



## *Stress Measurements by Hydraulic Fracturing for Design of Steel Liner for Head Race Tunnel – A Case Study*

*Rajbal Singh<sup>a,\*</sup>, A.K. Dhawan<sup>b</sup>, Ranjodh Singh<sup>c</sup>*

*<sup>a</sup>Central Soil and Materials Research Station, New Delhi, India*

*<sup>b</sup>Fugro Geotechnical Group, Gurgaon, India*

*<sup>c</sup>Energy Infratech Pvt. Ltd., Gurgaon, India*

*\*Corresponding author: [rajbal.singh@nic.in](mailto:rajbal.singh@nic.in)*

### **ABSTRACT**

Some serious leakage problems have occurred in the rock mass around the underground structures of hydroelectric projects where irregular topography had given rise to inadequate confinement of high pressure water conductor systems. The problem was overcome through adequate rock mass testing during construction stage by checking and ensuring sufficient magnitude of in-situ stresses. Head race tunnel for Nathpa Jhakri hydroelectric project in the Himalayan region is a high pressure tunnel which encountered low cover reach of 9 m where steel liner of 8.5 m diameter was provided. Location of start of steel liner had undergone considerable change in comparison to that fixed initially based on the minimum rock cover criteria. The locations of start and end of the steel liner in the head race tunnel were fixed finally on the basis of in-situ stress measurement using hydraulic fracturing method. Results of in-situ stress measurements at four locations for the start of steel liner and at two locations for the end of steel liner were used to reach at the final decision. In addition to confidence generated with reference to safety aspect, in-situ stress measurement during construction saved lot of money and time, which were always crucial for the successful execution of a project.

**Keywords:** Head race tunnel; Steel liner; Stress measurement; Hydraulic fracturing

### **1. INTRODUCTION**

The hydraulic fracturing test is one of the most common methods adopted for evaluating in-situ stresses inside rock mass. The hydraulic fracturing tests are less time consuming, easy to conduct and can be conducted at many locations inside the drill hole to get an overall view of the in-situ state of stress. Further, several alternate locations can also be chosen for the design of engineering structures in rock mass.

In the Himalayas, immense water potential exists for the generation of hydropower. Due to environmental considerations, run of the river schemes are becoming popular for the construction of hydroelectric projects as compared to reservoir schemes. The majority of the components of these projects are underground structures.

The 1500 MW Nathpa Jhakri hydroelectric project was constructed across the river Sutlej, down stream of the existing Bhaba underground power station (120MW) in Kinnaur and Shimla districts of Himachal Pradesh state in India. The project was commissioned in the year 2003 and has been

successfully operating since then. A 60.5 m high concrete gravity dam along with four underground desilting chambers (525 m long, 16.3 m wide and 27.5 m high each) were important components of the project. The project has a 10.15m diameter and 27374 m long head race tunnel (HRT) terminating in a 21.6 m diameter and 301 m deep surge shaft; underground powerhouse with a cavern size of 222 m length, 20 m width and 49 m height housing six Francis type turbine units of 250 MW each with a total installed capacity of 1500 MW for utilizing a design discharge of 405 cumecs with a rated head of 428 m; a 10.15 m diameter tail race tunnel for conveying the water back into the river Sutlej. The layout plan of the project is shown in Fig. 1.

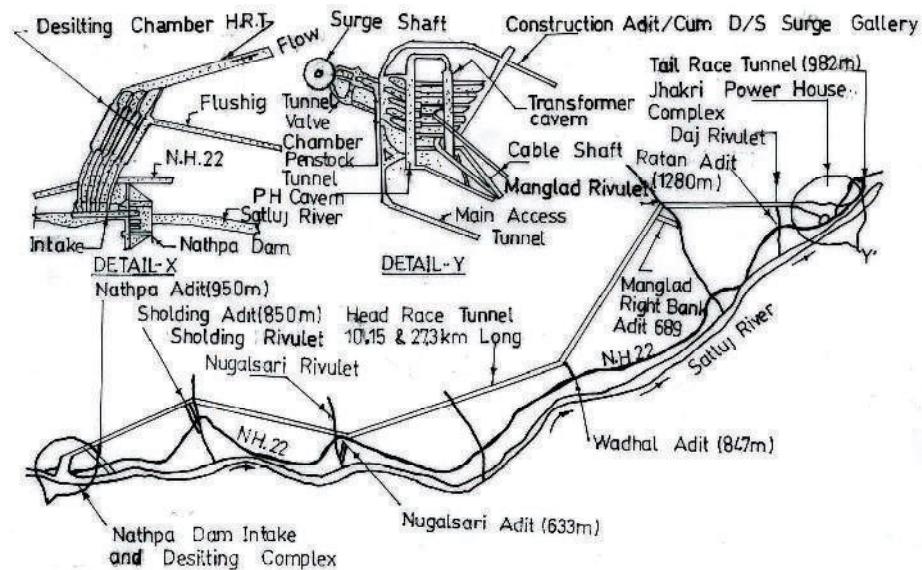


Fig. 1 - Layout plan of the Nathpa Jhakri hydroelectric project

In view of the fact that the rock cover was less than the required thickness in certain reaches of the tunnel under reference, and also based on the available geological features, it was proposed to go for steel liner of 8.5 m diameter at two locations in the doubtful reaches of head race tunnel.

With the above background, the Central Soil and Materials Research Station (CSMRS) was approached by the project authorities to verify whether the in-situ state of stress inside the tunnel was more than the required hydrostatic pressure of up to 3.08 MPa in these reaches so as to decide whether steel liner was required for the entire reach under reference. The CSMRS conducted hydraulic fracturing tests at six locations in drill holes at different depths using Minifrac System and verified the requirement of the project.

This paper covers the details of hydraulic fracturing tests, the use and importance of these tests to reduce the cost and time aspects in fixing the length of steel liner in the head race tunnel of hydroelectric project based on in-situ stress measurements.

## 2. HEAD RACE TUNNEL (HRT)

The total length of HRT from desilting chambers to surge shaft was 27374 m. There was a drop of 183 m in the tunnel invert from starting point to surge shaft. The tunnel grade along the alignment varied in different reaches from 1 in 61.30 to 1 in 500 excluding transitions which were much steeper. The tunnel with 10.15 m diameter was concrete lined except in the reaches of Manglad and Daj creeks totaling 1085 m where it was steel-lined with 8.50 m internal diameter. The thickness of concrete lining varied from 40 cm to 60 cm and it was unreinforced in the initial 17 km length

except small reaches of low cover zones or poor rock. The downstream reach of the tunnel was reinforced in view of hot water zones, high internal pressure and poor rock quality. The thickness of steel liner varied from 30 mm to 40 mm and stiffeners were also provided against buckling due to external water pressure. The transitions from concrete lined tunnel diameter of 10.15 m to steel lined tunnel diameter 8.50 m were also steel-lined.

The rock cover along the HRT varied from 90 m to 1480 m except at Manglad creek where it was only 4 m (total cover of 9 m including 5 m overburden of weathered and river borne material). The internal pressure in the HRT varied from 23 m to 216 m at beginning and end respectively under static condition. The internal pressure equivalent to rock cover in the steel-lined reaches of HRT varied from 201 m to 220 m under static condition and from 277 m to 308 m in dynamic condition.

### **3. GEOLOGICAL DETAILS FOR LOW ROCK COVER REACHES**

The Geological Survey of India has brought out the geological features along the tunnel alignment through surface mapping aided by photo interpretation and sub surface exploration. Sub surface explorations were carried out both through drifts and drill holes. The engineering properties of rock were determined by in-situ testing in drifts and drill holes aided by laboratory tests. The rock mechanics in-situ testing and most of the laboratory tests were conducted by CSMRS, New Delhi.

#### **3.1 Geology across Manglad Creek**

Rock encountered in this reach was quartz mica schist, which was rich in muscovite with some stretches of sericite schist and chloritic schists. The foliations were highly crenulated with some quartz veins even showing puckering. The rock was shattered with brick shaped pieces and even sugar cube structured where South-West dipping joints were closely spaced.

The head race tunnel just below the Manglad creek passed through quartz mica schist with a total cover of about 9 meters out of which 5 meters was overburden. Bed of the stream was composed of medium to large boulders resting on fine sand. The longitudinal section along with geology encountered in the steel liner reach of HRT below Manglad creek is shown in Fig. 2. Foliation joints spaced at 10-20cm have longer continuity and were undulating slightly rough/smooth and coated with chlorite at places with bands of biotite schist. Other joints sets were sheared joints with chlorite clay and crushed rock with thickness varying from 2-5 cm. These joints were spaced between 0.5 m to 2 m. A thick shear zone with filling more than 30 cm was also encountered in the tunnel below Manglad creek.

### **4. NECESSITY OF STEEL LINER**

A steel liner is installed where:

- It is required to control leakage out of HRT because of unfavourable geological condition;
- There is insufficient rock cover to withstand the internal pressure within the tunnel such that a potential for undesirable leakage exists because of hydraulic jacking along horizontal or near horizontal joints;
- Wherever the internal water pressure exceeds the minor principal stress in the surrounding rock mass such that potential for hydraulic fracturing or hydraulic jacking exist usually along vertical or near vertical joints.

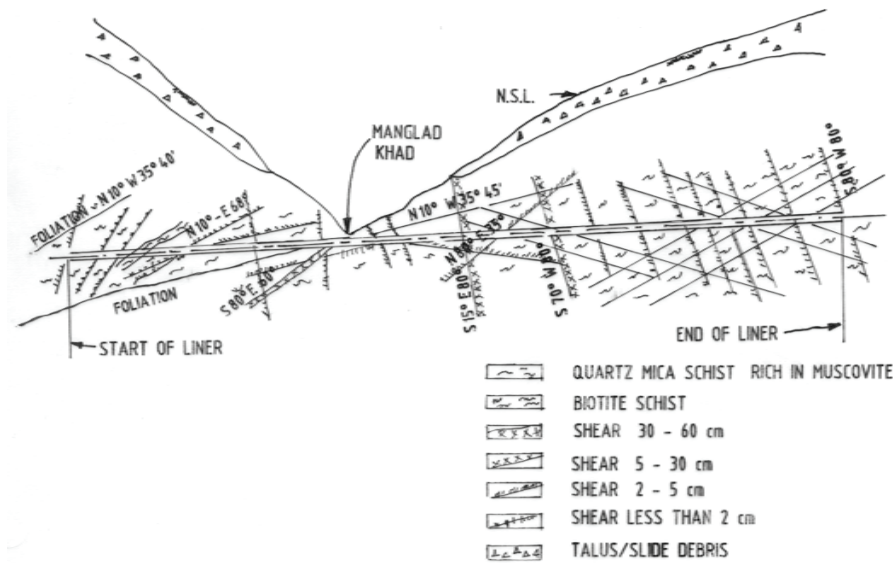


Fig. 2 - Geological section through HRT at Manglad creek (*khad*) for installation of steel liner

The tunnel reach passing through Manglad and Daj creeks low rock cover zones was subjected to a high internal pressure of up to 3.08 MPa during dynamic condition. This paper covers only the Manglad creek. The hydraulic jacking or uplift was expected to develop within a rock mass if water pressure imposed was greater than the in-situ compressive stress. Depending upon the deformability of the rock mass, and the area over which the hydraulic pressures act, existing joints could be opened. This might have resulted in jacking of a large mass of rock away from the tunnel with excessive leakage and large-scale landslides or instability. Many hydroelectric projects have suffered due to this phenomenon with disastrous consequences. When such failures occur it often takes many months to diagnose the problem and complete the repairs.

Inadequate confinement from irregular topography contributed to leakage problems, which developed at the Chivor 2 project in Colombia and the Bath County project in the United States. Hydraulic jacking will occur in any direction where movement of rock mass can develop due to a lack of adequate compressive in-situ stress. Thus vertical lifting of horizontally bedded rock; jacking or hoisting of rock masses towards valley walls; jacking of rock blocks into adjacent underground openings; or opening of fractures in a compressive rock mass can occur.

Many failures have occurred due to a deficiency in vertical cover. However, where the tunnels approached valleys, and either frontal or lateral cover was low, or where geological conditions resulted in low stresses, failure had occurred. Particularly, the problem of low lateral stresses near valleys had resulted in many failures due to an inadequate length of steel liner. The designers recognized the problem and the tunnel was either set back from the valley or reinforced concrete was used to span the section. However, reinforced concrete is not necessarily an appropriate solution in preventing hydraulic jacking. In some case this only affects the timing of the problem, as the pressure tends to build more slowly due to the decreased seepage through the reinforced liner. Accordingly, it was considered essential to provide steel liner in low rock cover reaches of HRT at Manglad and Daj creek zones.

## 5. DESIGN CRITERION

To withstand the internal hydrostatic and hydrodynamic loads, the length of the liner was arrived at based on the criteria of hydraulic containment, and quantity and reach of seepage flow from the

tunnel. The requirement of hydraulic containment or prevention of hydraulic jacking was ensured by conducting in-situ hydraulic fracturing tests in the vicinity of the proposed start and end points of the steel liner, initially fixed as per Norwegian criterion for preliminary design as discussed by Benson (1988). The second criterion has been satisfied by ensuring that the hydraulic gradient from the start to end of the liner to the nearest exit point was low enough to prevent instability of soil or rock at the point of exit. Acceptable hydraulic gradient for massive hard or widely jointed rock mass is 10-15 whereas for moderate to weak rock mass or moderately jointed rock the hydraulic gradient is 5-7.

For preliminary design, the length of the liner was fixed as per the following Norwegian criterion i.e. steel liner should extend up to a point where minimum rock cover is available (Fig. 3):

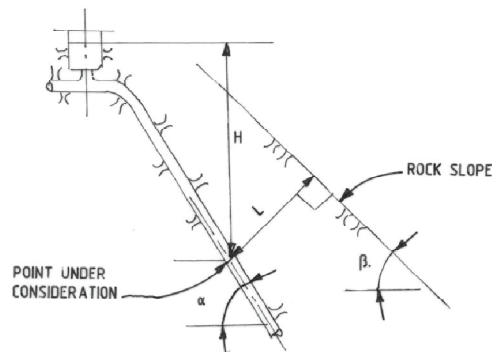


Fig. 3 - Norwegian criteria for steel liner

$$L > \frac{H \gamma_w F_s}{\gamma_r \cos \beta} \quad (1)$$

where,

L = shortest distance between rock surface and point under consideration,

H = normal water maximum pressure head at point under consideration,

$\gamma_w$  = density of water,

$\gamma_r$  = density of rock,

$F_s$  = factor of safety, and

$\beta$  = slope angle.

For preliminary design, value of the factor of safety were adopted as 1.8 against static head and 1.3 against maximum upsurge in comparison to 1.3 and 1.1 respectively, recommended as per Norwegian criterion. Conservative values were adopted for preliminary design since the geological conditions in creek area were not favourable. It was desirable to be somewhat liberal in fixing the length of steel liner and the same was extended by 10-15 m beyond the transitions. Rock cover and the length of steel liner fixed as per the above-mentioned criterion for Manglad creek is shown in Fig. 4.

## 6. NEED FOR IN-SITU STRESS MEASUREMENTS

Provision of steel liner in low rock cover zones of a pressure tunnel is a process which begins in the design but does not end until construction is complete and the geological conditions are known in detail. During the tendering stage, the liner design must be considered to be preliminary. As construction proceeds and the geological conditions become known and the requisite tests completed, the liner length can be fixed suiting to the actual conditions. The start and end of the

steel liner in Manglad area were accordingly fixed as per preliminary design for taking action for procurement of steel.

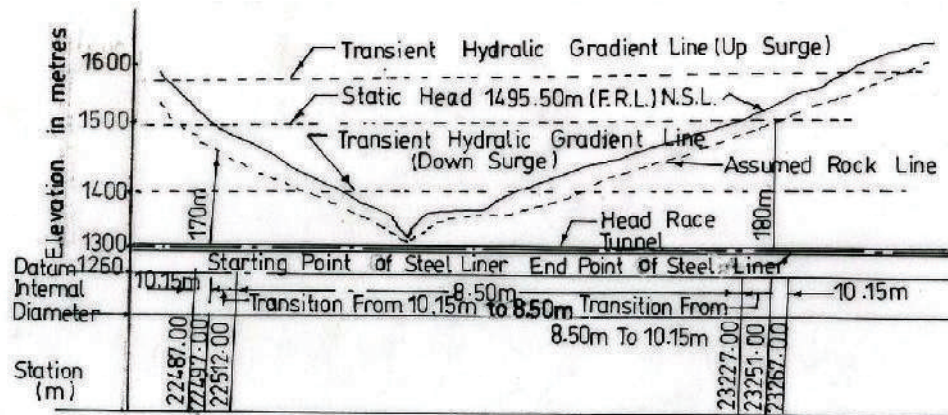


Fig. 4 - Steel liner at Manglad creek according to Norwegian criterion

The steel liner for the construction of HRT was required at this particular location because of the rock cover of only 9 m. The steel liner was proposed between Reduced Distances (RD's) 22487 m and 23267 m. The existing entrance (adit) to HRT was located at RD 22537 m.

For the start of the steel liner at this tentative location, it was proposed to excavate another 10.15 m diameter and 140.39 m long entrance adit for the installation of steel liner in addition to the existing one, through which the construction work in HRT was going on in full swing. The excavation and construction of this by pass adit, was, however, avoided after the in-situ stresses were measured at four locations at this end. The hydrostatic pressure was expected in the order of about 3 MPa inside the tunnel. Hence, the minimum stress around the tunnel lining must be more than 3 MPa to avoid the installation of steel liner or to avoid hydraulic fracturing in the rock mass. Based on the geological features, the designers fixed the locations of the start and end of steel liner tentatively and proposed to conduct the hydraulic fracturing tests to finalize the actual length of steel liner.

## 7. HYDRAULIC FRACTURING TEST

The MINIFRAC hydraulic fracturing equipment was used for determination of in-situ stresses. This equipment can be used inside an "Ex" size (38 mm diameter) drill hole up to a depth of about 30 m with an operational pressure of 35 MPa as mentioned by CSIRO (1991).

The equipment consists of installation rods, fracturing tool, impression packer, hoses, pumps etc. Installation rods and fracture tool can handle hydraulic fluid up to a maximum pressure of 35 MPa. The fracturing tool and impression packer has a diameter of 36 mm and length of 45 cm. Hydraulic fluid is delivered by a hand pump from tank to the measurement and control instrument through flexible hose and installation rods which in turn transfer pressure to the tool and packers. The hydraulic fluid comprising 10-20:1 ratio of clean water and water soluble oil (cutting oil), is used as fluid for applying pressure.

Hydraulic Fracturing technique has distinct advantages as discussed by Enever and Walton (1987) and Amadei and Stephenson (1992), over other methods of stress measurements. Based on his experience Enever (1993) presented the case studies of in-situ stress measurements by hydraulic fracturing in Australia. The results of hydraulic fracturing stress measurements have been discussed in detail by Singh et al. (1998), CSMRS (1996) and Singh (2008). The basic elements of hydraulic fracturing tests are shown in Fig. 5.

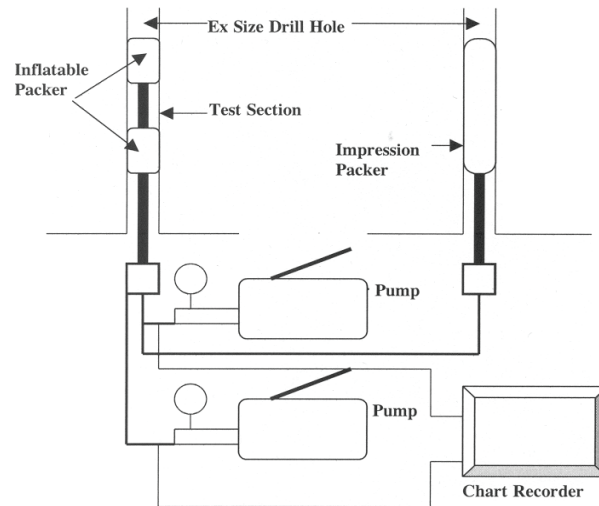


Fig. 5 - Schematic diagram of hydraulic fracturing equipment

The hydraulic fracturing involves applying hydraulic pressure to a drill hole wall for determining the fracture pressure and hence the stress. The method essentially consists of:

- selecting test locations after inspecting the drill hole rock cores,
- isolating the test section with the help of packers,
- pressurising the test location to obtain a fracture in the rock,
- obtaining a pressure time record,
- obtaining the impression of the fracture on an impression packer, and
- interpretation of magnitude and direction of in-situ stresses.

### 7.1 Test Procedure

The field test procedure involves an initial fracturing followed by a number of cycles of re-pressurisation. Initial fracturing is accomplished by increasing the pressure in the test section along with increasing pressure in the packers. Pumping is stopped as soon as a crack is formed (sudden drop in test section pressure) in order to preserve the initial geometry of fracture (as recorded by the impression packer) and to allow a first shut-in pressure to be recorded on the chart recorder. A relatively slow pressurisation rate is employed to ensure that the initiated crack retains its initial geometry during the first shut-in phase. Water with soluble oil was used and found to be the most appropriate and convenient test fluid.

The test interval is vented to atmosphere between cycles of pressurisation to allow the induced crack to close. A build up of pressure in the test interval upon temporarily sealing the system during venting is taken as evidence of continued flow of fluid out of a closing crack. Venting is continued until this phenomenon ceases. Further cycles of pressurisation and venting are used to determine the crack re-opening pressure and to gauge whether the orientation of the crack changes as it is propagated. Thus records of pressure versus time are obtained during the tests. The pressure record from hydraulic fracturing test with main features of test cycles is plotted on chart recorder.

The impression of the crack is taken on the impression packer by lowering it in the drill hole at the test location. The pressure is applied in the impression packer taking into account the orientation of the lowering rods as well as the fracture initiation/re-opening pressure. The direction of the crack is marked on the impression packer and recorded on the record sheet. This direction of crack gives the direction of maximum and minimum stresses in a plane perpendicular to the drill hole axis.

## 7.2 Interpretation of In-Situ Stresses

The fundamental principle underlying the application of hydraulic fracturing is that

- one of the principal in-situ stress components is co-axial with the test hole,
- long-term shut-in pressure is approximately the magnitude of the smaller (in-situ) horizontal stress component, and
- crack will generally tend to initiate in a plane normal to minimum (in-situ) stress (i.e. parallel to maximum stress).

In the case of an approximately axial fracture in a hole, the test pressure record can be used to estimate the magnitude of secondary principal stresses in the plane normal to the drill hole axis. The magnitude of maximum secondary principal stress component for impermeable rocks is determined from the expression:

$$\sigma_1' = 3\sigma_2' + S - P_i - P_o \quad (2)$$

where,

$\sigma_1'$  = maximum (in-situ) secondary stress,

$\sigma_2'$  = minimum (in-situ) secondary stress,

S = fracture strength of the rock,

$P_i$  = fracture initiation pressure, and

$P_o$  = ambient pore pressure.

The magnitude of minimum secondary principal stress is equal to the shut-in pressure ( $S_i$ ). Therefore,

$$\sigma_2' = S_i \quad (3)$$

The fracture strength, S can be found from the following expression:

$$S = P_i - P_r \quad (4)$$

where,  $P_r$  is the fracture-reopening pressure.

Neglecting  $P_o$  term, as the ambient pore pressure can normally be assumed to have been dissipated in close proximity of an underground opening, Eq. 2 can be rewritten as follows with the help of Eq. 4:

$$\sigma_1' = 3\sigma_2' - P_r \quad (5)$$

$P_i$ ,  $P_r$  and  $S_i$  can be obtained from pressure-time record.  $S_i$  is calculated by the double tangent method as shown in Fig. 6.

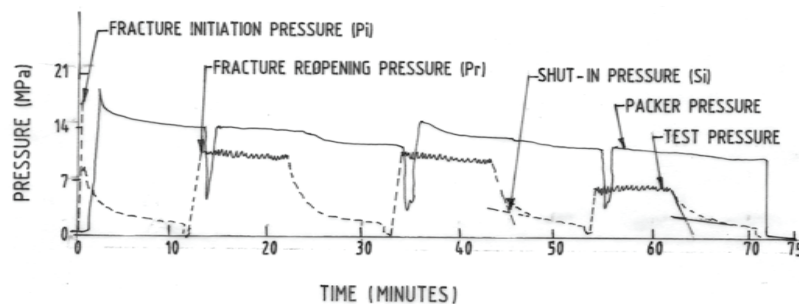


Fig. 6 – Pressure-time record of hydro fracturing test



Directions of the stresses are determined from the impressions of the cracks obtained on the impression packer. Directions are obtained with reference to the direction of the drill hole.

The vertical stress ( $\sigma_v$ ) can also be estimated from the overburden by the following equation:

$$\sigma_v = \gamma h \quad (6)$$

where,

$h$  = height of overburden, and

$\gamma$  = average density of rock mass.

## 8. RESULTS AND DISCUSSIONS

### 8.1 In-Situ Stress Measurement

The steel liner for the construction of HRT was required at this particular location near Manglad adit because of the rock cover of only 9 m as shown in Fig. 3. The steel liner was proposed between reduced distances (RD's) 22487 m and 23267 m. The existing entrance of Manglad adit to HRT was located at RD 22537 m as shown in Fig. 7. The bypass adit which was proposed to be constructed for the installation of steel liner is also shown in Fig. 7.

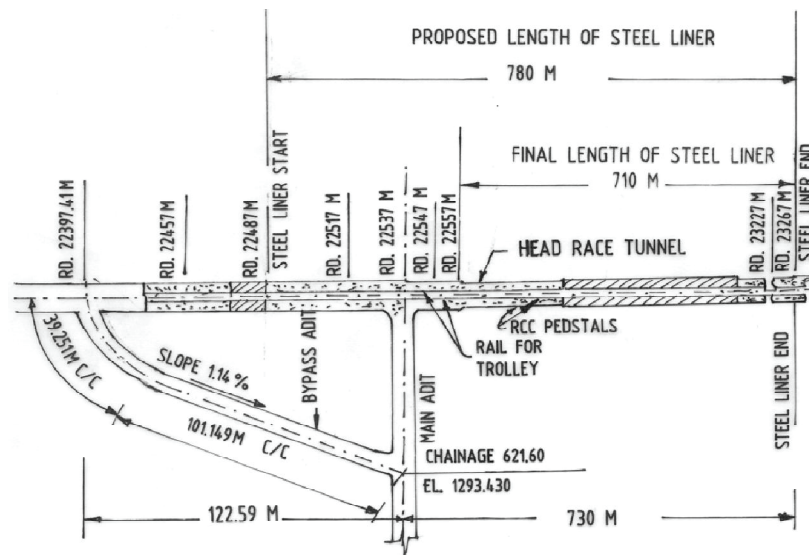


Fig. 7 - Proposed bypass adit and HRT at Manglad junction for the installation of steel liner

For the start of the steel liner at this tentative location, it was proposed to excavate another 10 m diameter and 140.39 m long entrance adit (Fig. 7) for the installation of steel liner in addition to the existing one at RD 22537 m, through which the construction work in HRT was going on in full swing. In-situ stress measurement were undertaken to fix the location of start and end points of the steel liner exactly as discussed by Singh et al. (1998). As the tests were conducted when proposal for the excavation of bypass adit to RD 22397 (Fig. 7) was still under consideration.

The hydrostatic water pressure was expected in the order of 3.08 MPa inside the tunnel. Hence, the minimum (in-situ) stress around the tunnel lining must be more than 3.08 MPa to avoid the installation of steel liner.

The hydraulic fracturing tests were conducted inside the Ex size (38 mm diameter) drill holes at six locations. Four locations at RD's of 22457 m, 22487 m, 22517 m and 22547 m for the start of steel liner (Table 1) and two locations at RD's of 23227 m and 23267 m for the end of steel liner (Table 2) were selected for the measurement of in-situ stresses as shown in Fig. 7. In all, twenty nine hydraulic fracturing tests were conducted inside twelve drill holes at different depths.

Table 1- Measured in-situ stresses at the start of steel liner

Test no.	Test depth (m)	Maximum stress (MPa)	Minimum stress (MPa)
<i>RD:22457 m, Right wall, Horizontal drill hole</i>			
1	13.55-14.00	5.17	3.79
2	14.12-14.57	3.96	3.62
3	15.00-15.50	2.76	2.76
<i>Left wall, Horizontally inclined drill hole (at 70° towards u/s side)</i>			
4	21.20-21.65	6.55	4.48
5	26.20-26.65	14.49	8.62
<i>RD:22487 m, Left wall, Horizontal drill hole</i>			
6	19.05-19.50	15.87	8.97
7	16.05-16.50	7.59	4.83
<i>Right wall, Horizontal drill hole</i>			
8	12.50-12.95	3.12	2.76
9	11.30-11.75	4.67	3.28
<i>RD:22517 m, Right wall, Horizontal drill hole</i>			
10	13.90-14.35	6.21	4.14
11	19.65-20.10	5.51	3.79
<i>Right wall, Horizontally inclined drill hole (at 45° towards u/s side)</i>			
12	27.00-27.45	15.52	7.93
<i>Right wall, Vertically inclined drill hole (at 35° downward)</i>			
13	15.50-15.95	8.63	5.52
14	20.33-20.80	8.97	5.86
<i>RD:22547 m, Right wall, Horizontal drill hole</i>			
15	13.55-14.00	11.04	6.21
16	15.15-15.60	6.90	4.83
17	19.45-19.90	6.90	4.48
18	26.10-26.55	11.73	5.55
<i>Left wall, Horizontal drill hole</i>			
19	10.10-10.55	5.52	4.48
20	13.10-13.55	5.17	5.17

As can be seen from Tables 1 and 2, the scatter of test results within the same drill hole for the different depths is rather large. This may be the result of the fact that testing was carried out on single fracture or non-fractured parts of drill hole stretches of only 45 cm. These tests can be influenced by small scale effects especially due to heterogeneity of the rock mass and seepage flow around the stress measurements and not only the bare minimum in-situ stress values.

As can be seen from the Tables 1 to 3, it is established in general that considerable minimum in-situ stresses are present in the rock mass and there are no stress relieved or alarming zones.

## 8.2 Upstream Starting Point of Steel Liner

The magnitude of minimum stress was found to be more than 3.08 MPa at all the locations except one value equal to 2.76 MPa at RD 22457 m. However, minimum stress at this location varied from 2.76 MPa to 8.62 MPa with an average value of 4.65 MPa. It was, therefore, decided to start the steel liner at RD 22557m to keep it sufficiently away from the adit junction which is at RD 22537m (Fig. 7). This location was safe as the magnitudes of minimum in-situ stress varied from 4.48 MPa to 6.21 MPa with

an average value of 5.26 MPa. The excavation of another adit of 140.4m long for the installation of steel liner could also be avoided as the existing adit entrance was located at RD 22537 m.

Table 2 – Measured in-situ stresses at the end of steel liner

Test no.	Test depth (m)	Maximum stress (MPa)	Minimum stress (MPa)
RD: 23227 m, Right wall, Horizontally inclined drill hole (at 35° towards u/s side)			
21	13.00-13.45	9.31	5.17
22	16.00-16.45	8.28	5.17
23	22.70-23.15	3.03	1.72
24	26.00-26.45	17.94	10.00
RD: 23227 m, Left wall, Horizontal drill hole			
25	10.45-11.40	4.13	2.41
26	23.40-23.85	10.35	6.55
RD: 23267 m, Left wall, Horizontal drill hole			
27	12.37-12.82	12.42	7.24
28	10.15-10.60	9.66	5.86
29	5.00-5.45	8.97	5.15

Table 3 - Minimum stress and corresponding factors of safety (FOS) at different test locations for Manglad steel liner

Location (Vertical cover)	Drill hole No. (Test no.)	Minimum stress measured (MPa)	Minimum value of minimum stress (MPa)	Average value of minimum stress (MPa)	Static internal pressure (MPa)	Dynamic internal pressure (MPa)	F.O.S. for static pressure	F.O.S. for dynamic pressure
22457 (304 m) Start Area	RW/H (1-3)	2.76-3.79	2.76	3.39	2.01	2.74	1.37	1.01
	LW/1 (4-5)	4.48-8.62	4.48	6.55	2.01	2.74	2.23	1.64
22487 (275m) Start Area	LW/BH1 (6-7)	4.83-8.97	4.83	6.90	2.01	2.75	2.40	1.76
	RW/BH3 (8-9)	2.76-3.28	2.76	3.02	2.01	2.75	1.37	1.00
22517 (248m) Start Area	RW/H (10-12)	3.79-7.93	3.79	5.29	2.02	2.76	1.88	1.37
	RW1 (13-14)	5.52-5.86	5.52	6.55	2.02	2.76	2.73	2.00
22547 (236 m) Start Area	RW/H (15-18)	4.48-6.55	4.48	5.52	2.02	2.76	2.22	1.62
	LW/H (19-20)	4.48-5.17	4.48	4.83	2.02	2.76	2.22	1.62
23227 (196 m) End Area	RW/1 (21-24)	1.72-10.0	1.72	5.52	2.10	2.86	0.82	0.60
	LW/H2(25-26)	2.41-6.55	2.41	4.48	2.10	2.86	1.15	0.84
23267 (224 m) End Area	LW/DH3 (27-29)	5.15-7.24	5.17	6.90	2.12	2.88	2.44	1.80

Values for the minimum stress and corresponding factors of safety at different test locations are given in Table 3. Necessity and essential in-situ rock mass testing for steel liner in tunnel was discussed by Singh (2000) during design. As seen from Table 3, both the down-stream locations i.e. RD 22517 m and 22547 m satisfy the criteria for a factor of safety of 1.88 against static pressure and 1.37 against dynamic pressure. In spite of somewhat lower overburden at these locations (Fig. 4 and Table 3), factors of safety are considerably higher as compared to upstream locations at RD 22457 m and RD 22487 m with values of 1.37 against static pressure and 1.4 against dynamic pressure (Table 3). This local stress relief could be due to presence of a shear zone and badly fractured rock near RD 22487 m.

Accordingly, the starting point of the steel liner was fixed at RD 22557 m to keep it sufficiently away from the adit junction at RD 22537 m to facilitate installation.

### 8.3 Downstream End Point of Steel Liner

In all, nine hydraulic fracturing tests were conducted inside three drill holes at different depths for fixing the end of the steel liner. The in-situ stresses measured and interpreted at two locations for

fixing the start of steel liner have been given in Table 2. Values for the minimum stress and corresponding factors of safety at different test location are given in Table 3.

The in-situ stresses measured and interpreted at two locations (RD's 23227 m and 23267 m) for fixing the end of steel liner have been given in Table 2. The magnitude of minimum stress was found to be more than 3.08 MPa at all the locations except 1.72 MPa and 2.41 MPa at RD 23227 m. However, minimum stress at this location varied from 1.72 MPa to 10 MPa with an average value of 5.17 MPa. Due to these low stresses at RD 23227 m, it was decided to end the steel liner at RD 23267 m. This location was safe as the magnitudes of minimum in-situ stress varied from 5.15 MPa to 7.24 MPa (Table 2) with an average value of 6.08 MPa.

As can be seen from Table 3, factors of safety at RD 23267 m are 2.44 against static pressure and 1.80 against dynamic pressure which are well above the desirable values. But just 40 m upstream at RD 23227 m, these are only 0.82 against static pressure and 0.60 against dynamic pressure which are much lower and below acceptable value. Because both the drill holes at the left and right wall at RD 23227 m are indicating low minimum stress, local small-scale effects can be ruled out. Further, as the results of hydraulic jacking tests also indicate values for minimum stresses higher than the internal pressure under dynamic condition, end point of steel liner was fixed at RD 23267 m.

#### 8.4 Hydraulic Jacking Test

The results of hydraulic jacking tests for the Manglad reach of HRT are given in Table 4 along with coefficient of permeability.

For further confirmation of the length of steel liner, in-situ simple hydraulic jacking tests (water injection test with a double packer up to high pressure until fracture opens) were conducted. These tests were conducted at RDs 22497 m, 22522 m and 22547 m for the start of the steel liner, at RDs 22927 m and 22967 m for the midway section and at RD 23242 m for the end of steel liner.

The results of hydraulic jacking tests (Table 4) also indicate that the minimum stresses, in general, are in the range of 2.5 to 4.0 MPa. The coefficient of permeability varied from  $1.90 \times 10^{-7}$  to  $0.88 \times 10^{-5}$  cm/sec. accordingly, the starting point of steel liner was fixed at RD 22557 m (to keep it sufficiently away from the adit junction) and the end point of steel liner was fixed at RD 23267 m.

#### 8.5 Length of Steel Liner

On the basis of the in-situ stresses measured by conducting hydraulic fracturing tests, the start and end of steel liner were decided to be fixed at RD's of 22557 m and 23267 m, respectively. The junction of Manglad adit and HRT was at 22537 m and the start of steel liner was fixed at RD 22557 m. This has decreased the length of steel liner from RD 22487 m to RD 22557 m i.e. by 70 m. The original proposed length of steel liner was 780 m and it was finally reduced to 710 m (Fig. 7). This has also resulted to avoid the construction of 140.40 m long and 7.50 m D-shape bypass adit as shown in Fig. 7.

In this way, the hydraulic fracturing test results were used fruitfully to fix the start and the end of steel liner, which saved a lot of time and money at this project site.

### 9. CONCLUSIONS

The following conclusions are drawn based on extensive investigations for the fixation of length of steel liner inside the head race tunnel:

- The measurement of in-situ stresses by hydraulic fracturing tests has served as a major tool for undertaking the decision with assurance of safety regarding the length of steel liner inside head race tunnel of the hydroelectric project.

Table 4 - Results of hydraulic jacking test for Manglad reach

Station (m)	Hole no. and test location	Test depth (m)	Maximum pressure (MPa)	water attained	Permeability (K) (cm/sec)
22497	BH-1, Right side	3.0-6.0	1.5		$3.81 \times 10^{-6}$
		14.0-17.0	2.5		$0.88 \times 10^{-5}$
	BH-2, Left side	15.0-18.0	3.0		$1.90 \times 10^{-7}$
		12.5-15.5	3.5		$2.98 \times 10^{-6}$
22522	BH-3, Right side	9.0-12.0	3.0		$2.72 \times 10^{-6}$
		20.0-17.0	3.0		$7.43 \times 10^{-6}$
	BH-4, Left side	16.0-13.0	3.5		$1.51 \times 10^{-6}$
		16.0-13.0	3.0		$1.54 \times 10^{-5}$
22547	BH-5, Right side (across foliation)	13.0-10.0	3.0		$7.53 \times 10^{-6}$
		20.0-17.0	3.5		$2.08 \times 10^{-6}$
		16.0-13.0	3.5		$4.32 \times 10^{-6}$
	BH-6, Right side (along foliation)	12.0-9.0	3.0		$2.18 \times 10^{-6}$
20.25-17.25		3.5		$1.75 \times 10^{-6}$	
22927	BH-9, Left side (along foliation)	16.50-13.5	3.5		$2.08 \times 10^{-6}$
		12.5-9.5	3.0		$2.57 \times 10^{-6}$
		20.0-17.0	4.0		$3.12 \times 10^{-6}$
	BH-10, Left side (across foliation)	16.0-13.0	4.0		$1.29 \times 10^{-6}$
13.0-10.0		2.5		$2.19 \times 10^{-6}$	
10.5-13.5		4.0		$1.68 \times 10^{-6}$	
22967	BH-11, Left side (along foliation)	14.0-17.0	4.0		$2.45 \times 10^{-6}$
		17.0-20.0	2.5		$2.14 \times 10^{-6}$
		17.0-20.0	4.0		$1.47 \times 10^{-6}$
	BH-12, Left side (along foliation)	13.0-16.0	4.0		$1.26 \times 10^{-6}$
10.0-13.0		4.0		$1.65 \times 10^{-6}$	
17.0-20.0		3.5		$1.68 \times 10^{-6}$	
23242	BH-7, Left side (across foliation)	13.5-16.5	4.0		$1.26 \times 10^{-6}$
		10.0-13.0	3.5		$2.40 \times 10^{-6}$
		20.0-17.0	3.5		$2.08 \times 10^{-6}$
	BH-8, Left side (along foliation)	14.0-11.0	3.5		$1.51 \times 10^{-6}$
20.0-17.0		3.5		$1.63 \times 10^{-6}$	
		17.0-14.0	3.5		$2.20 \times 10^{-6}$
		13.0-10.0	3.5		$3.30 \times 10^{-6}$

- The construction of an additional bypass adit (7.5m D-shape and 140.40 m length) to head race tunnel for the installation of steel liner was avoided in addition to the existing adit after fixing the start of steel liner based on in-situ testing, instead of tentatively fixed location.
- In-situ testing has helped in fixing the precise location of the start and end points of the steel liner. The length of final steel liner based on stress measurement was much lesser than that fixed by the Norwegian criterion.
- The length of steel liner was reduced by 70 m which further resulted in saving of time and construction cost. Thus, a lot of time and money was saved due to the measurement of the in-situ stresses by hydraulic fracturing tests.
- Besides achieving economy through reduction in the length of steel liner, in-situ testing ensured protection against hydraulic jacking and uplift, which often result in excessive leakage and large-

scale collapse or instability around the tunnel. A careful and conservative design to prevent problems of this nature is warranted for all hydro-electric projects, in particular for a mega projects of the magnitude of Nathpa Jhakri project when compared with project delays and money lost in revenues, the cost of repairs and the mental anguish of having to redo an unsafe design. The project is under operation without any problem in the steel lined tunnel since 2003 inspite of lowest rock cover.

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