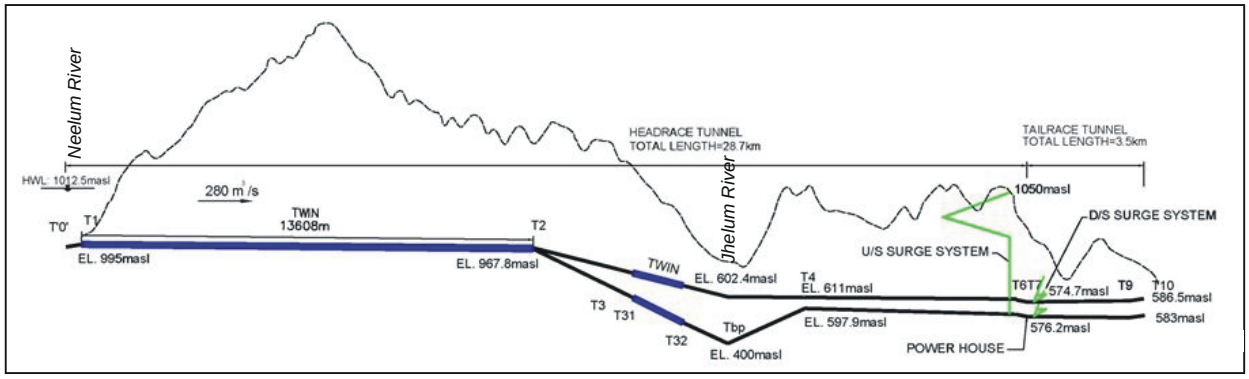



*Figure 10: Section at Jhelum River crossing showing both deep and shallow options*

Alternative to the deep crossing would be shallow crossing following the specified slope to downstream side. The favourable and unfavourable points of both schemes were considered as follows;

### DEEP CROSSING

### Favourable Points

- Same concrete liner as balance of tunnel with some additional reinforcement.
- Lower Risk of hydro-jacking and better factor of safety.
- No need of variation order.

### Unfavourable Points

- Increased tunnel length and higher mucking and haulage effort. Higher seepage during construction under high hydrostatic pressure. Drainage will need more energy.
- Greater head loss as compared to shallow alignment.

- Not self draining. Deep-Well multistage pumping required. Extra manpower, time and electricity required for dewatering.
- Inspection and maintenance of the dip areas will be difficult task. Debris and sediment deposit in lower part of dip.
- During dewatering and inspection the power plant is out of operation resulting in loss of income.
- Powerhouse shutdown time during inspections is higher – 6 to 8 weeks.

## SHALLOW CROSSING

### Favourable Points

- Inspection and maintenance can be performed with other sections of the headrace tunnel.
- Self draining tunnel. Drainage needs less energy.
- Only one debris trap is needed before powerhouse.
- Tunnel cost is less because of less excavation volume. Mucking and haulage is easy.
- Powerhouse shutdown period during periodic inspections is lesser – 3 to 4 weeks.

### Unfavourable Points

- Major Concern for shallow crossing is hydro-jacking.
- Requires reinforced concrete lining involving extra cost.
- Requires additional 800 m steel lining.
- Liner cans do not fit into present access Adit A4.There is need to re-profile the adit or make new access.
- Risk of delay in steel procurement and cost escalation.

In the discussions with the Consultants and the PoE the following items were pointed out for special attention;-.

a. In the case of rather homogenous stress condition in the rock mass the deep crossing satisfies the hydro fracturing safety factor of 1.5 whereas the shallow crossing has a safety factor of less than 1.5.

b. Due to the proximity of the Muzaffarabad fault zone significant tectonic stresses will be present in the zone of the shallow and deep crossings. These stresses are unknown and it will be very difficult to define a related stress field, which will be altered in the case of future earthquakes. As a consequence it must be assumed that even for the deep crossing hydro fracturing is possible locally.

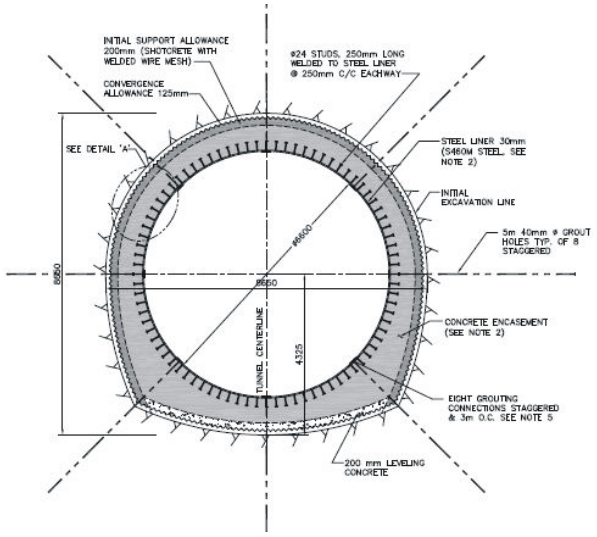



*Figure 11: Cross Section Steel Lined Headrace Tunnel under Jhelum River Crossing*

c. Hydro fracturing is usually confined to the near-field of the tunnel's cross-section. It cannot be excluded that locally lateral pressure coefficients may vary from zero beyond one.

d. A reinforced concrete liner will also be required for the deep crossing considering poor rock and low subgrade reaction parameters.

After several technical meetings, it was decided that we should build a safe project requiring only reasonable O&M efforts and hence shallow crossing was adopted.

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