

Back calculation of in-situ rock stresses for a tunnel project in Himachal, India

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ABSTRACT

Determination of in-situ stresses in the rock mass is necessary for stability assessment and proper design of underground openings. It is important to know the state of stress surrounding the opening so that right and optimum rock support is assigned as preliminary and permanent rock support. However, the majority of long tunnels with high rock cover face severe tunnel instability problems related to rock stresses. The headrace tunnel of Parbati II hydroelectric project is one of such tunnels, especially the tunnel segment passing through Manikaran quartzite. It is known fact that the extent and type of stress induced instability vary greatly upon rock type, deformability properties, jointing and inter-bedding characteristics in the rock mass. This paper back calculates the state of stress using *Phase 2* finite element model in a TBM bored segment of the tunnel and also briefly reviews mechanical properties of the intact rock that may have direct link on the nature of stress induced instability. It is believed that back calculated stress magnitude may be useful for the stability assessment in other segment of headrace tunnel.

Keywords: Rock mass properties, rock stresses, tunnel squeezing, rock burst, back calculation

Received: 12 December 2010

revision accepted: 5 April 2011

INTRODUCTION

An underground opening is mainly subjected to three areas of engineering geological uncertainties, which directly influence in the state of stability. These are rock mechanical properties, in-situ stress conditions and groundwater inflow through open fractures and weakness zones. The priorities given in dealing with these uncertainties may vary from project to project. As discussed in many occasions, due to high tectonic activity the rock mass in the Himalaya are fractured, faulted, sheared and deeply weathered. Meeting numerous zones of weakness, fractures and faults is thus a matter of reality. Majority of long tunnels meet several instability and excavation challenges. Selection of right excavation approach based on sufficient engineering geological investigation at planning is crucial for successful completion of tunnel projects and to reduce uncertainty associated with delay in completion and cost overruns.

Due to steep topographic conditions prevailing in the Himalaya, majority of long tunnels have a chance to face instability problems associated to rock stresses. Determination of in-situ stress state is hence crucial, and this may be done either by in-situ stress measurements or by back calculation using numerical modeling tools. This paper aims to back calculate the state of in-situ stress in the headrace tunnel of Parbati II project. It also reviews mechanical properties of intact rock/rock masses.

BRIEF ON THE PROJECT

Parbati II hydroelectric project is a run-off-river scheme located in Himachal, India. The project is aiming to harness hydro potential of lower reach of the Parbati River by utilizing gross head of 862 m. The river is being diverted from near village Pulga by constructing a 85 m high and 113 m long concrete gravity dam. The project is designed to have picking reservoir with a storage potential to run powerplant at an installed capacity of 800 MW for four hours a day during lean flow season. A 31.37 km long headrace tunnel and twin inclined penstock pressure shafts with each length of 1.54 km will convey water from Parbati River to a semi-underground powerhouse located on the right bank of Sainj River near Suind village (NHPC 2000). The lay out plan of the project is shown in Fig. 1.

The headrace tunnel is being excavated from five different construction Adits (Fig. 1). Apart from tunnel stretch upstream Adit 2, the headrace tunnel is excavated using Drill and Blast Method (DBM) of excavation.

The project has different three Lots of Civil Work Contracts. The Construction Lot PB.2 includes civil works contract for 21.23 km middle segment of the headrace tunnel (HRT) from chainage 3.5 km to chainage 24.73 km. The headrace tunnel between chainage 6.1 km (Adit 1 junction) to 19.46 km (Adit 2 junction) is longest reach with 13.36 km tunnel length (Fig. 1). Due to environmental

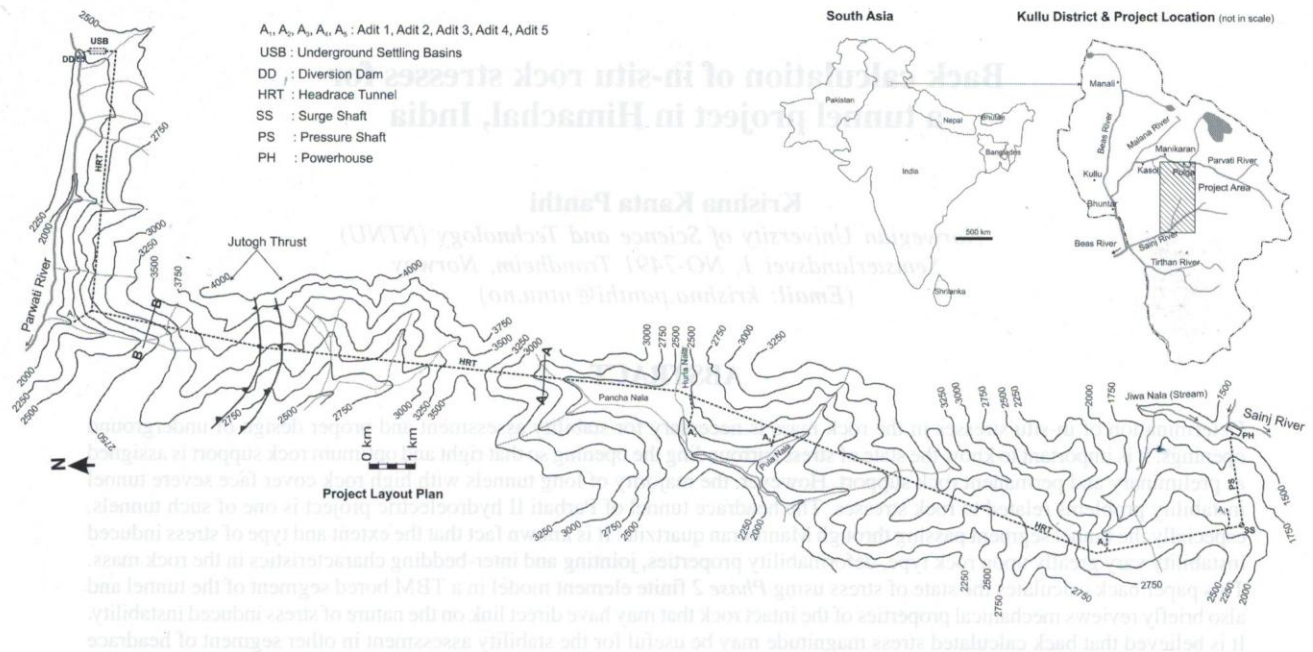


Fig. 1: Location and layout plan of the Parbati II Project giving two study sections (Panthi 2009)

restrictions related to flora and fauna, there was no possibility to allocate an intermediate Adit within this tunnel stretch. Hence, 9.05 km headrace tunnel stretch upstream from Adit 2 (between chainage 19.46 km to 10.3 km) was planned to be excavated using Tunnel Boring Machine (TBM). The TBM tunnel is designed as 6.8 m excavation diameter. Rock bolts, steel rings and wire mesh is being used as temporary tunnel support for TBM stretch and systematic rock bolting, steel fiber shotcrete, steel arch ribs and steel plates is being used as temporary tunnel support for headrace tunnel stretch excavated using DBM (NHPC 2000, Panthi 2009).

GEOLOGY OF THE PROJECT AREA

Geologically, the project area is located in so called ‘Kulu Window’ that represents Lesser Himalayan rock formation. The area is bounded by a major fault system in the Himalaya called as Main Central Thrust (MCT) (Fig. 2). The Kulu Window is mainly comprised by rocks from crystalline to meta-sedimentary sequence. Main rock types in the area are schistose granitic gneiss, biotite- quartzitic schist, quartzite, slate, phyllite and dolomite.

Being surrounded by the MCT, the rocks in the project area have undergone intense deformation and are faulted, folded and jointed. As shown in Fig. 1 the headrace tunnel passes through roughed topography with steep slopes, high mountains and deep valleys. Weathering effect in the rock mass near surface is considerable. Mountains and valleys below elevation 3,000 meters are mostly covered with thick forest and vegetation (Panthi 2003).

ROCK MASS CONDITION BETWEEN ADIT A1 AND ADIT A2

The headrace tunnel passes mainly through four categories of rock formations. These are; biotite schist, carbonaceous phyllite, Manikaran quartzite and schistose granitic gneiss (Fig. 3). The biotite schist is intercalated with small bands of quartzitic schist, whereas carbonaceous phyllite is deformed and folded. Granitic gneiss is occasionally intercalated with small bands (1-10 meters in thickness) of chlorite/talcosic mica schist, and hence is highly schistose. However, Manikaran quartzite is fresh and massive (Panthi 2003). Headrace tunnel also crosses several major and minor weakness zones. Mostly, major weakness zones represent contact zones between different rock formations. In addition, some prominent shear and fracture zones are also present.

The TBM reach of headrace tunnel begins at approximate chainage 19.46 km at Adit 2 junction located at the left bank slope of Hurla Nala (Fig. 1) and ends at approximate chainage 10 km. This reach of headrace tunnel passes mainly through two rock formations i.e. schistose granitic gneiss and Manikaran quartzite. Similarly, the DBM tunnel downstream from Adit 1 (downstream from chainage 6.1 km) passes through three rock formations i.e. biotite schist, Manikaran quartzite and carbonaceous phyllite (Fig. 3).

Rock mass between Adit 1 and Adit 2 has three prominent joint sets with occasional occurrence of random joints. The foliation joints (Jf), cross joint (J1) and cross joints (J2) have mean orientation N150E/65NE, N060E/

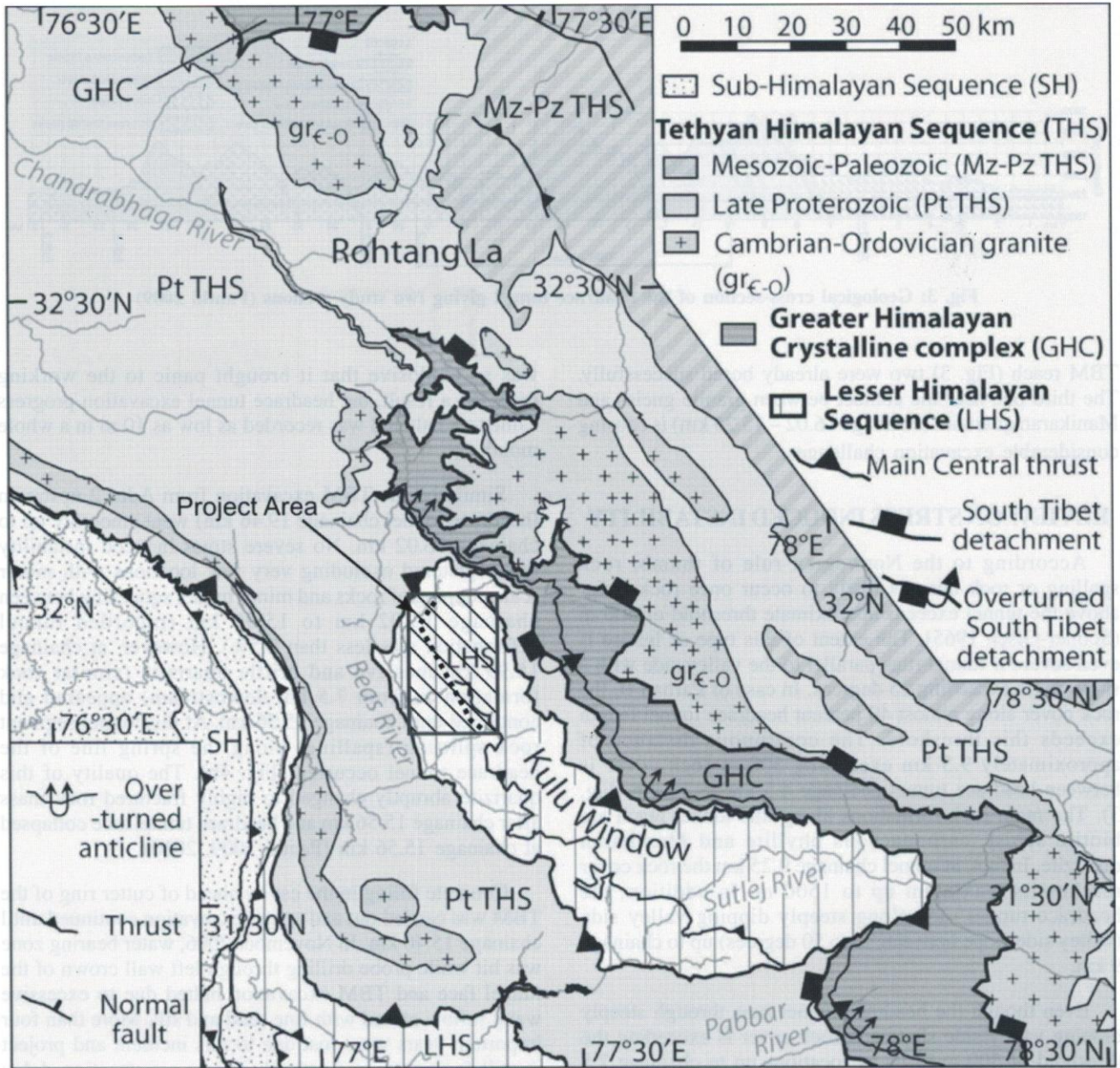


Fig. 2: Geology in the project area (after Alexander et al. 2007)

70NW and N105E/20SW, respectively. Joints in the Manikaran quartzite are either open or filled with silt material. On the other hand, joints in granite gneiss, biotite schist and carbonaceous phyllite are sheared, schistose and filled with mica clay (Panthi 2006b).

Seven major and minor weakness zones were predicted along the headrace tunnel between Adit 1 and Adit 2 (Panthi 2003). These weakness zones should be treated specially during tunneling. In addition, varied degree of stress induced instabilities as well as groundwater inflow into the tunnels is likely to occur. The granite gneiss in contact with Manikaran quartzite is highly schistose and impermeable,

whereas, the Manikaran quartzite is fractured and open jointed. Therefore, substantial groundwater inflow is likely to occur at this contact zone.

Most of the predicted weakness zones (excluding weakness zones at approximate chainage 11.25 km and 12.5 km, where tunnel excavation is still remaining) were met during tunnel excavation with certain degree of deviation in location. The tunnel excavation from Adit 1 downstream was completed up to 10 km using drill and blast method. The TBM excavation of headrace tunnel from Adit 2 upstream is going on and has faced serious excavation challenges. Out of five weakness zones predicted along

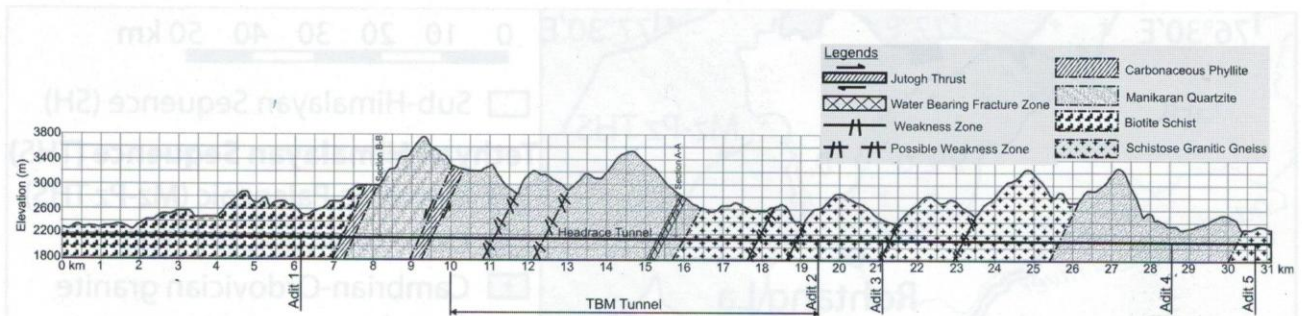


Fig. 3: Geological cross-section of the headrace tunnel giving two study sections (Panthi 2009)

TBM reach (Fig. 3) two were already bored successfully. The third one near the contact between granite gneiss and Manikaran quartzite (chainage 16.02 – 15.25 km) is causing considerable excavation challenges.

REVIEW ON STRESS INDUCED INSTABILITY

According to the Norwegian rule of thumb, rock spalling or rock burst is likely to occur once rock cover above the tunnel exceeds approximate threshold of 500 m (Selmer-Olsen 1965). The extent of this type of failure is even severe if tunnel runs parallel to the valley side with a slope angle exceeding 25 degrees. In case of Parbati II, the rock cover along almost 40 percent headrace tunnel length exceeds this threshold. The continuous distance of approximately 9.5 km exceeding 500 m rock cover is between headrace tunnel chainage 4.5 km to 16 km (Fig. 3). The main rock formations along this tunnel reach are biotite schist, carbonaceous phyllite and Manikaran quartzite. In fact, at tunnel chainage 9.25 km the rock cover reaches its maximum up to 1500 m. In addition, the headrace tunnel runs along steeply dipping valley side (valley side slope between 30 to 50 degrees) up to chainage 9 km.

Even though the headrace tunnel runs through steeply dipping valley side slope and rock cover is exceeding the threshold of 500 m in many locations up to chainage 7.5 km, no noticeable rock spalling or rock burst occurred. Excavation work went smoothly with no major stress related instability observed excluding minor tunnel squeezing. As shown in Fig. 3, the headrace tunnel up to this chainage mainly passes through biotite schist and a small band of carbonaceous phyllite exists between chainage 7.4 km to 7.5 km. At carbonaceous phyllite minor tunnel squeezing (plastic deformation) with a magnitude maximum up to 2.5 percent was recorded, which was not that significant in regards with the rock cover prevailing (Panthi 2010).

However, after chainage 7.5 km massive and brittle Manikaran quartzite appeared. In this quartzite severe rock burst occurred continuously in the valley side roof of the headrace tunnel (Fig. 4a) after almost every alternate blasting round. The extent of rock burst

was so explosive that it brought panic to the working crew. As a result, the headrace tunnel excavation progress came to a halt and was recorded as low as 10 m in a whole month.

Similarly, the TBM excavation from Adut 2 upstream (headrace tunnel chainage 19.46 km) went smoothly up to chainage 16.02 km. No severe stress induced instability was registered excluding very few locations with minor scale popping of rocks and minor tunnel squeezing between chainage 16.02 km to 15.99 km (recorded tunnel deformation was less than 2.5%). However, at chainage 15.99 km massive and brittle quartzite (similar rock formation as from 7.5 km downstream) appeared and continued until chainage 15.56 km. At this tunnel segment rock splitting (spalling) along the spring line of the headrace tunnel occurred (Fig. 4b). The quality of this quartzite abruptly changed to highly fractured rock mass after chainage 15.56 km and headrace tunnel face collapsed at chainage 15.56 km (Panthi 2009, 2010).

Concrete filling in the cavity ahead of cutter ring of the TBM was carried out and tunnel excavation continued until chainage 15.40 km. In November 2006, water bearing zone was hit while probe drilling through left wall crown of the tunnel face and TBM excavation halted due to excessive water inflow mixed with fine sand and silt. More than four important years were lost due to this incident and project is still facing serious setback due to construction delay and financial loss. Fortunately, it is reported that the groundwater inflow is now controlled and it is hoped that TBM excavation will be a success.

BACK CALCULATION OF IN-SITU ROCK STRESS

Magnitude of principle stresses in the rock mass may be established by two means; (1) in-situ stress measurement, and (2) back calculation using numerical modeling. The later one is somewhat delicate. An ideal condition is required in achieving representative back calculation results. This idealism may generally be referenced to following three points;

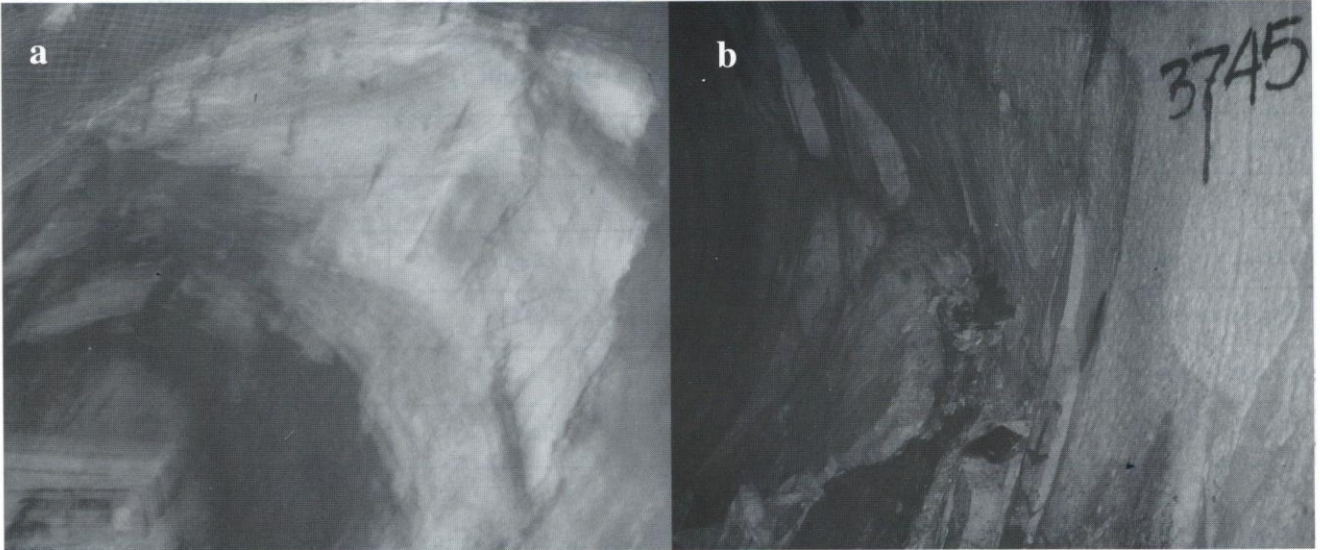


Fig. 4: Rock burst in Manikaran quartzite. Damage in drill and blast tunnel in the valley side roof around chainage 8.6 km (a) and damage in TBM tunnel in the spring line around chainage 15.7 km (b)

- the tunnel has a circular shape (preferably TBM excavation),
- reliable lab tested rock mechanical properties are available and,
- the tunnel is in the border of elastic-brittle failure

In this case the headrace tunnel upstream Adit 2 is being excavated using TBM and tunnel is perfectly circular in shape. The lab tested rock mechanical properties for both Manikaran quartzite and biotite schist are available (Table 1) and minor rock splitting occurred at around chainage 17.5 km. That too along the spring line of headrace tunnel (Fig. 4b), which is an ideal condition for back calculation. Failure along spring line of the tunnel indicates that major (σ_1) and minor (σ_3) principle stresses are vertical and horizontal, respectively.

ROCK MECHANICAL PROPERTIES

Rock mass mechanical properties, in particular rock mass strength and deformability properties are the parameters that govern on what type of failure (instability) may occur due to stress an-isotropy, i.e. rock burst or tunnel squeezing. Table 1 gives laboratory tested mechanical properties for biotite schist and Manikaran quartzite.

Intact rock strength (σ_c) and elasticity modulus (E_c) (Table 1) are for 50 mm equivalent core diameter. A brittle failure occurred in Manikaran quartzite indicated that this rock is brittle and fails abruptly once it exceeds uniaxial compressive strength. Regarding mineralogical composition, Manikaran quartzite consists of 99 % quartz. Biotite schist, however, is made of 48 % mica, 8 % chlorite and 42 % quartz.

As seen in Table 1, biotite schist has considerably reduced strength and deformability properties than quartzite due to high mica content, and is weak and deformable, whereas quartzite is very strong and brittle. The mechanical properties of two rocks had direct influence on the type of instability, one can expect in the high rock cover area.

INPUT PARAMETERS FOR NUMERICAL ASSESSMENT

Since Manikaran quartzite is massive and is an elastic-brittle material, Mohr-Coulomb failure envelope is considered relevant for numerical analysis using Phase 2. For carrying out numerical assessment, required input variables are estimated and presented in Table 2. Rock mechanical properties given in Table 1, relationships proposed by Hoek et al. (2002) and Panthi (2006a) and actual rock mass quality conditions observed inside the headrace tunnel are used while estimating input variables. The RockLab is used for the computation.

NUMERICAL ANALYSIS

As discussed, the failure in Manikaran quartzite at chainage 15.7 is elastic-brittle and major principle stress (σ_1) is oriented vertically. Meaning, the major principle stress (σ_1) is induced by gravity alone and is a product of specific weight (γ) and rock cover above headrace tunnel (h). At chainage 15.7 km the rock cover is 775 m and Manikaran quartzite has specific weight (γ) of 0.026 MN/m³ (Table 1). This gives vertical stress, in this case major principle stress (σ_1), equal to 20.15 MPa. So, one set of principle stress is known. Concerning horizontal stress, in this case minor principle stress (σ_3), the only known fact

Table 1: Mean values of lab tested properties for biotite schist and Manikaran quartzite

| Mechanical properties | Unit | Biotite schist (Adit 1) | Manikaran quartzite (Adit 2) |
|--|-------------------|-------------------------|------------------------------|
| Specific weight | MN/m ³ | 0.028 | 0.026 |
| Intact rock strength, UCS, ($\bar{A}ci$) | MPa | 75 ± 15 | 175 ± 25 |
| Elasticity modulus (Eci) | GPa | 12 ± 5 | 65 ± 10 |
| Poisson's ratio (ν) | - | 0.13 | 0.20 |
| Strength anisotropy coefficient | - | 2 | 1.1 |
| Mode of failure | - | Plastic | Brittle |

is that it is oriented perpendicular to the major principle stress (σ_1). The magnitude of this horizontal stress is not fully known, but according to Panthi (2008) it may be expressed by following relationship;

$$\sigma_3 = \frac{\nu}{1-\nu} \times \sigma_1 + \sigma_{tec} \dots \dots \dots \text{Eq. 1}$$

In Equation 1, σ_{tec} may be expressed as horizontal component of stress contributed by plate tectonics.

The Himalayan region is tectonically active and magnitude of horizontal stress may vary considerably depending upon geographical location, geological environment and distance from main tectonic faults, i.e. distance from the Main Boundary Thrust (MBT) to the Main Central Thrust (MCT). According to World Stress Map 2008 the orientation of horizontal tectonic stress at far western part of the Himalaya is close to east-west. Meaning horizontal tectonic stress makes an orientation of approximately 60 to 65 degrees with headrace tunnel alignment between Adit 1 and Adit 2 (Fig. 1).

With this understanding and with the use of data sets presented in Tables 1 and 2, back calculation of minor principle stress (σ_3) perpendicular to the headrace tunnel alignment is carried out using *Phase2* finite element code. In addition to data sets given in Tables 1 and 2, three additional parameters; i.e. dilation, residual frictional angle, and residual cohesion needed to be assigned for Mohr-Coulomb failure envelope.

Therefore, dilation of 1.1, residual frictional angle of 27 degrees and residual cohesion of 1 MPa is assumed to be representative for Manikaran quartzite.

The splitting of rock mass that occurred along spring line at chainage 15.7 km indicates strength factor close to or just below one. This particular condition is achieved in *Phase2* by realizing yielding after simulation. This yielding assessment is carried out for three different horizontal to

vertical stress ratios with constant vertical major principle stress (σ_1) and two different input variables of the rock mass representing maximum and mean values given in Table 2. The results achieved by simulation are presented in Fig. 5.

Fig. 5 shows that no yielding occurred for all three horizontal to vertical stress ratios (0.6, 0.55 and 0.5) with rock mass property variables representing upper limits (mean plus standard deviation) given in Table 2. Similarly, simulation result also indicates that the strength factor is more than one (exceeding 1.38). On the other hand, clear yielding occurred for all three horizontal to vertical stress ratios (0.6, 0.55 and 0.5) with rock mass properties representing mean values. Simulation result also indicated considerable reduction in the strength factor, which is close to or below one in all three stress ratios.

The yielding occurred for horizontal to vertical stress ratio representing 0.5 looks symmetric (Fig. 5). The yielding is also concentrated along spring line of the headrace tunnel. The strength factor in this case is 0.96, which is just below one and is considered representative for the extent of splitting occurred at this headrace tunnel chainage.

CONCLUSIONS

This analysis helped to conclude that the horizontal to vertical stress ratio at chainage 15.7 km (rock cover 775 m) is very close to 0.5. This gives in-situ minor principle stress (σ_3) equivalent to 10.1 MPa. The rock splitting occurred along spring line of the headrace tunnel (Fig. 4b) indicates that there is no valley side effect, which is fairly logical because the headrace tunnel at this particular point is located 700 m below from the bottom of the valley (Figs. 1 and 3). With this conclusion it is possible now to back calculate horizontal tectonic contribution of in-situ stress using Equation 1. With mean value of Poisson's ratio 0.2 for Manikaran quartzite (Table 1) and vertical stress (σ_1)

Table 2: Estimated input variables for Manikaran quartzite at chainage 15.7 km

| Descriptions | Unit | Estimated Values | Remarks / Relationships |
|--|--------|------------------|--|
| Geological strength index (<i>GSI</i>) | - | 75 ± 5 | Blocky rock mass consisting of widely spaced and well interlocked three joint sets |
| Material constant (<i>m_i</i>) | - | 20 ± 3 | For quartzite (Hoek 2007) |
| Disturbance factor (<i>D</i>) | - | 0 | TBM tunnel / controlled blasting (Hoek et al. 2002) |
| Reduced material constant (<i>m_b</i>) | - | 7.6 ± 0.75 | $m_b = m_i \times \exp\left(\frac{GSI - 100}{28 - 14D}\right)$ (Hoek et al. 2002) |
| Constant (<i>s</i>) | - | 0.035 | $s = \exp\left(\frac{GSI - 100}{9 - 3D}\right)$ (Hoek et al. 2002) |
| Cohesion (<i>c</i>) | MPa | 6.2 ± 0.9 | $c = \frac{\sigma_{ci} [(1 + 2a)s + (1 - a)m_b \sigma_{3n}] \times (s + m_b \sigma_{3n})^{a-1}}{(1 + a)(2 + a)\sqrt{1 + (6am_b(s + m_b \sigma_{3n})^{a-1})/((1 + a)(2 + a))}}$ |
| Peak frictional angle (<i>α</i>) | degree | 53.6 ± 1 | $\phi = \sin^{-1}\left[\frac{6am_b(s + m_b \sigma_{3n})^{a-1}}{2(1 + a)(2 + a) + 6am_b(s + m_b \sigma_{3n})^{a-1}}\right]$ (Hoek et al. 2002) |
| Tensile strength (<i>σ_t</i>) | MPa | 1.1 ± 0.4 | $\sigma_t = \frac{s \times \sigma_{ci}}{m_b}$ (Hoek et al 2002) |
| Deformation modulus (<i>E_{cm}</i>) | GPa | 14.3 ± 1.8 | $E_{cm} = \frac{1}{60} \times E_{ci} \times \sigma_{ci}^{0.5}$ (Panthi 2006a) |

magnitude of 20.15 MPa, the tectonic stress component (σ_{tec}) perpendicular to the length axis of the headrace tunnel between Adit 1 and Adit 2 is approximately 4.4 MPa (10.07 - 5.68). With this finding it is now possible to make stability assessment on the remaining part of the TBM tunnel, which will be bored in the days to come.

ACKNOWLEDGEMENTS

The author is thankful to civil engineer Mr. Aditya Kapoor for latest updates in the project events. The author notes that the views and opinions expressed in this paper are of author's own and not of the parties involved in the project. The author expresses his sincere thanks to Mr.

Subash C. Sunuwar and Mr. Ram Hari Sharma for their helpful comments in the first draft manuscript.

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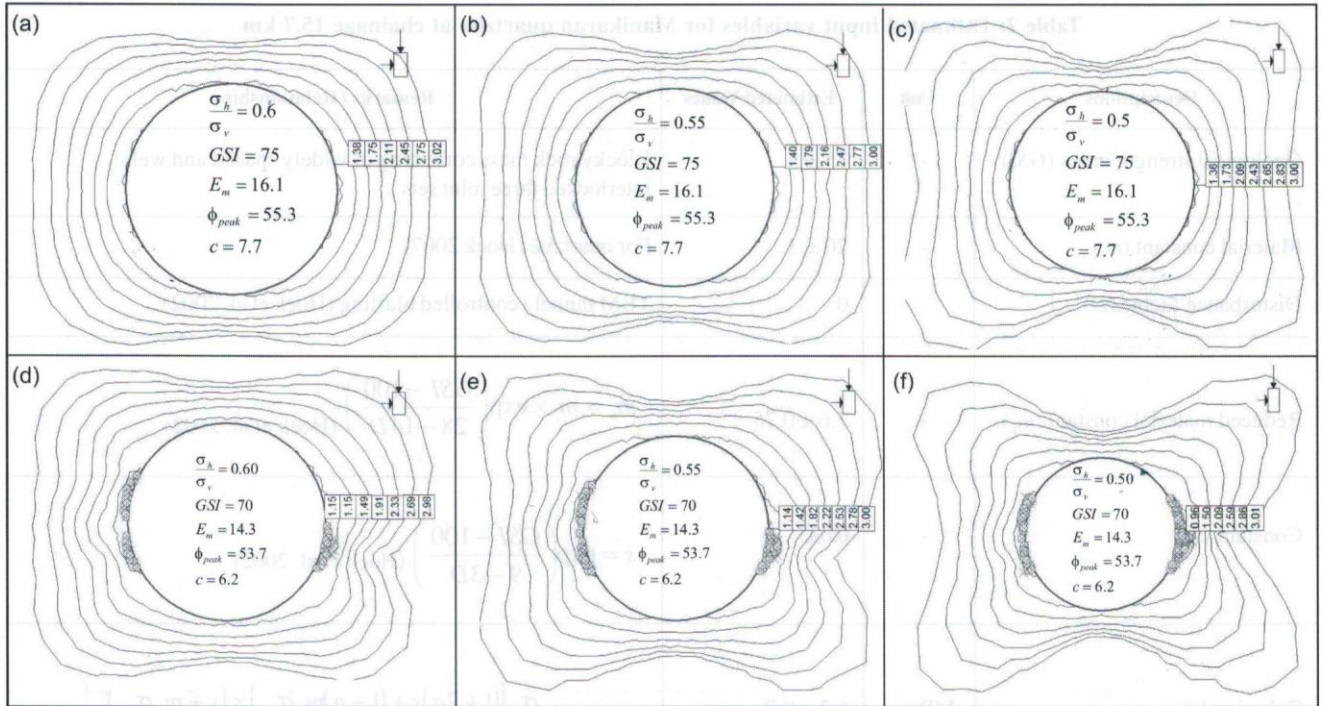


Fig. 5: Strength factor counters and yielding conditions achieved by Phase2 numerical modeling using three different horizontal to vertical stress ratios and two different rock mechanical property variables for the headrace

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ACKNOWLEDGEMENTS

The author is thankful to civil engineer Mr. Aditya Kapoor for latest updates in the project. The author notes that the views and opinions expressed in this paper are of author's own and not of the parties involved in the project. The author expresses his sincere thanks to Mr.