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A Technical Note on

Monitoring of Deformation and Assessment of Support System During Tunnel Excavation

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1. INTRODUCTION

Before the tunnel is excavated, the in situ stresses σ_v, σ_{h1} and σ_{h2} are uniformly distributed in the slice of rock mass under consideration (Fig. 1). When an underground opening is excavated into a stressed rock mass, the stresses in the vicinity of the new opening are re-distributed and the new stresses are induced. Three induced principal stresses σ_1 , σ_2 and σ_3 acting on a typical element of rock are shown in Fig. 1.

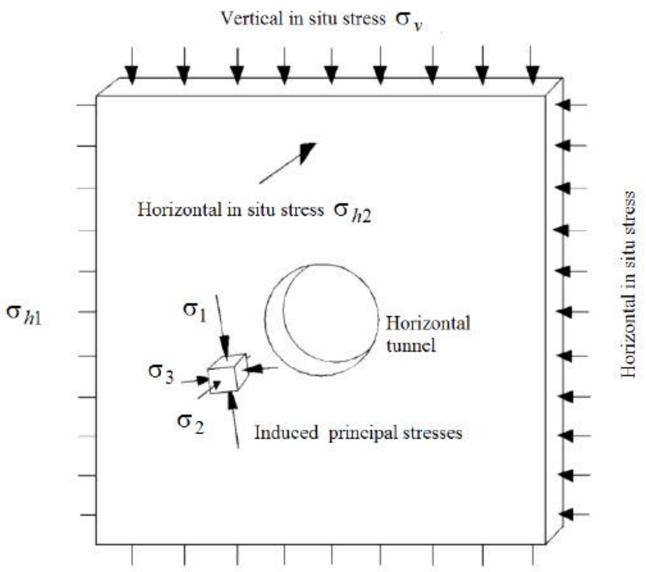

Fig. 1 - Illustration of in situ stresses and induced stresses around a circular opening (Hoek et al., 2014)

During the stress re-distribution process, the rock mass starts moving towards the tunnel opening. This deformation/convergence of rock mass in the tunnel is an important factor for evaluating the tunnel behavior. Apart from the convergence of tunnel periphery, the deformation within the rock mass is equally important to be known sometimes when the rock mass is heterogeneous or some weak zones are present. Thus, knowledge about the displacement/convergence of rock mass, including the deformation, are important to ascertain the stability of tunnel and implementing appropriate excavation and support measures.

Empirical, observational and the analytical are the three approaches for the tunnel design. New Austrian Tunnelling Method (NATM) is based on the observational approach. Success of observational approach depends on the reliable instrumentation, monitoring and analysis of data and timely implementation of results.

The instrumentation based observational method is an accepted alternative to conventional design methods for geotechnical structures. The observational method is an approach for managing uncertainty in tunnel design through heterogeneous rock masses of unknown behaviour. In case of empirical and analytical approaches also, instrumentation helps in evaluation of approaches and to apply corrections accordingly when prediction of geology along deep tunnels and geotechnical behaviour is difficult. In observational method, the design is reviewed during construction.

Thus, one of the most important factors in the construction of tunnels is the observation of rock mass behavior during construction and accordingly upgrading or modifying the support design. Monitoring and interpretation of convergence, deformations, strains and stresses and accordingly optimizing working procedures and support requirements are important specially in weaker rock masses.

Instrumentation and monitoring are an integral part of the contract document now-a-days, which shall be implemented seriously to get its full benefits. In this technical note deformation/ convergence in tunnels and its role in the optimization of tunnelling is presented.

2. INFLUENCE ZONE AND ROCK MASS DEFORMATION DUE TO TUNNELLING

There will be no effect of tunnel excavation beyond a point in the rock mass around the tunnel. At this point the in situ stresses would remain unaffected due to tunnelling. The zone up to which the in situ stresses are disturbed because of tunnel excavation is known as the 'zone of influence'. The rock mass in the zone of influence, depending upon the rock mass properties, induced stresses and the tunnel size, would deform and thus affected by the tunnel excavation. The rock mass deformation would be maximum at the tunnel periphery and negligible at the boundary of zone of influence.

2.1 Deformation Profile Along the Tunnel, GRC and SCC

The deformation profile along the tunnel is a graph that represents the radial displacement of the ground at the tunnel periphery at a given time during the tunnel excavation. It is used to determine the unsupported tunnel distance from the excavation face and considered important for determining the support installation distance and timing in the convergence-confinement method (Ha et al., 2021). Figure 2 shows the convergence-confinement plot, which has the deformation along the tunnel, the ground reaction curve (GRC) and the support reaction/characteristic curve (SCC).

Important points in Fig. 2 and their brief description are as follows.

(I) Deformation profile along the tunnel

- Point F: Convergence in the tunnel face
- Point I: Convergence at a location separated by distance L behind the tunnel face, ur^0

(II) Ground reaction curve

- Point O: Initial state, pi = σ_0 = P_o, ur = 0 (pi is internal pressure; σ_0 is vertical stress; P_o is overburden pressure and ur is radial convergence)
- Point M: Final point of reach if there are no support members, pi = 0, ur = ur^m (ur^m is maximum radial convergence)
- Point E: When a plastic area is created around the tunnel excavation surface, pi = pi^{cr}
- Point N: Virtual ground pressure when installing the support members.

(III) Support characteristic curve

- Point K: The time of installing the support members, ps = 0, ur = ur^0 (ps is support pressure)
- Point D: Status of ground pressure exerted by the additional excavation after installing the support members, ps = ps^D
- Point R: Yield of support members, ps = ps^{max} (ps^{max} is maximum support pressure)

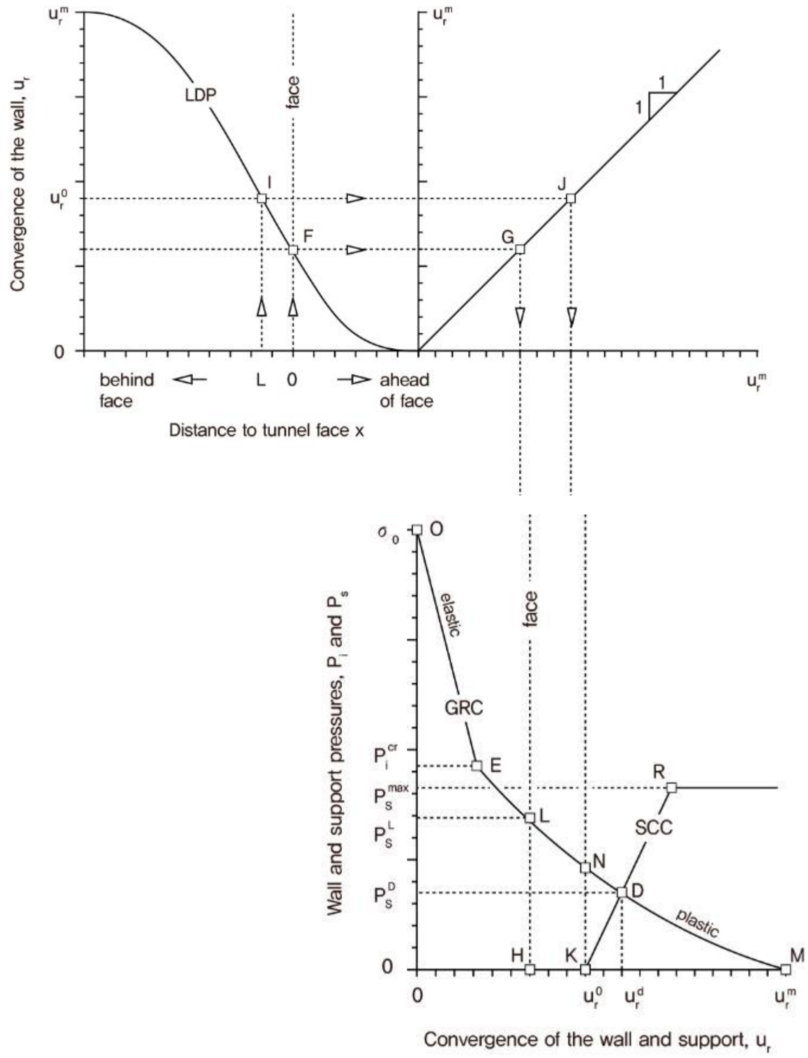

Fig. 2 - Schematic presentation of the deformation along tunnel, ground reaction curve (GRC) and support characteristic curve (SCC) [Carranza-Torres & Fairhurst, 2000 & Ha et al., 2021]

The trend of GRC depends on the geology, ground conditions, excavation methods, and installed support.

2.2 Deformation along the Tunnel with Moving Tunnel Face

Hoek et al. (2014) have given Fig. 3, which shows the variation in deformation ahead and behind the moving tunnel face of an unsupported tunnel. Measurable displacement in the rock mass begins at a distance of about one half a tunnel diameter ahead of the face. The displacement increases gradually and, when the tunnel face is coincident with the measuring point, the radial displacement is about one third of the maximum value. The displacement, in general, reaches a maximum when the face has progressed about one to one and half tunnel diameters beyond the measuring point and the support provided by the tunnel face is no longer effective.

2.3 Elastic and Plastic Conditions

Up to point 'E' in the plot of GRC in Fig. 2, the rock mass behaves elastically (shown by the straight-line trend OE). But, beyond this point the rock mass started behaving plastically. The plastic failure of the rock mass surrounding the tunnel does not necessarily mean that the tunnel collapses. The failed material still has considerable strength and, provided that the thickness of the plastic zone is small compared with the tunnel radius, the only evidence of failure may be a few fresh cracks and a minor amount of ravelling or spalling. On the other hand, when a large plastic zone is formed and when large inward displacements of the tunnel wall occur (as in squeezing ground condition), the loosening of the failed rock mass will lead to severe spalling and ravelling and to an eventual collapse of an unsupported tunnel. The primary function of support is to control the inward displacement of the roof and the walls and to prevent the loosening, which can lead to collapse of the tunnel. The installation of rockbolts, shotcrete lining or steel sets cannot prevent the failure of the rock surrounding a tunnel subjected to significant overstressing; but these support types do play a major role in controlling tunnel deformation. A graphical summary of this concept is presented in Fig.4 (Hoek et al., 2014).

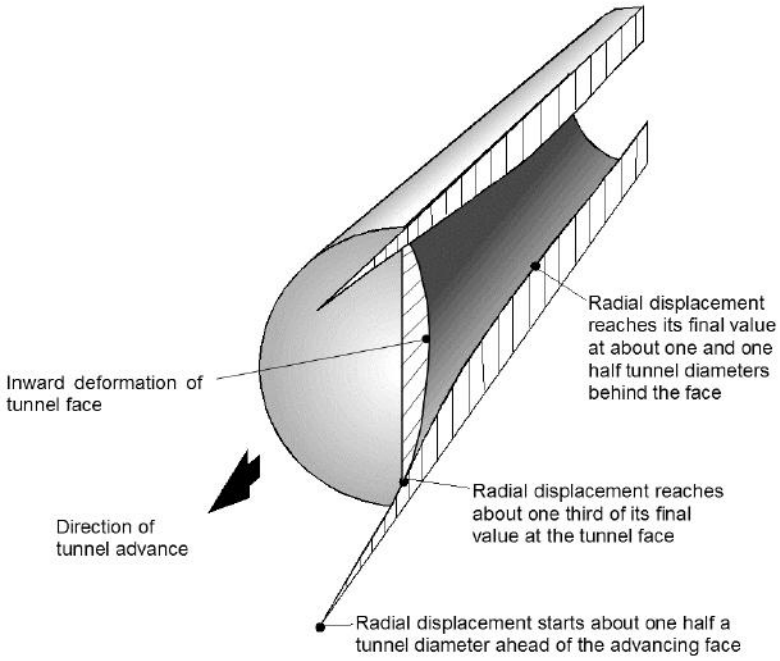

Fig. 3 - Deformation scenario in the rock mass surrounding an advancing unsupported tunnel (Hoek et al., 2014)

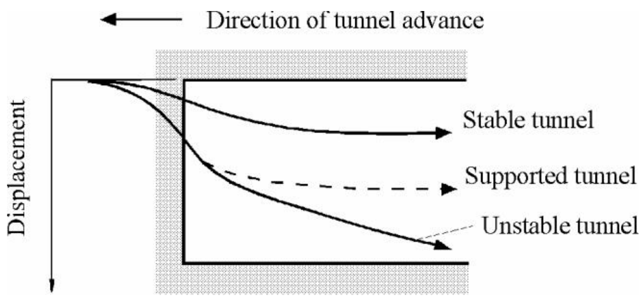

Fig. 4 - Displacement plots for tunnel roof showing different stability conditions (Hoek et al., 2014)

The deformation is related to the rheological properties and creep potential of the surrounding rock mass. For the design of tunnels in rock masses at depth, it is often important to account for creep. In tunnelling, time-dependent behaviour (creep) is often observed in weak rock masses that exhibit squeezing conditions. The creep behaviour may extend through the initial construction period and beyond. The time effect can contribute up to 70% of the total deformation (Sulem et al., 1987).

2.4 Estimation of Deformation

Barton (2008) plotted the tunnel roof and wall deformations with Q on a log-log scale (Fig. 5) to develop equations for predicting the deformation or closure in underground opening. Not just as in SRF, he has also introduced the ‘competence factor’ (ratio of stress to strength) directly in the proposed Eqs. 1 and 2 as follows.

$$\Delta_v = \frac{\text{Span}}{100\ Q}\sqrt{\frac{\sigma_v}{q_c}} \tag{1}$$

$$\Delta_h = \frac{\text{Height}}{100\ Q}\sqrt{\frac{\sigma_h}{q_c}} \tag{2}$$

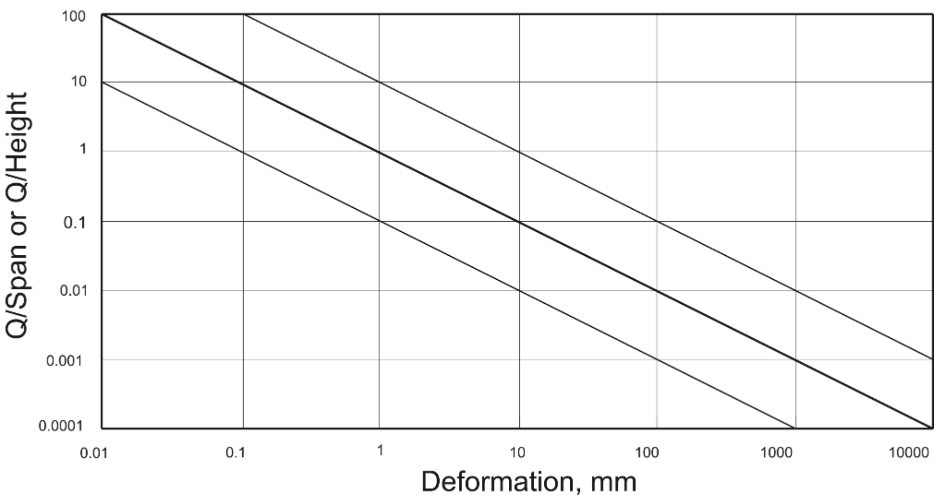

Fig. 5 - Deformation vs of Q/Span or Q/Height (Barton, 2008)

where,

Δ_v & Δ_h = Roof and wall deformations respectively,

σ_v & σ_h = in situ vertical and horizontal stresses respectively in MPa, and

q_c = UCS of intact rock material in MPa.

Barton and Grimstad (2014) have observed that the deformation estimated from the above Eqs. 1 & 2 in Nathpa-Jhakri power station cavern in India and Gjovik Olympic cavern in Norway are matching with the measured values.

Hoek et al. (2014) have given the following Eq. 3 to estimate the radial deformation/convergence in elastic condition (when support pressure p_i> critical support pressure p_{cr}).

$$u_{re}^{m} = \frac{r_o(1+\nu)}{E}(p_o - p_i) \tag{3}$$

where

u_{re}^{m}= Maximum elastic radial displacement,

r_o = tunnel radius,

ν = Poisson's ratio,

p_i = Support pressure,

p_o = Overburden pressure and

E = Deformation modulus of the rock mass.

Radial deformation in plastic condition (u_{ip}) when $p_i < p_{cr}$ is,

$$u_{ip} = \frac{r_o(1+\nu)}{E}\left[2(1-\nu)(p_o - p_{cr})\left(\frac{r_p}{r_o}\right)^2 - (1-2\nu)(p_o - p_i)\right] \tag{4a}$$

where

r_p = Radius of plastic zone (or the zone of influence),

$$r_o\left[\frac{2(p_o(k-1)+\sigma_{cm})}{(1+k)((k-1)p_i+\sigma_{cm})}\right]^{\frac{1}{(k-1)}} \tag{4b}$$

$$p_{cr} = \frac{2p_o - \sigma_{cm}}{1+k} \tag{4c}$$

$$\sigma_{cm} = \frac{2c\ cos\emptyset}{1-sin\emptyset} \tag{4d}$$

$$k = \frac{1+sin\emptyset}{1-sin\emptyset} \tag{4e}$$

ϕ = angle of internal friction, and

p_i & p_o = As defined for Eq. 3.

Panet and Guenot (1982) proposed convergence/displacement for elastic condition using the tunnel face distance (x) in the following form (Eq. 5) by using finite element analysis,

$$\frac{u_r}{u_r^m} = 0.28+0.72\left[1-\left(\frac{0.84}{0.84+{}^{x}/_{R_T}}\right)^2\right] \tag{5}$$

where,

u_r = Radial displacement at a distance 'x' from the face,

u_r^m = Maximum radial displacement and

R_T = tunnel radius.

Corbetta et al. (1991) proposed an empirical formula in the form of an exponential function as follows,

$$\frac{u_r}{u_r^m} = 0.29+0.71\left[1-exp\left(-1.5\left({}^{x}/_{R_T}\right)\right)^{0.7}\right] \tag{6}$$

Based on the measured data, Hoek (in Carranza-Torres & Fairhurst, 2000) proposed following best-fit empirical Eq. 7a.

$$\frac{u_r}{u_r^m} = \left[1+exp\left(\frac{-\left({}^{x}/_{R_T}\right)}{1.1}\right)^{-1.7}\right] \tag{7a}$$

Notations used in Eqs. 6 and 7 have been defined after Eq. 5.

Based on Eq.7a, Ha et al. (2021) have proposed the following generalized equation for elasto-plastic condition for estimating the deformation profile along the tunnel considering Bieniawski's RMR (from 5 to 90) and overburden pressure P_o (from 3 to 20 MPa).

$$\frac{u_r}{u_r^m} = \left[1+exp\left(\frac{-\left({}^{x}/_{R_T}\right)}{\alpha}\right)^{-\beta}\right] \tag{7b}$$

Where α and β are the parameters related to rock mass condition and overburden pressure or initial stress state as follows:

$$\alpha = 0.305\, ln\left({}^{P_o}/_{RMR^{1.5}}\right) + 2.419 \quad (\alpha \text{ value in the range of } 0.898 - 2.416) \tag{8}$$

$$\beta = 2.926\exp(-0.01RMR) \quad (\beta \text{ value in the range of } 1.362 – 2.851) \tag{9}$$

Equations 7, 8 and 9 may be used to estimate the rock mass deformation in the tunnel and accordingly plan for the supports and their optimisation using a monitoring program.

3. INSTRUMENTATION FOR MONITORING

3.1 Typical Instrumentation Section

Typical instrumented section in a tunnel where all the instruments, like multi-point borehole extensometer (MPBX), load cells, pressure cells and bireflex targets (BRT) for roof/wall deformation are installed to monitor the support and rock mass behaviour is shown in Fig. 6. The load cells and pressure cells/stress meters shall be replaced by rock bolt load cells in case of rock bolts and shotcrete supported sections. A few critical locations in tunnel shall have such complete instrumented section in the tunnel.

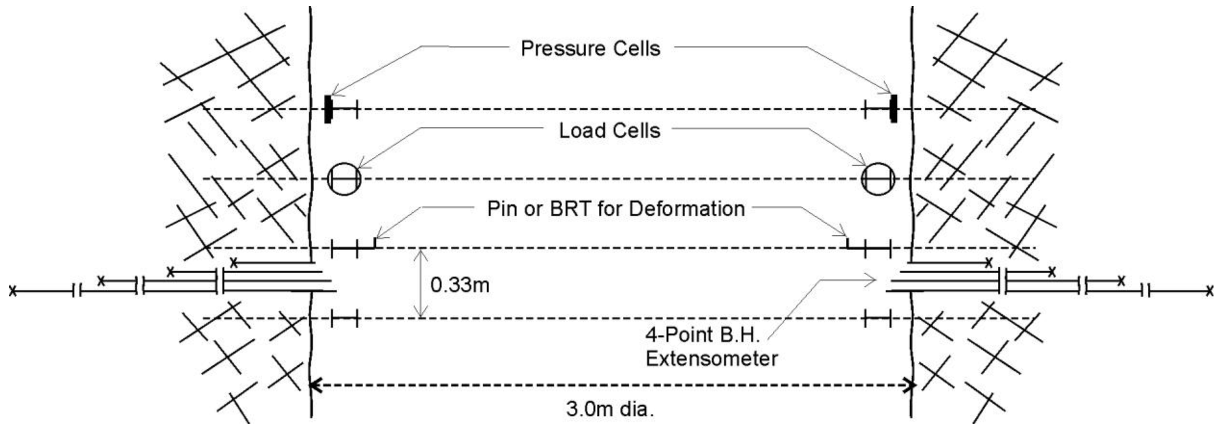

Fig. 6 – Typical instrumented section

Besides the above-mentioned instrumentation, following data should also be collected which is required while analyzing the instrument data.

A. Geology - mapping, fracture spacing and orientation, width of fracture zone, alteration and ground water
B. Rock mass quality (Q), rock mass rating (RMR) and geological strength index (GSI)
C. Geophysical observations - seismic activity, in situ stresses and their orientation, micro-seismic activity inside opening.
D. The time lag between excavation and installation of instrumentation shall be recorded along with a time record of further support installations or face advancements vis-a-vis observed deformations.

Note: Adequate number of instruments should be used as their survival rate is very low in tunnels in the weak rocks.

The instrumentation schemes are meant for some specific purposes. During the data analysis, it must be ensured that the data are analyzed to serve these purposes. Some of the important purposes are listed below,

(a) Provide data for selection of tunnel support capacity
(b) To ensure that the tunnel closure do not exceed the desired levels i.e. < 1 % of tunnel diameter in non-squeezing and > 1 % in squeezing ground condition.

(c) To investigate if the major discontinuity stable?
(d) To decide on time to provide concrete lining?

Basically, two sets of information are required while dealing with the tunnel support in an effective manner.

- Are the tunnel supports strong enough for the purpose for which they were installed?
- How the rock mass around the tunnel is behaving? Where does the rock load/pressure come from? Is the rock pressure due to loosening of the rock or due to squeezing of the rock or due to its swelling?

Timely answers to these questions are expected from the instrumentation and monitoring program as per the defined role and purpose so that effective control measures are taken well in time before the occurrence of any mishap.

3.2 Deformation Monitoring

3.2.1 Deformation of tunnel periphery

Systematic tunnel monitoring by fixing 3D bireflex targets for tunnel roof and walls deformation/convergence shall always be carried out in the tunnels for better understanding of rock mass-tunnel support interaction. The convergence targets shall be fixed immediately after face excavation at a regular interval of 50 m or as and when required on the basis of the ground condition/geology.

At one location, generally, five bireflex target points (or as per the decision of designer) are fixed to measure tunnel convergence (Fig. 7). X, Y & Z co-ordinates of each target point are recorded at regular basis. The readings are analysed on the day of taking readings to get the displacement of individual target point and the chord convergence between various target points. Accordingly, the results are used for countermeasures required, if any.

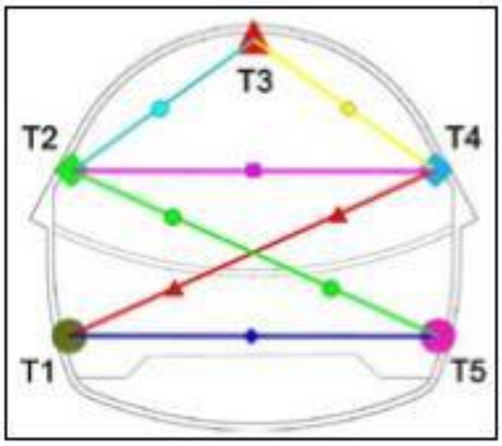

Fig. 7 - Array of five bireflex target points at one location to measure tunnel deformation/convergence

3.2.2 Deformation of rock mass around tunnel roof and walls

Deformation of rock mass around the underground opening is carried out using multi point borehole extensometer (MPBX) to know the affected zone of rock mass. This measurement is useful in knowing the zone of influence around the underground opening. MPBX also helps in monitoring the behavior of the weak rock or weak band/shear zone within the rock mass. The targets for deformation measurements shall also be installed at the location of MPBX.

3.3 Determining the Stability Condition

Primary purpose of instrumentation and monitoring is to determine and check the stability of tunnels. Simple plot of time vs displacement/deformation/pressure can provide this information. For example, curve 1 in Fig. 8 indicates a stable condition, whereas curves 2 & 3 in Fig. 8 indicate instability and requirements of additional supporting measures.

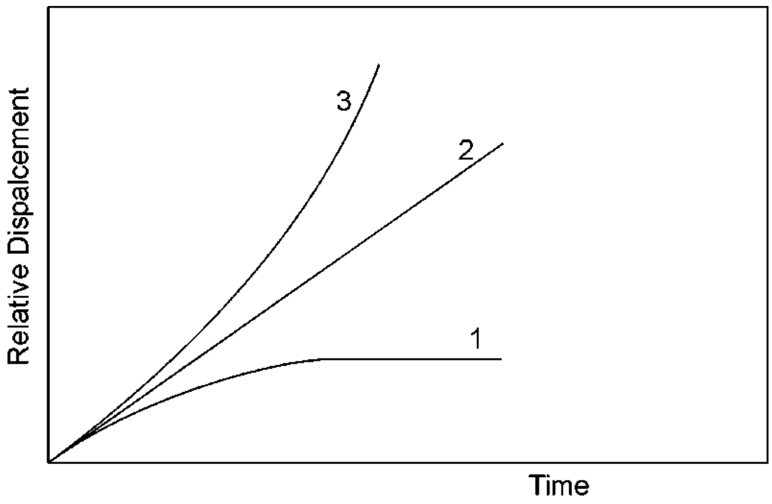

Fig. 8 – Instability as expressed by curves 2 and 3

3.4 Deformation Limits and Action Plan

Action on the basis of deformation measurements is very much important. If timely action on strengthening of primary supports is not taken, there can be instability problems leading to roof falls. Moreover, in areas where large deformations are observed, though in controlled manner, the roof and walls need to be trimmed to get the space for secondary lining. Thus, it is important to know the limits of allowable deformation and implement the action program accordingly.

A dual-level action plan for remedial measures is generally used as follows:

- *Attention level/limit* – It is a percentage of the predicted deformation. On exceeding this level/limit of deformation/convergence, the frequency of readings shall be increased in order to get the deformation speed. This trigger limit is set to study the deformation trends more closely and take countermeasures if the deformation remains continue with the same speed to the alarm level/limit. The attention level/limit, in general, is 70 percent of alarm level/limit.

- *Alarm level/limit* – It is the complete expected deformation from the design (coincides with the latest support section that will support the reached displacement and stress). Crossing of this limit will require initiating the procedure for actions and countermeasures.

Depending upon the site condition and experience the attention and alarm levels may be modified.

Precautions

- Limits of deformation to be allowed shall be known and followed.
- Some time, where high deformations are observed, the deformed roof and walls encroaches the space available for secondary lining. Trimming the primary support and applying the secondary lining is not only costly and time consuming but also disturb the stabilized rock mass around the opening. Monitoring shall again be carried out to study ground stabilization and once the ground is stabilized, generally the secondary lining shall be applied.

It should be kept in mind that tunnel closure may continue even for sufficiently longer duration (more than 26 months as observed in one tunnel in India) in the highly squeezing ground.

4. CASE HISTORIES HIGHLIGHTING THE IMPORTANCE OF MONITORING

4.1 Evaluation of Supports

In most of the cases the instruments are installed to evaluate the designs. Rock mass being heterogeneous and anisotropic, even the best design, sometimes found to be inadequate or over-design. In case of NATM, which is based on the philosophy of 'build as you go', instrumentation is the backbone for its success.

Figures 9, 10, 11 and 12 highlight the importance of instruments for evaluation of designs and optimization of supports. Timely information from the instruments has helped in strengthening the supports and stabilizing the tunnel.

In the main (Fig. 10) and escape (Fig. 11) tunnels of Chenani-Nashri highway tunnel in J&K state, India the convergence has crossed the alarm limit which called for the countermeasures in terms of longer rock bolts and additional shotcrete layer.

Figure 11 shows that even after countermeasures the deformation remains continued suggesting that the countermeasures are not adequate (Fig. 11). Therefore, again additional supports were applied to make the tunnel stable. This is a good example of NATM showing how the supports were strengthened with time. In this type of condition, MPBX data is important to know the zone of influence or the influence of weak rock or band (as shown in Fig. 9) and to decide the bolt length.

Figures 10 and 11 are from the same project, but showing the behavior of two tunnels of different sizes having different strata conditions. Figure 10 is for main tunnel having size more than the escape tunnel (Fig. 11). The bolt length required in escape tunnel excavated through weaker rock mass was more than the main tunnel excavated through comparatively better rock mass. This shows that in addition to the size of the tunnel, the bolt length varies with the rock mass condition.

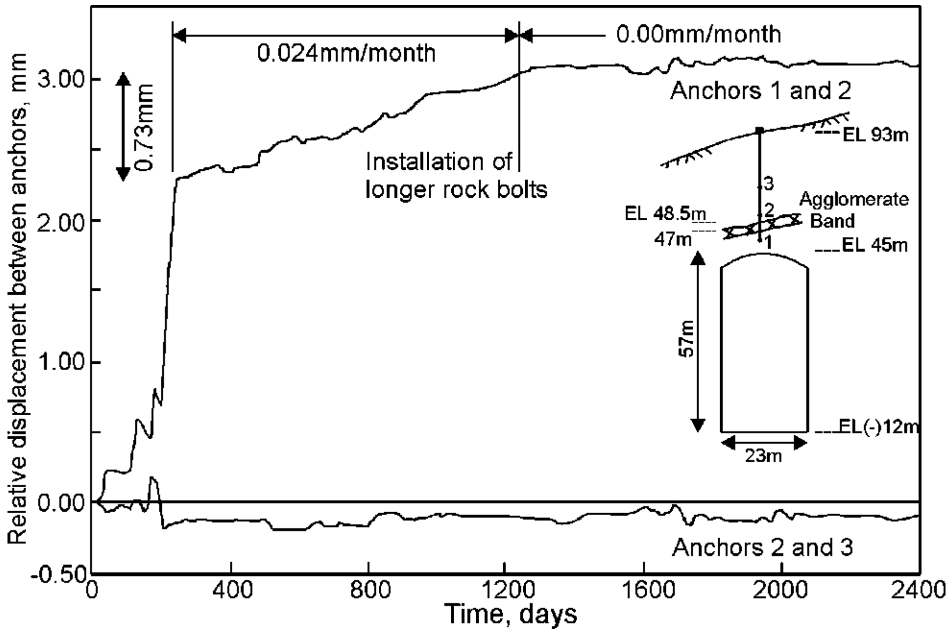

Fig. 9 – Monitoring agglomerate band contact with host rock mass basalt using multi-point borehole extensometer highlights the need for longer rock bolts in the powerhouse cavern of Sardar Sarovar project, Gujrat (Goel, 2001). The relative displacement between anchors 1 and 2 stabilized after installation of longer rock bolts.

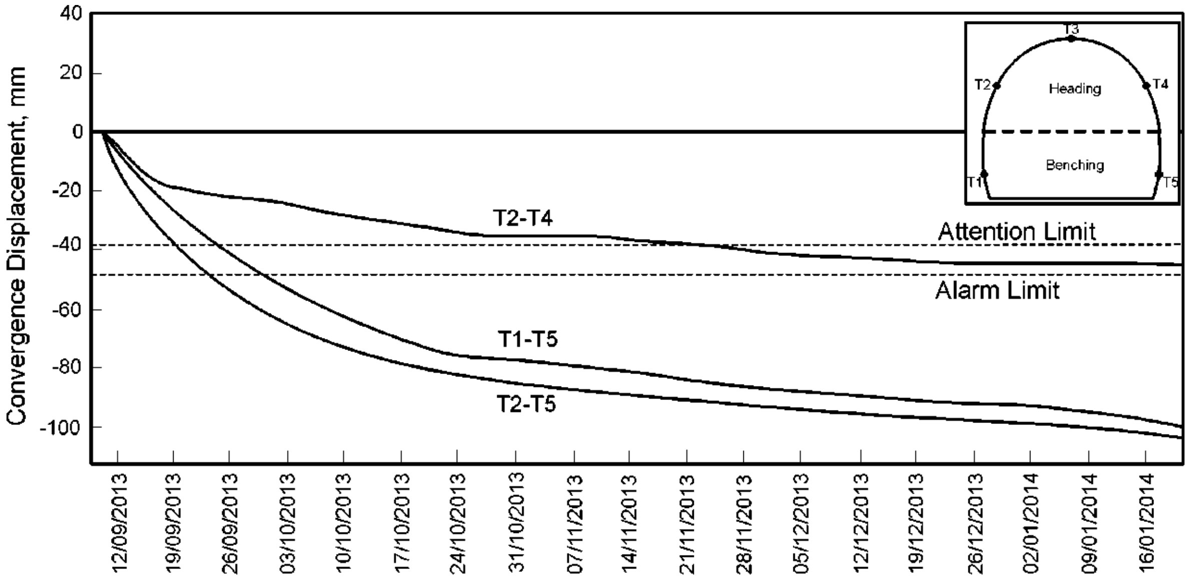

Fig. 10 – Additional rock bolt and shotcrete support installed after the convergence crossed the designed alarm limit in the main tunnel of Chenani-Nashri highway tunnel project constructed using NATM [T1, T2,…,T5 = Bireflex targets; T1-T5, T2-T4, T2-T5 = convergence between respective targets]

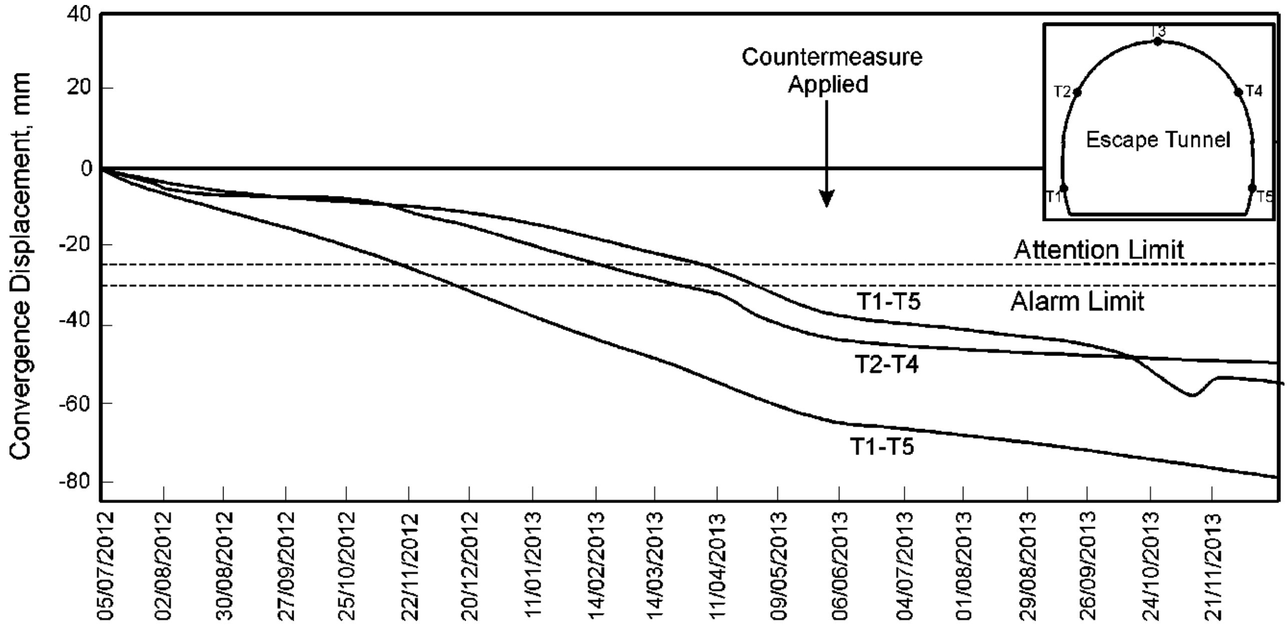

Fig. 11 – Additional rock bolt and shotcrete support could not effectively control the convergence showing that more countermeasures are required in the Chenani-Nashri highway tunnel project constructed using NATM

Figure 12 shows the importance of invert support (closing the support ring) in case of weaker rock masses where high side support pressure is expected and/or where high horizontal stresses are present.

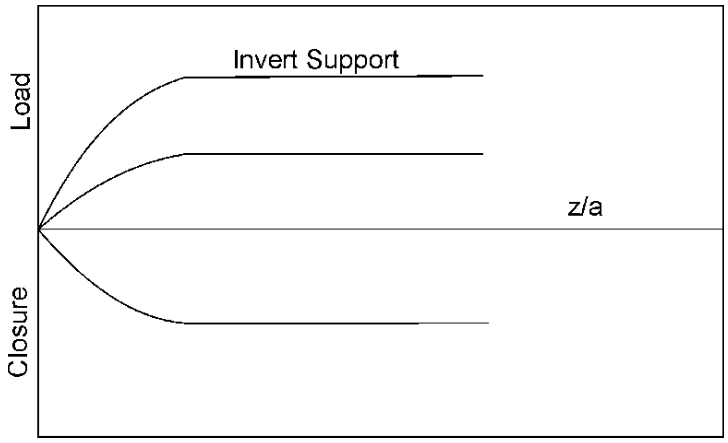

Fig. 12 – The beneficial effect of invert support has been shown schematically. Invert support is important in tunnels through weaker rock masses expecting high side pressure [z = face advance and a = tunnel radius]

4.2 Support Optimisation in Mixed and Layered Rock Masses

The rock masses along the Chenani-Nashri tunnel comprise of sequence of argillaceous and arenaceous rocks that includes a sequence of interbedded sandstone, siltstone/claystone beds with thickness ranging from a few metres up to 10m. The Q, RMR and N values of these rocks are given in Table 1.

Table 1 - Q, RMR and N values for different rocks of Chenani-Nashri tunnels (Goel et al., 2013)

S.No.	Rock(s)	Range of RMR	Range of Q	Range of N
1	Sandstone	50-64	3.5-8.0 (5.30)	8.75-20.0 (13.2)
2	Siltstone	49-54	2.0- 4.58 (3.02)	5.0-11.45 (7.5)
3	Claystone	22-26	0.08-0.14 (0.10)	0.4-0.7 (0.53)
4	Mixture of sandstone and siltstone	44-48	1.3-1.85 (1.55)	3.25-4.62 (3.87)
5	Mixture of siltstone and claystone	32-43	0.3-1.0 (0.54)	1.5-5.0 (2.74)

Note: RMR – Bieniawski's rock mass rating; Q – Barton's rock mass quality; N – rock mass number (Q with SRF=1) and values in () are the log average values

The deformation has been estimated using Eq. 1 along this tunnel at different chainage and given in Table 2.

Table 2 - Estimated roof convergence from Eq. 1 (Goel et al., 2013)

S.No.	Average Q	Average UCS, MPa	Tunnels	Roof convergence for different tunnel depths, mm					
				100m	200m	300m	400m	500m	600m
1	5.30	95	Main	3.9	5.6	6.9	7.9	8.9	9.7
			Escape	1.8	2.5	3.1	3.6	4.0	4.4
2	3.02	32.5	Main	11.9	16.9	20.7	23.9	26.7	29.2
			Escape	5.4	7.6	9.3	10.7	12.0	13.2
3	0.10	11.5	Main	606.1	857.2	1049.8	1212.2	1355.3	1484.7
			Escape	273.2	386.4	473.2	546.4	610.9	669.2
4	1.55	60	Main	17.1	24.2	29.6	34.2	38.3	41.9
			Escape	7.7	10.9	13.3	15.4	17.2	18.9
5	0.54	21	Main	83.06	117.5	143.9	166.1	185.7	203.4
			Escape	37.4	52.9	64.8	74.9	83.7	91.7

Note: Q = Barton's rock mass quality; UCS=uniaxial compressive strength

The bireflex targets for deformation monitoring were fixed at five location as shown in Fig. 7. Actual measured deformation values and rock mass exposed at the bireflex target position are given in Table 3.

Table 3 - Exposed rock type near the monitoring targets and radial deformation of various target points in main tunnel (based on Goel et al., 2013)

S.No.	Chainage (ch.), m	Q	Tunnel depth, m	Exposed Rock Types (Radial Deformation of Targets, mm)				
				T1	T2	T3	T4	T5
1	261	0.86	140	ST and CT (45)	ST (30)	ST and CT (28)	CT (35)	ST (15)
2	385	0.58	200	CT (52)	Sandy ST and CT (11)	CT (25)	CT and ST (125.8)	ST (23)
3	465	0.58	210	ST (53)	ST and	CT (45)	Ct	Clayey

					CT (11)		(61.3)	ST (41)
4	527	0.83	230	ST (28)	ST and CT (70)	ST and SST (10)	ST (30)	ST (46)
5	621	1.03	265	ST (17)	ST and SST 19.1	ST (52.4)	ST (166)	Sandy ST (142)
6	837	1.37	345	ST (51)	ST (79)	ST (38)	ST (150)	ST (3)
7	948	2.72	385	CT and ST (20)	ST (59)	Sandy ST (61)	Sandy ST (240)	CT and ST (62)

Note: value in ( ) is the radial deformation of supported rock in mm; Q - Barton's rock mass quality; ST - siltstone, SST - sandstone; CT - claystone

The deformation of target varies with the rocks exposed near the target. Because of the effect of varying strength of different rocks, deformation obtained using Q is more than the observed values for clay stone and less for sandstone. Also, more deformation is recorded near weaker rocks like claystone and less near good rock like sandstone. Accordingly, the supports were used and strengthened wherever required as sown in Figs. 10 and 11.

4.3 Flexible Supports in Squeezing Ground Condition

Designing of supports in squeezing ground condition is a challenging task. In order to reduce the ultimate pressure coming on supports, the supports in this case shall be optimally flexible to allow the controlled deformation. Jethwa (1981), first time suggested the concept of loose backfill, which helped in reducing the support pressure in a controlled manner (Fig. 13).

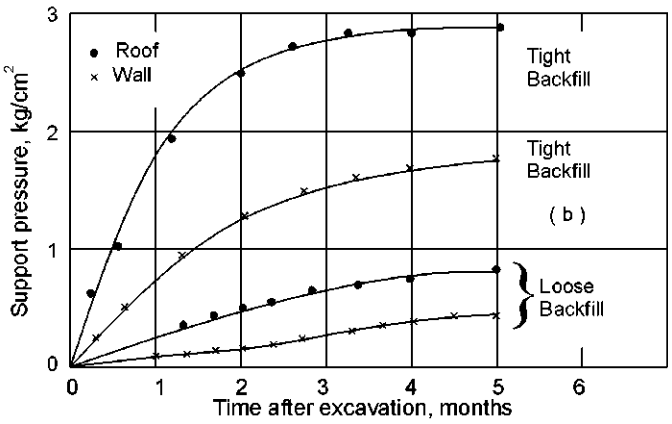

Fig. 13 – Influence of loose backfill flexible support in reducing support pressure (Jethwa, 1981)

Similarly, in a railway tunnel of Udhampur-Katra section the same concept of loose backfill (using tunnel muck as the backfill material between excavated rock periphery and the outer flange of steel rib) was used successfully.

Taking the clue from this flexible support concept, various types of flexible supports have been tried and developed for tackling the support design issues in tunnels facing squeezing ground conditions.

4.4 Effect of Stoppage of Work on Deformation

The work of Chu et al. (2020) is presented here to analyze the time-dependent behavior of tunnels as the longitudinal excavation stops at the halfway point. This case pertains to a deep horse-shoe shaped tunnel through soft rheological rock mass showing the time-dependent deformation. The deformation monitoring was carried at five points, Fig. 14.

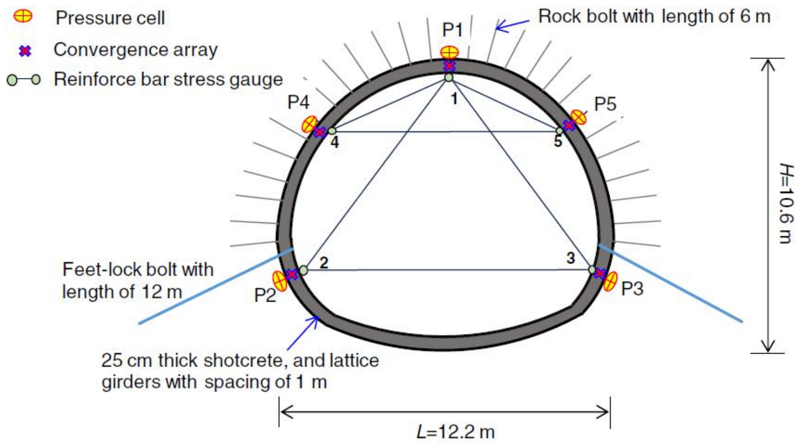

Fig. 14 – Showing various instruments and supports installed at one location (Chu et al., 2020)

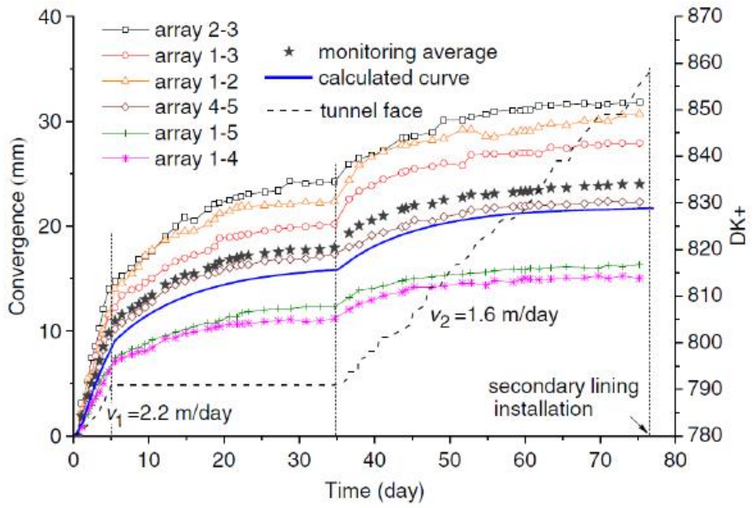

Fig. 15 – Time convergence plot showing the effect of stoppage of work on convergence (Chu et al., 2020)

Figure 15 shows the variation of convergence with the face advancement. The face advancement work remained stopped between day 5 and 34. The rate of advancement before 5 days was 2.2m/day and after 34 days 1.6m/day. From the Fig. 15, it can be seen that the convergences still increase with time when the tunnel face stops, while its increasing trend is evidently smaller than that as the face is advancing. Apparently, the tunnel convergence rates depend on the advancement rate and the rock's rheological behaviors. Also, whether the tunnel face is in continuous or discontinuous advancement, for an unlined tunnel, the ultimate convergence is identical to that without the tunnel face effect. That is, the advancement process of the tunnel face only has an effect on the convergent rate, not on the ultimate convergence.

5. CONCLUDING REMARKS

- The instrumentation is important to tackle the heterogeneous characteristics of the rock mass.
- The expected rock mass behaviour and tunnel deformation shall be estimated in advance.
- Accordingly, the allowable limits of controlled deformation shall be decided.
- As per the allowable deformation, the tunnel excavation size shall be decided.
- In case the tunnel deformation is more (or vice a versa) than the estimated/ expected deformation, primary supports shall be timely strengthened or reduced.
- Data shall be properly preserved to help in the development of the state-of-the-art.
- The work shall be carried out in the supervision of an expert.

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