

## Retrospective Analysis of Tunnel Squeezing – A Case Study of Tunnel No. 10 along Sivok – Rangpo Railway Project (SRRP), Darjeeling, West Bengal, India.

Saradindu Mukherjee<sup>1</sup>, Sanket Tirpude<sup>2</sup>, Somnath Mishra<sup>3\*</sup>

<sup>1</sup> M.Tech. (Tunnel Engineering) Student, Dr. Vishwanath Karad MIT World Peace University, Pune, India.

Email: [saradindu.mukherjee@mitwpu.edu.in](mailto:saradindu.mukherjee@mitwpu.edu.in)

<sup>2</sup> M.Tech. (Tunnel Engineering) Professor, Dr. Vishwanath Karad MIT World Peace University, Pune, India.

Email: [sanket.tirpude@mitwpu.edu.in](mailto:sanket.tirpude@mitwpu.edu.in)

<sup>3</sup> Geologist, Cemindia Projects Limited (formerly ITD Cementation India Ltd), India.

Email: [somnath.mishra@cemindia.co.in](mailto:somnath.mishra@cemindia.co.in)

Corresponding Author

Saradindu Mukherjee

Email: [saradindu.mukherjee@mitwpu.edu.in](mailto:saradindu.mukherjee@mitwpu.edu.in)

### Abstract

The Government of India has undertaken the construction of “Sivok – Rangpo Railway Project (SRRP)” to establish rail route connectivity between Sivok (in West Bengal) and Rangpo (in Sikkim). The railway line consists of total 14 tunnels of variable lengths with connecting bridges. The Tunnel No. 10, being the longest tunnel with 5300 m length, underpasses maximum height of overburden i.e. approximately 800m. As an integral part of the geologically complex Himalayan Orogenic Belt, the tunnelling ground poses various threats towards safe and economic construction of the Tunnel. During construction phase, the tunnel has experienced squeezing and rock burst that basically represent high in-situ stress induced tunnel instability. Below about 725m overburden, the tunnel suffered from very severe to extreme tunnel squeezing and had to undergo re-excavation. To figure out the root cause of tunnel squeezing, an attempt to estimate the ratio of average horizontal stress to vertical stress (Earth Pressure Co-efficient or ‘k’) has been made from measured tunnel strain data. Considering the vertical rock pressure as minimum principal stress direction, complete in-situ stress tensor has been estimated. Retrospective analysis for tunnelling ground behavior has been exercised based on laboratory tested rock strength parameters and 50% reduced rock strength parameters (due to time dependent deformation or creep deformation) under the influence of ‘Back-analyzed’ in-situ rock stress magnitudes. Empirical, Semi-Empirical, Analytical (Convergence Confinement Method) as well as Numerical (Finite Element Method) analyses confirm the apprehension of very severe to extreme tunnel deformation that appears with concordance to the reality..

**Keywords:** convergence confinement method, finite element method, ground – support equilibrium, long term ground response curve, yielding tunnel support system..

### INTRODUCTION

Tunnel No. 10, along Sivok – Rangpo Railway Project (SRRP), experienced tunnel squeezing for a length of 124m between chainages 30+091m (Tunnel Meter - 1940m) and 30+215m (Tunnel Meter – 2064m). During its excavation through Portal No – 1, the tunnel suffered from very severe to extreme category of tunnel deformation along a length of 54m from chainage 30+161m (Tunnel Meter – 2010m) to 30+215m (Tunnel Meter – 2064m). The tunnel closure, in conjunction with complete dismantlement of primary tunnel support system was so intense that re-excavation of the tunnel had become inevitable. As a result, the mining work of the tunnel had to be stopped on and from 21.11.2023. For about next 8 (eight) months, no construction activity was feasible until re-excavation of the deformed tunnel portion between chainages 30+161m and 30+215m was commenced on 24.07.2024, along with installation of stronger primary tunnel support system.

### LITERATURE REVIEW

In his Ph.D. thesis, G.L.Shrestha (2006) compiled the influencing factors that contribute to tunnel squeezing in different degrees. These factors have been identified by previous researchers from time to time on the basis of analysis and case studies. The factors are (i) stress condition, (ii) strength and deformability of rock mass, (iii) rock type, (iv) water pressure and porosity of rock mass, (v) orientation of geological structures and (vi) construction procedure and support system. The conditions of these influencing factors that facilitate tunnel squeezing are (i) ratio of rock strength to rock stress less than 2, (ii) rock mass with low strength and high

deformability, (iii) Phyllosilicate mineral rich rock type like Phyllite, Schist, with Claystone, Tuff, and fault zone crushed rock, (iv) high water pressure and high porosity, (v) orientation of geological structures (fault or discontinuity) parallel to tunnel axis, and (vi) to resist tunnel squeezing, construction methodology that permits very short / minimum distance between tunnel face and tunnel support ring closure is favorable. Steel sets in combination with shotcrete or concrete and a circular cross-section may provide much higher support pressures than a dense pattern of rock bolts. Hoek and Brown (1978) proposed a universal principle to estimate vertical stress by multiplying the overburden depth with average density of earth's crust. Anderson (1951) identified three basic categories of 'stress regime' depending on which principal stress direction coincides with vertical stress. Hudson et al (2003) concluded that the vertical stress component is generally the lowest principal stress, independently of the local tectonics, for the first few hundred meters depth in hard rock. Hoek and Marinos (2000) established a relation between tunnel strain percentage with ratio of rock mass UCS and in situ stress in form of  $\epsilon (\%) = 0.2 (\sigma_{cm}/P_0)^{-2}$ . Alam and Sharma (2014) estimated maximum value of in situ rock stress subjected to the Head Race Tunnel by way of back analysis through the principle suggested by Hoek and Marinos (2000) and modified by Hoek (2001). Goel et al (1995) identified the depth of overburden required for onset of tunnel squeezing by incorporating overburden depth, tunnel diameter and Rock Mass Number (N value). This method is applicable for unsupported tunnel. Vlachopoulos and Diederichs (2009) in their research paper presented method to construct Longitudinal Displacement Profile (LDP) by which the quantity of tunnel strain can be ascertained along the tunnel length behind the tunnel face. It is used to know the amount of tunnel deformation at a specific distance behind the tunnel face. Carranza-Torres and Fairhurst (2000) in their research presented detail procedure to construct Ground Reaction Curve (GRC), Longitudinal Deformation Profile (LDP) and Support Reaction Curve (SRC) to conduct Convergence Confinement Analysis to estimate requirement of support to stabilize a tunnel. This method is applicable for circular tunnels, exposed to hydrostatic stresses. But in their research paper, a method has been proposed by which the asymmetrical ground stresses can be converted to hydrostatic stress value and can be applied in case of non-circular tunnels too. It was concluded that tunneling ground deformation ( $\pi$ ) and support element deformation must meet at an equilibrium point corresponding to any allowed tunnel displacement ( $\pi$ ) at a certain distance ( $x$ ) behind the tunnel face.

## HYPOTHESES

The tunnel squeezing under consideration is presumed to be an outcome of one or more than one causes, mentioned below.

*The high magnitude of vertical stress generated due to huge overburden (~725m) pressure has caused 'Creep Movement' within the rock mass surrounding the tunnel.*

*Higher horizontal stress magnitude relative to vertical stress is responsible for such squeezing behaviour. Such horizontal stress may be a product of tectonic plate movement, operative along the Himalayan Orogenic Belt.*

*During long-term constant loading condition, the rock mass lost significant portion of its original rock mass strength.*

*The equilibrium state for 'Ground – Support interaction mechanism' was not attained while implementing NATM method.*

| Table. I: Geotechnical character of each General Ground Type (GGT) for Tunnel No. 10 (SRRP) |                         |                         |                           |                           |                            |                            |            |
|---------------------------------------------------------------------------------------------|-------------------------|-------------------------|---------------------------|---------------------------|----------------------------|----------------------------|------------|
| Rock strength parameters                                                                    | Schist (more disturbed) | Schist (less disturbed) | Phyllite (more disturbed) | Phyllite (less disturbed) | Quartzite (more disturbed) | Quartzite (less disturbed) | Average    |
| GSI                                                                                         | 21                      | 36                      | 21                        | 36                        | 21                         | 36                         | 28.5       |
| $\sigma_{ci}$ (MPa)                                                                         | 20                      | 20                      | 15                        | 15                        | 20                         | 20                         | 18.33      |
| Mi                                                                                          | 6                       | 6                       | 10                        | 10                        | 24                         | 24                         | 13.33      |
| D                                                                                           | 0                       | 0                       | 0                         | 0                         | 0                          | 0                          | 0.00       |
| Ei (MPa)                                                                                    | 5000                    | 5000                    | 5250                      | 5250                      | 6500                       | 6500                       | 5583.33    |
| mb                                                                                          | 0.357125                | 0.610208                | 0.595209                  | 1.01701                   | 1.4285                     | 2.44083                    | 1.07       |
| s                                                                                           | 0.00015412              | 0.00081599              | 0.00015412                | 0.00081599                | 0.00015412                 | 0.00081598                 | 0.00048505 |
| a                                                                                           | 0.540887                | 0.514908                | 0.540887                  | 0.514908                  | 0.540887                   | 0.514908                   | 0.53       |
| $\sigma_3$ (MPa)                                                                            | 0.610393                | 0.626476                | 0.609874                  | 0.625259                  | 0.63828                    | 0.653317                   | 0.63       |
| Unit weight (MN/m <sup>3</sup> )                                                            | 0.026                   | 0.026                   | 0.026                     | 0.026                     | 0.026                      | 0.026                      | 0.026      |

|                        |                |                |                     |                |                     |                 |                     |
|------------------------|----------------|----------------|---------------------|----------------|---------------------|-----------------|---------------------|
| c (MPa)                | 0.117716       | 0.184723       | 0.125068            | 0.183584       | 0.184512            | 0.25866         | 0.175711            |
| $\phi$ (°)             | 32.6412        | 38.5423        | 35.1629             | 40.8691        | 45.1791             | 50.4045         | 40.4665             |
| $\sigma_t$ (MPa)       | -<br>0.0086312 | -<br>0.0267446 | -<br>0.0038840<br>2 | -<br>0.0120351 | -<br>0.0021577<br>9 | -<br>0.00668614 | -<br>0.0100231<br>4 |
| $\sigma_{cm}$<br>(MPa) | 1.27863        | 1.97243        | 1.26063             | 1.90952        | 2.69223             | 3.9688          | 2.18037             |
| Erm<br>(MPa)           | 240.231        | 606.976        | 252.243             | 637.325        | 312.3               | 789.068         | 473.024             |

## DATA COLLECTION

For the current research work, following data are collected.

### Regional Geology

The region under consideration belongs to the Himalayan Geological terrain and no way less complex than it. The intense tectonic activities consequent to the Himalayan Orogeny have left massive footprint to the rocks of Darjeeling and Sikkim in form of intricate folding, shearing, thrusting etc. The rocks found in this region range in age from Pre-cambrian to Recent and represent all three genetic groups viz. igneous, sedimentary and metamorphic. The tectonic units of Darjeeling and Sikkim Himalaya occur in a reverse order of stratigraphic superposition. The youngest Siwaliks in the south, followed by thrust sheets of older Gondwana, Buxa, Dalings and Darjeeling groups. towards north. The contact between different groups are represented by thrusts dipping at high angle towards north. The Daling group occupies the largest area of Teesta valley and form a dome below the Darjeeling gneiss. The most important main tectonic unit is the E-W trending north dipping thrust called Main Boundary Thrust (MBT) which crosses the main river Teesta near Kalijhora township

### Geology of the Project Area

The rock found in the project area mainly consist of “Daling Group” rocks. The Daling group of rocks are represented by Chlorite-Sericite Schists, Quartz-Schist, Quartzitic-Phyllite, Slaty Quartzite, etc. Two important formations of Daling group of rocks can be observed in this area. The older (1) Daling Group represented by Garubathan Formation comprising Chlorite-Sericites Schist / Phyllite with occasional Quartz veins and interbedded with Quartzite. The younger (2) Reyang Formation comprising Greywacke type of Slaty Quartzite that is hard and massive. Three generations of folding are discernible in this rock group. The older planar structure is bedding schistosity (S1) co-folded with quartzite interbeds into tight isoclinal fold (F1), Second generation (F2) folds are tight over-turned structures of recline type. The last generation of fold is open, broad and controlling the topography.

### Geotechnical Character of the Tunneling Ground

During geotechnical investigation, six types of ‘General Ground Types’ (GGT) have been identified. Rock strength parameters of all GGTs are listed below in Table. I.

| Table. II: Engineering Geological data collected during excavation of Tunnel No. 10 (SRRP) |                  |                |                     |                   |            |                       |                           |       |      |           |            |
|--------------------------------------------------------------------------------------------|------------------|----------------|---------------------|-------------------|------------|-----------------------|---------------------------|-------|------|-----------|------------|
| Sl No                                                                                      | Chain age (from) | Chain age (to) | Tunnel Meter (from) | Tunnel Meter (to) | Length (m) | Lithological Property | Discontinuity orientation | R M R | G SI | Derived Q | Rock Class |
| 1                                                                                          | 30+161           | 30+165         | 2010                | 2014              | 4          | Quartzose Phyllite    | Very Unfav.               | 28    | 41   | 0.034     | V          |
| 2                                                                                          | 30+165           | 30+167         | 2014                | 2016              | 2          | Phyllite              | Favourable                | 27    | 24   | 0.029     | V          |
| 3                                                                                          | 30+167           | 30+169         | 2016                | 2018              | 2          | Phyllite              | Very Unfav.               | 33    | 40   | 0.074     | V          |
| 4                                                                                          | 30+169           | 30+171         | 2018                | 2020              | 2          | Phyllite              | Fair                      | 24    | 24   | 0.018     | V          |
| 5                                                                                          | 30+171           | 30+172         | 2020                | 2021              | 1          | Phyllite              | Unfav.                    | 20    | 25   | 0.010     | VI         |
| 6                                                                                          | 30+172           | 30+173         | 2021                | 2022              | 1          | Phyllite              | Very Unfav.               | 17    | 24   | 0.006     | VI         |

|    |        |        |      |      |    |          |             |    |    |       |    |
|----|--------|--------|------|------|----|----------|-------------|----|----|-------|----|
| 7  | 30+173 | 30+174 | 2022 | 2023 | 1  | Phyllite | Very Unfav. | 19 | 26 | 0.009 | VI |
| 8  | 30+174 | 30+175 | 2023 | 2024 | 1  | Phyllite | Very Unfav. | 17 | 24 | 0.006 | VI |
| 9  | 30+175 | 30+179 | 2024 | 2028 | 4  | Phyllite | Very Unfav. | 15 | 22 | 0.005 | VI |
| 10 | 30+179 | 30+195 | 2028 | 2044 | 16 | Phyllite | Very Unfav. | 19 | 26 | 0.009 | VI |
| 11 | 30+195 | 30+202 | 2044 | 2051 | 7  | Phyllite | Very Unfav. | 20 | 27 | 0.009 | VI |
| 12 | 30+202 | 30+215 | 2051 | 2064 | 13 | Phyllite | Very Unfav. | 20 | 27 | 0.009 | VI |
|    |        |        |      |      |    |          |             |    |    |       |    |
|    |        |        |      |      |    |          |             |    |    |       |    |

#### Engineering Geological Data Collected During Tunnel Excavation

A rock mass classification system was exercised based on 'Rock Mass Rating' and 'Geological Strength Index' systems where RMR and GSI values corresponding to rock mass exposed at each tunnel face was evaluated. The Geological Tunnel Face Mapping was done to collect the Geological and Engineering Geological data from each tunnel face concurrent to tunnel advancement. Rock Tunnelling Quality Index or Q value have been derived from RMR values and are presented in above Table. II.

#### Construction Methodology

The 're-excavated' tunnel length between chainages 30+161m and 30+185m was excavated up to full face profile (excluding invert) by 'top-heading and bench' mode of tunnel construction. Invert closing was not done along this tunnel length. Whereas, the heading portion only was excavated between chainages 30+186m and 30+215m. The primary tunnel support system was installed at an average distance of 3.4m behind the tunnel face. The height of overburden was 725m and capacity of primary tunnel system was 3.70 MPa (see Table. V).

#### Tunnel Geometry

The tunnel profile geometry with respect to Class – V and Class – VI profiles are presented in below Table III.

**Table III: Tunnel Geometry for Class – V and Class – VI profiles**

| Tunnel Profile Class | Height of excavation profile (m) | Width of excavation profile (m) | Height of crown from spring level (m) |
|----------------------|----------------------------------|---------------------------------|---------------------------------------|
| Class – V            | 9.36                             | 8.67                            | 5.08                                  |
| Class – VI           | 9.51                             | 8.82                            | 5.15                                  |

#### Tunnel Wall Displacement Data

Crown and wall displacement data from chainages 30+161m to 30+185m are collected from each tunnel profile sections with 1m interval (see Fig. 1). These data are presented in below Table. IV. The tunnel strain data between chainages 30+186m and 30+215m are not used because this portion was left excavated with top-heading portion only.

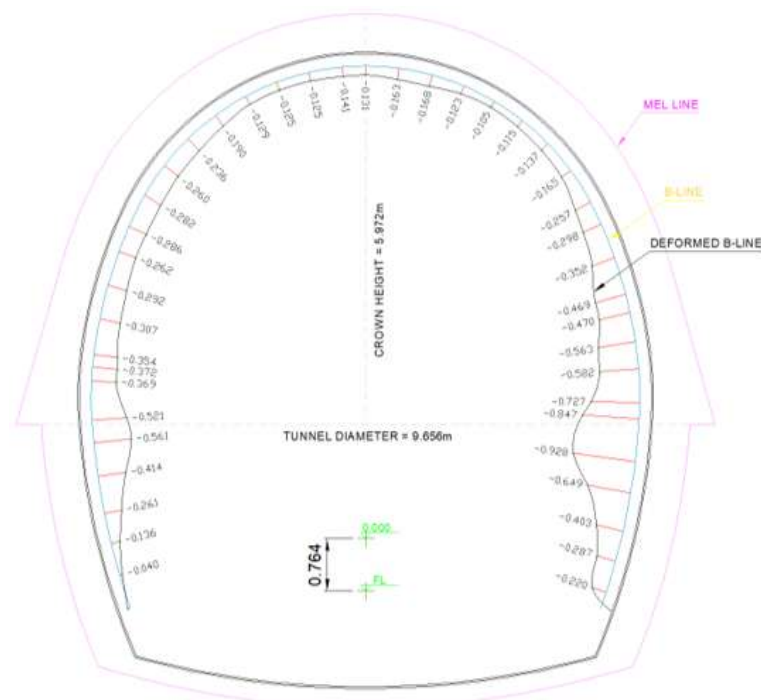


Figure. 1: Tunnel wall displacements at a single profile section

Table IV: Tunnel wall displacement data along squeezed tunnel

| Sl No | Chainage (m) | Tunnel Meter (m) | Tunnel convergence (mm) |      |     |
|-------|--------------|------------------|-------------------------|------|-----|
|       |              |                  | Crown                   | RSL  | LSL |
| 1     | 30+161       | 2010             | 169                     | 420  | 102 |
| 2     | 30+162       | 2011             | 285                     | 318  | 256 |
| 3     | 30+163       | 2012             | 483                     | 350  | 164 |
| 4     | 30+164       | 2013             | 464                     | 433  | 196 |
| 5     | 30+165       | 2014             | 433                     | 706  | 236 |
| 6     | 30+166       | 2015             | 120                     | 854  | 384 |
| 7     | 30+167       | 2016             | 124                     | 724  | 359 |
| 8     | 30+168       | 2017             | 209                     | 828  | 464 |
| 9     | 30+169       | 2018             | 131                     | 928  | 561 |
| 10    | 30+170       | 2019             | 140                     | 754  | 441 |
| 11    | 30+171       | 2020             | 293                     | 919  | 501 |
| 12    | 30+172       | 2021             | 228                     | 1054 | 449 |
| 13    | 30+173       | 2022             | 233                     | 980  | 352 |
| 14    | 30+174       | 2023             | 219                     | 908  | 330 |
| 15    | 30+175       | 2024             | 274                     | 988  | 146 |
| 16    | 30+176       | 2025             | 441                     | 1005 | 119 |
| 17    | 30+177       | 2026             | 475                     | 960  | 112 |
| 18    | 30+178       | 2027             | 384                     | 1004 | 115 |
| 19    | 30+179       | 2028             | 314                     | 777  | 111 |
| 20    | 30+180       | 2029             | 404                     | 608  | 172 |
| 21    | 30+181       | 2030             | 255                     | 511  | 251 |
| 22    | 30+182       | 2031             | 228                     | 463  | 246 |
| 23    | 30+183       | 2032             | 254                     | 408  | 287 |
| 24    | 30+184       | 2033             | 216                     | 218  | 187 |
| 25    | 30+185       | 2034             | 214                     | 293  | 150 |

#### Detail of Primary Tunnel Support System

The squeezing affected tunnel was supported with a primary tunnel support system comprising of Lattice Girder, Rock bolt, Wiremesh, Shotcrete etc (see Table. V).

**Table V: Support capacity of primary tunnel support system**

| Sl No | Element of support  | Particulars (Grade, Length, Thickness, Spacing etc)        | Support Pressure (Pi) MPa | Stiffness (K) MPa |
|-------|---------------------|------------------------------------------------------------|---------------------------|-------------------|
| 1     | Face shotcrete      | Grade - M25, Thickness - 50mm                              | Not applied               | Not applied       |
| 2     | face bolt           | Nil                                                        | Not applied               | Not applied       |
| 3     | 1st layer wiremesh  | Replaced by SFRS (M25/100mm)                               | 0.562                     | 195.1             |
| 4     | 2nd layer wiremesh  | Grade - Fe 500, Dimension - 150x150x6mm, 150mm overlapping | 0.562                     | 195.1             |
| 5     | Lattice Girder      | Type - 3 (P-72-32-36), Spacing 1.00m c/c                   | 1.069                     | 101.2             |
| 6     | 1st layer shotcrete | M25 / 100 mm, SFRS                                         | 0.562                     | 195.1             |
| 7     | 2nd layer shotcrete | M25 / 150 mm, plain shotcrete                              | 0.838                     | 210.3             |
| 8     | Rockbolt (Heading)  | 32mm dia, 6m long SDA, 8 & 7 nos staggerd pattern,         | 0.100                     | 14.471            |
|       | Rockbolt (Bench)    | 4m long SN, 2 & 4 nos staggerd pattern, fully grouted      |                           |                   |
| 9     | Forepoling          | 32mm SDA, 350mm spacing c/c (heading part only),           | 0.008                     |                   |
| 10    | Drainage hole       | Length unspecified, 3m x 3m staggered pattern              | 0                         | 0                 |
|       |                     | Total                                                      | 3.701                     | 911.27            |

## METHODOLOGY

With proven quantity of tunnel squeezing for Tunnel No. 10 (SRRP), a “back analysis” procedure is adopted to establish one or more hypotheses as root cause for such extreme tunnel squeezing. The steps for data analysis are presented below.

Method for Quantification and Classification of Tunnel Squeezing

To quantify tunnel squeezing ( $\varepsilon_t$ ) percentage, below equations are used.

$$\varepsilon_t = \frac{\delta_t}{B} \times 100 \quad (1)$$

Where  $\varepsilon_t$  is the percentage of tunnel strain,  $\delta_t$  is horizontal convergence in meters and B is the excavation width or diameter of the tunnel in meters according to Hoek and Merinos (2000).

$$\varepsilon_t = \frac{u_i}{r_o} \times 100 \quad (2)$$

Hoek (2001) further modified (1) and proposed (2), where  $\varepsilon_t$  is the percentage of tunnel strain,  $u_i$  is tunnel sidewall deformation and  $r_o$  is tunnel radius in meters.

The following Table. VI presents different category of tunnel squeezing behaviour based on Hoek and Marinos, (2000) and Hoek, (2001).

**Table VI: Category of Tunnel Squeezing**

| Sl No | Tunnel Strain (%) | Category of Tunnel Squeezing |
|-------|-------------------|------------------------------|
| 1     | < 1               | Non – squeezing              |
| 2     | 1 – 2.5           | Minor squeezing              |
| 3     | 2.5 – 5           | Severe squeezing             |
| 4     | – 10              | Very severe squeezing        |
| 5     | >10               | Extreme squeezing            |

## Quantification and Classification of Tunnel Squeezing for Tunnel No. 10 (SRRP)

To figure out overall tunnel strain percentage, the amount of horizontal convergences along 'spring level' of tunnel is collected from each squeezing affected tunnel profiles. Consequently the 'crown settlement' is calculated from the amount of vertical displacement at crown centreline. The tunnel squeezing percentages calculated along 24m of 'full face' tunnel length and tabulated below in Table. VII.

. Table VII: Squeezing category along Tunnel No. 10 (SRRP)

| SINO | Chainage (m) | Crown settlement (%) | RHS strain (%) | LHS strain (%) | Overall tunnel strain (%) | Tunnel squeezing category |
|------|--------------|----------------------|----------------|----------------|---------------------------|---------------------------|
| 1    | 30+161       | 2.14                 | 9.69           | 2.35           | 6.021                     | Very severe               |
| 2    | 30+162       | 3.61                 | 7.34           | 5.91           | 6.621                     | Very severe               |
| 3    | 30+163       | 6.12                 | 8.07           | 3.78           | 5.928                     | Very severe               |
| 4    | 30+164       | 5.88                 | 9.99           | 4.52           | 7.255                     | Very severe               |
| 5    | 30+165       | 5.49                 | 16.29          | 5.44           | 10.865                    | Extreme                   |
| 6    | 30+166       | 1.52                 | 19.70          | 8.86           | 14.279                    | Extreme                   |
| 7    | 30+167       | 1.57                 | 16.70          | 8.28           | 12.491                    | Extreme                   |
| 8    | 30+168       | 2.65                 | 19.10          | 10.70          | 14.902                    | Extreme                   |
| 9    | 30+169       | 1.66                 | 21.41          | 12.94          | 17.174                    | Extreme                   |
| 10   | 30+170       | 1.77                 | 17.39          | 10.17          | 13.783                    | Extreme                   |
| 11   | 30+171       | 3.68                 | 20.84          | 11.36          | 16.100                    | Extreme                   |
| 12   | 30+172       | 2.86                 | 23.90          | 10.18          | 17.041                    | Extreme                   |
| 13   | 30+173       | 2.92                 | 22.22          | 7.98           | 15.102                    | Extreme                   |
| 14   | 30+174       | 2.75                 | 20.59          | 7.48           | 14.036                    | Extreme                   |
| 15   | 30+175       | 3.44                 | 22.40          | 3.31           | 12.857                    | Extreme                   |
| 16   | 30+176       | 5.53                 | 22.79          | 2.70           | 12.744                    | Extreme                   |
| 17   | 30+177       | 5.96                 | 21.77          | 2.54           | 12.154                    | Extreme                   |
| 18   | 30+178       | 4.82                 | 22.77          | 2.61           | 12.687                    | Extreme                   |
| 19   | 30+179       | 3.94                 | 17.62          | 2.52           | 10.068                    | Extreme                   |
| 20   | 30+180       | 5.07                 | 13.79          | 3.90           | 8.844                     | Very severe               |
| 21   | 30+181       | 3.20                 | 11.59          | 5.69           | 8.639                     | Very severe               |
| 22   | 30+182       | 2.86                 | 10.50          | 5.58           | 8.039                     | Very severe               |
| 23   | 30+183       | 3.19                 | 9.25           | 6.51           | 7.880                     | Very severe               |
| 24   | 30+184       | 2.71                 | 4.94           | 4.24           | 4.592                     | Severe                    |
| 25   | 30+185       | 2.69                 | 6.64           | 3.40           | 5.023                     | Very severe               |
|      | Average      | 3.52                 | 15.89          | 6.12           | 11.00                     | Extreme                   |

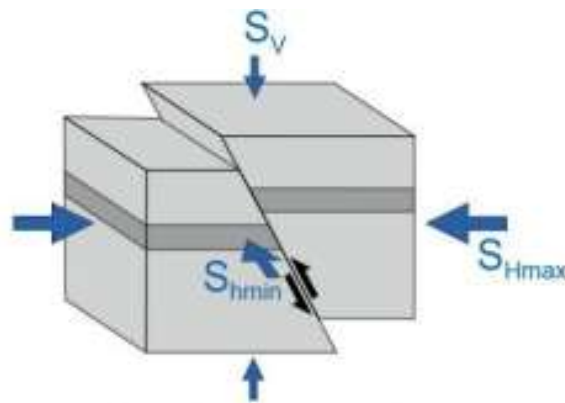
Therefore, it is established that the tunnel has suffered from a maximum 17.174% and an average 11.00% of tunnel squeezing. Hence the tunnel has suffered from "extreme category of tunnel squeezing" (see Fig. 2).



**Figure. 2: Extreme squeezing at Tunnel No. 10 (SRRP)**

#### Identification of Stress Regime

“In fact, the vertical stress component is generally the lowest principal stress, independently of the local tectonics, for the first few hundred metres depth in hard rock” (Hudson et al, 2003). Anderson (1951) identified a stress regime where  $S_{Hmax} > S_{hmin} > S_v$  for thrust fault regions (see Fig. 3). The tunnel project location is situated within the Himalayan tectonic region which is affected by all types of active tectonic activities including thrusting. Therefore, inference of a local stress regime where vertical stress direction ( $S_v$ ) coincides with the minimum principal stress direction ( $\sigma_3$ ) is valid. Furthermore, the proximity of Main Boundary Thrust (MBT) conforms the assumption of existence of a compressional stress regime where ratio of average horizontal stress to vertical stress is greater than one.



**Figure. 3: Compressional or Thrust Fault Stress Regime**

Furthermore, the position of cavity formation at crown overhead in competent rock mass as well as deformation of lattice girder along crown centreline (see Fig. 4 and Fig. 5) indicate the existence of larger horizontal stress relative to vertical stress in the tunnelling ground. Fossen (2016) proposed this aforesaid principle to identify in-situ stress directions for any tunnel construction operation.

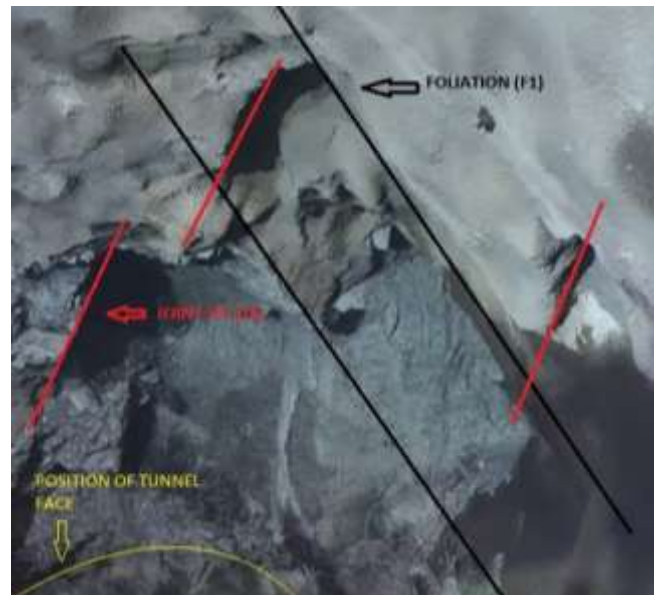


Figure. 4: Cavity formation at Crown in competent rock mass



Figure. 5: Deformation of Lattice Girder along crown centerline

#### Estimation of Vertical Stress ( $\sigma_v$ )

The average height of overburden above the specified length of tunnel is calculated as 725 m. Taking the average value of unit weight of rock as  $0.027 \text{ MN/m}^3$ , the vertical stress magnitude becomes 19.575 MPa (after Hoek and Brown, 1978).

#### Estimation of Earth Pressure Co-efficient (k) and in-situ Stress ( $P_0$ )

AECOM Asia Company Limited, in their Geotechnical Interpretative Report for Tunnel No. 4 (along the same project, i.e. SRRP), considered a value of k (ratio between horizontal and vertical stresses) bigger than 1.0, with principal tensor NNE – SSW oriented.

The earth Pressure Co-efficient (k) is the ratio of average horizontal stress to vertical stress. There is more than one empirical relationship to determine it. To get the horizontal and vertical stress magnitudes, the tunnel strains from crown settlement and side wall displacements are converted to in-situ stresses using the following relationship, suggested by Hoek and Marinos, (2000).

$$\varepsilon (\%) = 0.2 \left( \frac{\sigma_{cm}}{P_0} \right)^{-2} \quad (3)$$

Where,  $\varepsilon$  is the percentage of tunnel squeezing,  $\sigma_{cm}$  is UCS of rock mass and  $P_0$  is the magnitude of in situ stress. Using the average value of  $\sigma_{cm} = 2.18 \text{ MPa}$  from Table. I, the in-situ stresses corresponding to tunnel squeezing

at each chainage (see Table. VII) are evaluated along the full-face excavated tunnel between 30+161m and 30+185m and are presented in below Table. VIII.

| Table VIII: Conversion of Strain (%) to in-situ Stress (MPa) |              |                                  |                            |                            |                                       |
|--------------------------------------------------------------|--------------|----------------------------------|----------------------------|----------------------------|---------------------------------------|
| Sl No                                                        | Chainage (m) | Stress by Crown settlement (MPa) | Stress by RHS strain (MPa) | Stress by LHS strain (MPa) | Stress by Overall tunnel strain (MPa) |
| 1                                                            | 30+161       | 7.13                             | 15.18                      | 7.48                       | 11.96                                 |
| 2                                                            | 30+162       | 9.26                             | 13.20                      | 11.85                      | 12.54                                 |
| 3                                                            | 30+163       | 12.06                            | 13.85                      | 9.48                       | 11.87                                 |
| 4                                                            | 30+164       | 11.82                            | 15.41                      | 10.37                      | 13.13                                 |
| 5                                                            | 30+165       | 11.42                            | 19.68                      | 11.38                      | 16.07                                 |
| 6                                                            | 30+166       | 6.01                             | 21.64                      | 14.51                      | 18.42                                 |
| 7                                                            | 30+167       | 6.11                             | 19.92                      | 14.03                      | 17.23                                 |
| 8                                                            | 30+168       | 7.93                             | 21.31                      | 15.95                      | 18.82                                 |
| 9                                                            | 30+169       | 6.28                             | 22.56                      | 17.54                      | 20.20                                 |
| 10                                                           | 30+170       | 6.49                             | 20.33                      | 15.55                      | 18.10                                 |
| 11                                                           | 30+171       | 9.35                             | 22.26                      | 16.43                      | 19.56                                 |
| 12                                                           | 30+172       | 8.25                             | 23.84                      | 15.56                      | 20.13                                 |
| 13                                                           | 30+173       | 8.34                             | 22.98                      | 13.77                      | 18.95                                 |
| 14                                                           | 30+174       | 8.08                             | 22.12                      | 13.34                      | 18.27                                 |
| 15                                                           | 30+175       | 9.04                             | 23.08                      | 8.87                       | 17.48                                 |
| 16                                                           | 30+176       | 11.47                            | 23.27                      | 8.01                       | 17.40                                 |
| 17                                                           | 30+177       | 11.90                            | 22.75                      | 7.77                       | 17.00                                 |
| 18                                                           | 30+178       | 10.70                            | 23.26                      | 7.87                       | 17.37                                 |
| 19                                                           | 30+179       | 9.68                             | 20.46                      | 7.73                       | 15.47                                 |
| 20                                                           | 30+180       | 10.98                            | 18.10                      | 9.63                       | 14.50                                 |
| 21                                                           | 30+181       | 8.72                             | 16.60                      | 11.63                      | 14.33                                 |
| 22                                                           | 30+182       | 8.25                             | 15.80                      | 11.52                      | 13.82                                 |
| 23                                                           | 30+183       | 8.70                             | 14.83                      | 12.44                      | 13.69                                 |
| 24                                                           | 30+184       | 8.03                             | 10.84                      | 10.04                      | 10.45                                 |
| 25                                                           | 30+185       | 7.99                             | 12.57                      | 8.99                       | 10.93                                 |
|                                                              | Avg.         | 8.96                             | 19.03                      | 11.67                      | 15.91                                 |

The stress regime identified as  $S_{Hmax} > S_{Hmin} > S_v$  that clearly represents a compressional or thrust fault stress regime. From the above table, the value of Earth Pressure Co-efficient (k) is evaluated as 1.716.

Estimation of Complete in-situ Stress Tensor

With respect to the overburden height of 725m above the squeezing effected tunnel length, and a value of Earth Pressure Co-efficient (k) as 1.716, the complete in situ stress tensor is estimated and furnished in below Table. IX. The directions are collected from the World Stress Map 2025.

**Table IX: Complete in-situ stress tensor**

| Name of Stress                            | Magnitude (MPa) | Direction  |
|-------------------------------------------|-----------------|------------|
| Maximum Horizontal Stress / $\sigma_1$    | 38.696          | NNE - SSW  |
| Minimum Horizontal Stress / $\sigma_2$    | 28.470          | WNW - ESE  |
| Vertical Stress / $\sigma_v$ / $\sigma_3$ | 19.575          | Vertical   |
| Average Horizontal Stress ( $P_0$ )       | 33.583          | Horizontal |
| Earth Pressure Co-efficient (k)           | 1.716           |            |

## RETROSPECTIVE ANALYSIS OF TUNNEL SQUEEZING

With available rock strength data and estimated in situ rock stress data, retrospective analysis for tunnel squeezing behaviour have been exercised to cross check it's apprehension. In doing so, empirical, semi-empirical, analytical as well as numerical methods have been adopted.

## Empirical Analysis

Goel et al (1995), proposed the below expression for a tunnel squeezing to occur.

$$H \gg (275N^{0.33}).B^{-0.1} \text{ meter} \quad (4)$$

Where,

H is height of overburden,

N is 'Rock Mass Number' that is a variant of Rock Tunnelling Quality Index or Q value where SRF=1 always.

B is tunnel span or diameter.

With the input values of H = 725m, N = 0.52 and B = 8.820m, the (4) suggests the height of overburden, required for onset of tunnel squeezing is 178 m.

**Table X: Tunnel Squeezing Prediction (Singh and Goel, 1999)**

| SI No | Degree of squeezing                                                                      | Normalized tunnel closure (%) |
|-------|------------------------------------------------------------------------------------------|-------------------------------|
| 1     | Very mild squeezing<br>( $270N^{0.33}$<br>$B^{-0.1} < H < 360N^{0.33} B^{-0.1}$ )        | 1 – 2                         |
| 2     | Mild squeezing<br>( $360N^{0.33}$<br>$B^{-0.1} < H < 450N^{0.33} B^{-0.1}$ )             | 2 – 3                         |
| 3     | Mild to moderate squeezing<br>( $450N^{0.33}$<br>$B^{-0.1} < H < 540N^{0.33} B^{-0.1}$ ) | 3 – 4                         |
| 4     | Moderate squeezing<br>( $540N^{0.33}$<br>$B^{-0.1} < H < 630N^{0.33} B^{-0.1}$ )         | 4 – 5                         |
| 5     | High squeezing<br>( $630N^{0.33}$<br>$B^{-0.1} < H < 800N^{0.33} B^{-0.1}$ )             | 5 – 7                         |
| 6     | Very high squeezing<br>( $800N^{0.33}$<br>$B^{-0.1} < H$ )                               | >7                            |

Also, according to the above Table. X after Singh & Goel (1999), with overburden of 725m and same rock mass characteristic, the apprehension for normalized tunnel squeezing is greater than 7%.

## Semi-empirical Analysis

Hoek and Marinos (2000), proposed the following expression to determine tunnel convergence for supported tunnel.

$$\varepsilon_t = \left(0.2 - 0.25 \times \frac{p_i}{\sigma_v}\right) \times \left[\frac{\sigma_{cm}}{\sigma_v}\right]^{(2.4 \times \frac{p_i}{\sigma_v} - 2)} \quad (5)$$

With the input data of  $\sigma_{cm} = 2.18$  MPa,  $\sigma_v = 19.575$  MPa,  $P_0 = 26.58$  MPa and  $P_i = 3.70$  MPa (see Table. V), the above (5) reveals that with respect to vertical stress, it gives a tunnel squeezing of 4.55% that does not match with the reality. However, if analysis is carried out with  $P_0 = 26.58$  MPa instead of Vertical Stress, the resultant tunnel strain becomes 10.65% that closely matches with the average tunnel strain of 11.00% (see Table. VII). This result once again testifies that there is larger stress magnitude than vertical stress, acting on the tunnelling ground.

## Analytical Analysis (Convergence Confinement Method)

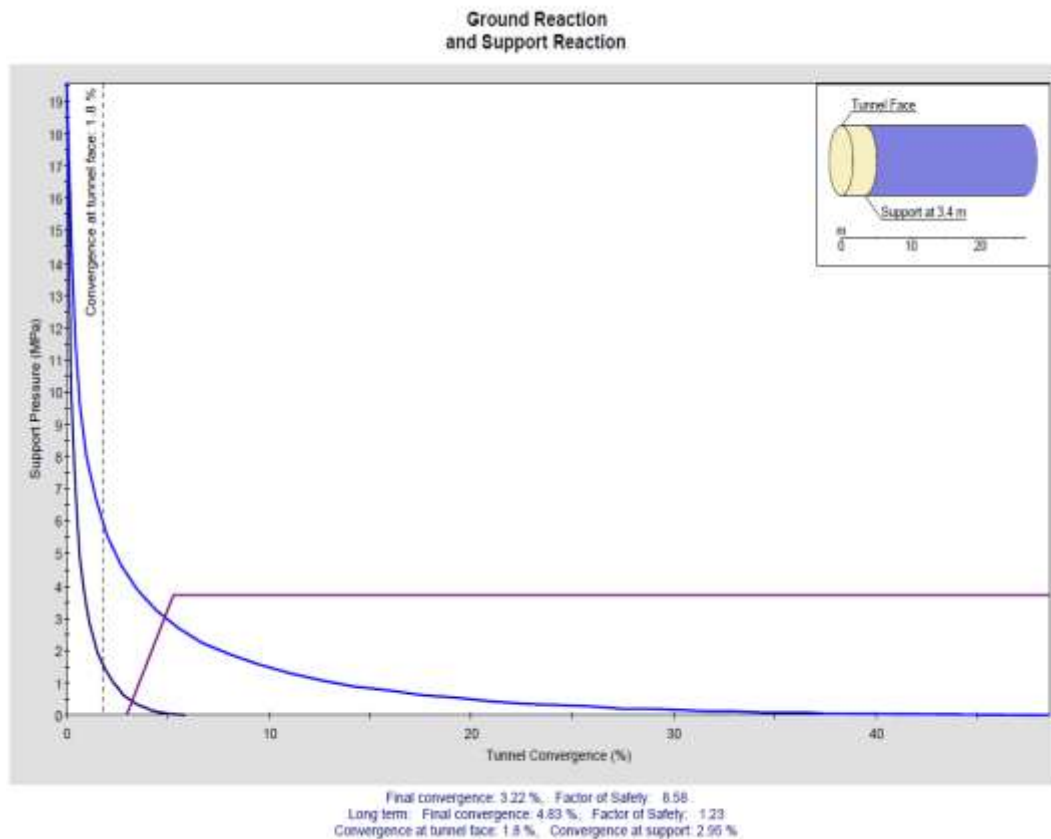
Although the distribution of ground pressure and internal displacement of tunnel crown and wall due to tunnelling is essentially a 3-dimensional phenomenon, it can be represented by a 2-dimensional graphical method called the Convergence Confinement Method (CCM). This method shows the inter-relationship between ground pressure and inward peripheral tunnel deformation along the tunnel between twice tunnel diameter length, behind and ahead of tunnel face. Initially this method was proposed for circular tunnel under hydrostatic stress condition. This method is a simple tool to determine the support pressure, required to stabilize a tunnel with respect to any acceptable quantity of inward tunnel wall displacement. The CCM basically consists of 3 curves, namely Ground Reaction Curve (GRC), Support Characteristic Curve (SCC) and Longitudinal Deformation Profile (LDP).

As the CCM is originally applicable for circular tunnel under hydrostatic stress condition, Carranza-Torres and Fairhurst (2000), suggested a method to convert asymmetrical in situ ground stress values to a hydrostatic form so that it can be used for non-circular tunnels too. The formula is mentioned below.

$$P_0 = \{[(\sigma_1 + \sigma_2) / 2] + \sigma_3\} / 2 \quad (6)$$

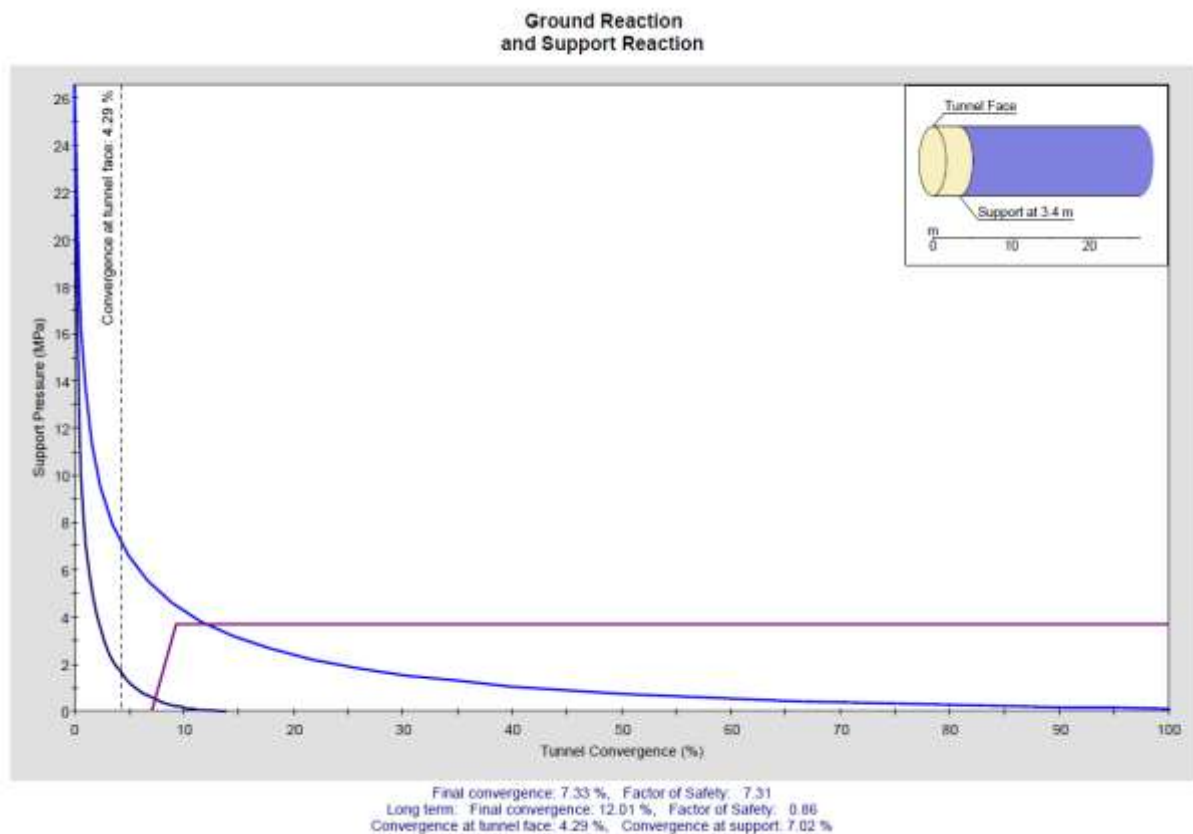
By putting the stress values from Table. IX, the above (6) yields a value of  $P_0 = 26.58$  MPa that exactly matches with the in-situ stress value, inferred by Semi-Empirical Method corresponding to 10.65% tunnel squeezing.

One major drawback of this method is that it does not take the effect of time dependent deformation or ‘creep’ deformation into account while estimating the tunnel support pressure. However, this fault can be rectified by constructing the ‘Long Term Ground Reaction Curve’ to accurately predict the tunnel support pressure under squeezing ground condition. Lajtai and Schmidtke (1986) and Lajtai and Bielus (1986) suggested “under long term constant stress loading, most rock suffers a reduction in strength of about 50%”. Therefore, by reducing the rock strength values in Table. I by 50%, a GRC can be constructed that shows long term deformation behaviour of the tunnelling ground under constant loading condition. Such GRC is called as ‘Long-Term GRC’ and can be used to predict support pressure under squeezing tunnelling ground condition.



**Figure. 6: Convergence Confinement analysis with respect to vertical stress only**

Therefore, with the ‘long-term rock strength’ values (equal to 50% of ‘short term rock strength’ values presented at Table. I) and ‘in situ rock stress’ ( $P_0$ ) = 26.58 MPa, Convergence – Confinement analysis is exercised by constructing ‘Long-Term GRC’ and SCC by using RocSupport software. The Support Characteristic Curve (SCC) is constructed from the support pressure and stiffness values of all components of primary tunnel support system. With respect to the Vertical Stress only, the CCM analysis (see Fig. 6) reveals the potential of 4.83% tunnel squeezing and Factor of Safety of 1.23 for installed tunnel support system. This once again proves that there exists larger stress magnitude relative to the vertical stress. Under long-term constant loading condition or ‘creep’ deformation, the rock mass surrounding the tunnel lost 50% of its original strength. Under larger horizontal stresses, the tunnel exhibits potential for 12.01% tunnel squeezing (see Fig. 7). For long term tunnel behaviour, no equilibrium state was attained between the ground deformation and the tunnel support elements deformation. The installed tunnel support system is found to be neither competent enough nor flexible enough to ensure the tunnel stability. For long term tunnel stability, the installed tunnel support system yields a Factor of Safety equal to 0.86 and therefore was not stable.



**Figure. 7: Convergence Confinement analysis with respect to Long – Term GRC**

#### Numerical Analysis (Finite Element Method)

Numerical analysis by Phase 2 (RS2) software from RocScience is adopted for the estimation of tunnel deformation with installed primary tunnel support system. The properties of rock mass for numerical modelling are collected from the Geotechnical Interpretative Report for Tunnel No. 10 that are furnished in Table. I.

2D plain strain, finite element analyses are performed twice to simulate tunnelling ground behaviour to know the total tunnel wall displacement. First time with respect to the 'short term strength' of rock mass and second time with 'long term strength' of rock mass that are obtained from 50% reduction of 'short term strength' parameters relevant to the effect of 'creep deformation' under constant loading condition. In both the cases, in-situ rock stress ( $P_0$ ) and internal support pressure ( $P_i$ ) values remain unchanged as 26.58 MPa and 3.7 MPa respectively. The results of FEM analysis are presented in Fig. 8 and Fig. 9.

In spite of installation of tunnel support system, the tunnelling ground shows tendency of tunnel squeezing. The FEM simulation indicates potential of 4.09% average tunnel strain without considering the effect of time dependent 'creep' deformation (see Fig. 8) Whereas, incorporating the effect of 'creep' deformation, the software simulation reveals possibility of 10.91% tunnel squeezing (see Fig. 9). Thus, it is proved that the rock mass surrounding the tunnel has been subjected to time dependent deformation or 'creep' deformation under constant loading condition and have undergone an overall 50% strength reduction.

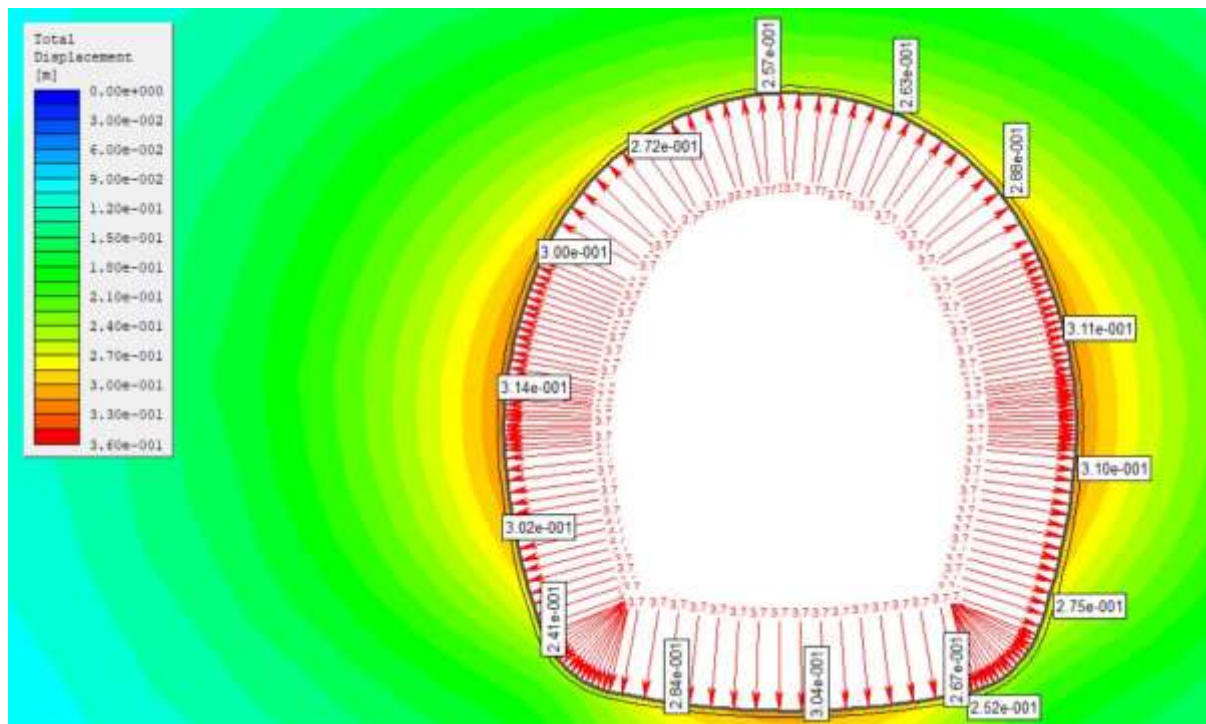


Figure. 8: Finite Element Analysis without considering the effect of 'creep deformation'

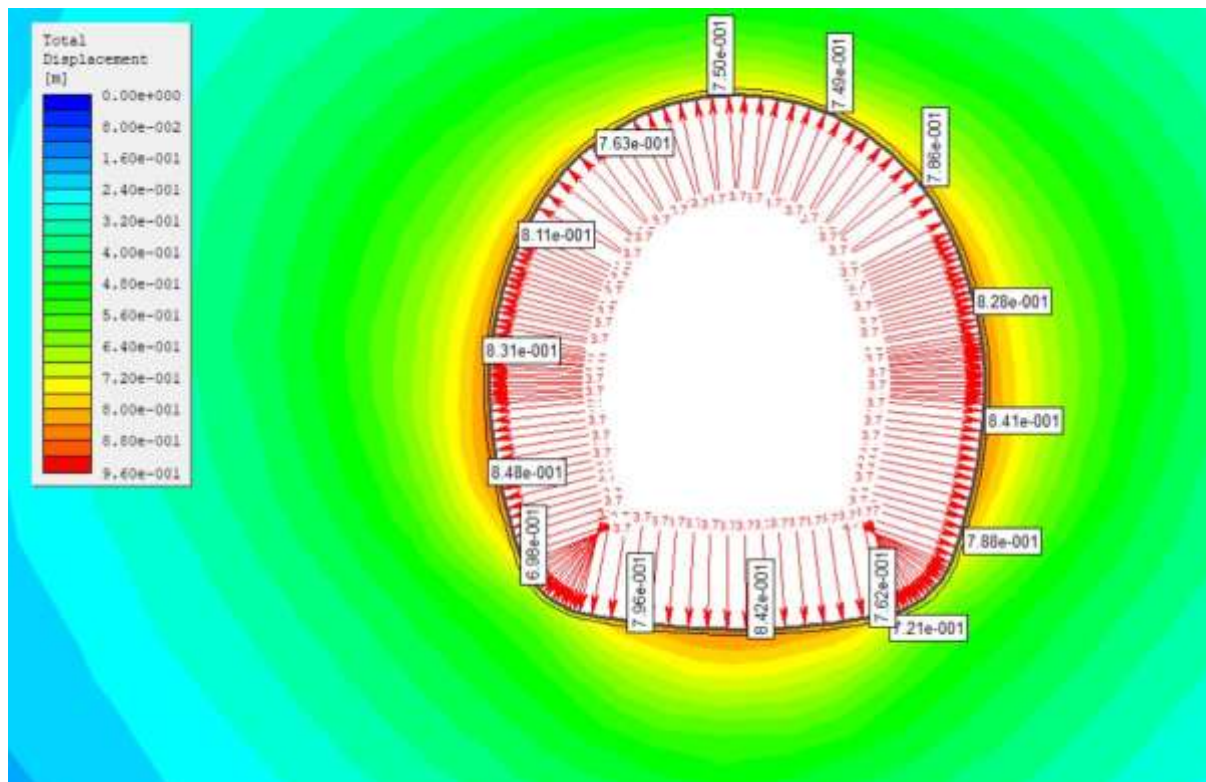


Figure. 9: Finite Element Analysis considering the effect of 'creep deformation'

## CONCLUSION

The following conclusions can be drawn out of the foregoing research work.

There exists larger horizontal ground stress relative to vertical stress or overburden pressure. The ratio of average horizontal stress to vertical stress (Earth Pressure Co-efficient,  $k$ ) is estimated to be 1.716.

There prevails a 'constant loading condition' within the tunnelling ground under the influence of high in situ stresses. It has caused time dependant deformation or 'creep' deformation to the tunnelling ground.

Under such constant loading condition, the rock mass strength was reduced to half of its original strength.

The lithology, orientation of discontinuities, in-situ ground stress condition and strength / deformability of rock mass surrounding the tunnel are found to be enough susceptible to bring up tunnel squeezing.

For the Factor of Safety of installed primary tunnel support system, the Semi-Empirical analysis yields a value of 0.41 and Analytical (CCM) analysis yields a value of 0.86 respectively. It indicates that it was neither strong enough nor flexible enough to resist the tunnel squeezing against 'creep' deformation.

Retrospective analysis for tunnel squeezing by Empirical, Semi-Empirical, Analytical as well as Numerical Methods clearly indicate the apprehension of 'extreme (>10% tunnel strain)' tunnel squeezing along the re-excavated portion of Tunnel No. 10 (SRRP).

In brief, the extreme tunnel squeezing of Tunnel No. 10 (SRRP) can be attributed to the following pivoting factors.

Existence of larger 'in situ ground stress' magnitude relative to vertical stress or overburden pressure.

Reduction of overall rock mass strength relative to initially determined strength values, due to constant loading condition.

Installation of primary tunnel support system with inadequate support capacity as well as flexibility to accommodate such extreme tunnel squeezing pressure.

For future tunnelling under similar geotechnical environment at Tunnel No. 10 (SRRP), following recommendations would be relevant.

Measurement of 'in situ rock stress' by either Over-coring method or Hydraulic Fracturing (HF) in combination with Hydraulic Testing in Pre-existing Fractures (HTPF) methods.

Use of Geo-physical methods such as Tunnel Seismic Prediction (TSP) is recommended to identify precise character of tunnelling ground.

Excavation of about 11% extra cross-sectional area of the tunnel.

Installation of 'yielding tunnel support system' that should be strong as well as flexible enough to accommodate approximately 11% tunnel closure, with concurrent invert closing.

With respect to 800m overburden, the value of in situ stress magnitude should be considered as 29.33 MPa to determine the requirement of tunnel support capacity.

The primary tunnel support system should be installed adjacent to the tunnel face to enforce equilibrium state between ground behaviour and support behaviour.

## ACKNOWLEDGEMENT

The authors express their utmost gratitude and respect to Mr. Mohinder Singh (Project Director, SRRP, Ircon International Limited), Mr. Pratap Chandra Dhang (DGM, Geology, SRRP, Ircon International Limited), Mr. Subhendu Ghosh (Project Director, Cemindia Projects Limited, formerly ITD Cementation India Limited), Mr. Abhijit Pyne (General Manager, Cemindia Projects Limited, formerly ITD Cementation India Limited), Dr. Sayantan Chakraborty (Deputy Manager, Cemindia Projects Limited, formerly ITD Cementation India Limited) for their active help and encouragement to take up this whole research work to a success....

## References

- [1] G. L. Shrestha, Ph.D. thesis "Stress induced problems in Himalayan tunnels with special reference to squeezing", 2006
- [2] E. Hoek, and P. Marinos, "Predicting tunnel squeezing problems in weak heterogeneous rock masses." Tunnels Tunnel Int; 32 (11 and 12) p. 45–51 and p. 34–36, 2000
- [3] E. T. Brown, and E. Hoek, "Trends in relationships between measured in-situ stresses and depth." In International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts (Vol. 15, No. 4, pp. 211-215). Pergamon, 1978
- [4] Md. Hadi Alam, and M. K. Sharma, "Criticalities encountered in Construction of (HRT) Head Race Tunnel of Kameng Hydro Electric Project (600MW), In Arunachal Pradesh, India." International Journal of Scientific & Engineering Research, Volume 5, Issue 9, ISSN 2229-5518, September 2014
- [5] R. K. Goel, J. L. Jethwa, and A. G. Paithankar, "Indian experiences with Q and RMR systems." Tunnelling and Underground Space Technology Incorporating Trenchless, 10 (1), 97–109, 1995
- [6] C. Carranza-Torres, and C. Fairhurst, "Application of the convergence confinement method of tunnel design to rock masses that satisfy the Hoek-Brown failure criteria." Tunnelling and underground space Technology, 15(2), Pages 187-213, 2000

- [7] N. Vlachopoulos, and M. S. Diederichs, "Improved longitudinal displacement profiles for convergence confinement analysis of deep tunnels." *Rock mechanics and rock engineering* 42.2: 131-146, 2009
- [8] B. Singh, and R. K. Goel, "Rock Mass Classification - A Practical Approach in Civil Engineering", 1999
- [9] J. A. Hudson, F. H. Cornet, and R. Christiansson, "ISRM Suggested Methods for rock stress estimation—Part 1: Strategy for rock stress estimation," *Rock Mech. Rock Eng.* 2003
- [10] E. M. Anderson, "The dynamics of faulting and dike formation with application to Britain." Oliver and Boyd, 2nd Edition, Edinburgh, 133147, 1951
- [11] H. Fossen, "Structural geology", Cambridge, U.K., Cambridge University Press, 2016
- [12] "Geotechnical Interpretative Report for Tunnel T9 & T10."
- [13] "Geotechnical Interpretative Report for Tunnel T3 & T4."
- [14] Instrumentation & Monitoring Reports for Tunnel T9 & T10, for the months from November 2023 to December 2024.
- [15] RESS (before re-excavation) issued by IRCON.
- [16] DEFORMATION REPORT prepared by Client, Designer and Contractor before re-excavation of Tunnel No 10 (SRRP).
- [17] <https://www.world-stress-map.org..>

..