Design of Tunnel Support and Blasting for Railway Tunnels in Basaltic Rocks – A Case Study

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ABSTRACT

Four railway tunnels were proposed in the Kurduwadi-Latur section of Central Railway, Solapur division, India. The rock mass in the area was predominantly basalts composed of mainly compact basalt with amygdaloidal basalt at a few locations. Investigations were conducted to formulate support and blast design guidelines for these proposed tunnels using empirical methods. The investigation included rock mass characterization along the proposed tunnels using Barton's Q and Bieniawski's RMR approaches and the estimation of tunnel stability parameters. The rock mass was classified as good in most of the tunnel length and as poor at a few locations. The ground condition was estimated as non-squeezing type. The recommended support system included a combination of plain and steel fibre reinforced shotcrete (SFRS) along with systematic rock bolting in the tunnel length and the steel supports and back fill concrete in the portal regions. The blast design included heading and benching method with detail on blast patterns, estimated charge factors, etc.

Keywords: Railway tunnel, rock mass characterisation, support design, blast design.

1. INTRODUCTION

It was proposed to construct four tunnels of varying lengths along the Kurduwadi-Latur section to connect Osmanabad city under Central Railway, Solapur, India. As these tunnels were to serve as railway tunnels, it was therefore required to conduct appropriate feasibility studies and to devise tunnel support and blasting guidelines.

The authors conducted required investigation and formulated the recommendations for tunnel support and blasting. This paper details the investigation carried out for this study along with the recommendations.

2. PHYSICAL DETAILS AND LOCATION OF PROPOSED TUNNELS

The tunnels were proposed in the area known as Osmanabad diversion in the Kurduwadi - Latur (Pangri - Yedshi) section. The nearest railway station being Solapur at a distance of around 80 km. A total of 4 tunnels with varying lengths were proposed (Fig.1). Tunnel wise length, location and axis are shown in Table 1. The cross-sections of all the tunnels were the same with width from 6.2 to 6.5 m and height at 7.85 m (Fig. 2).

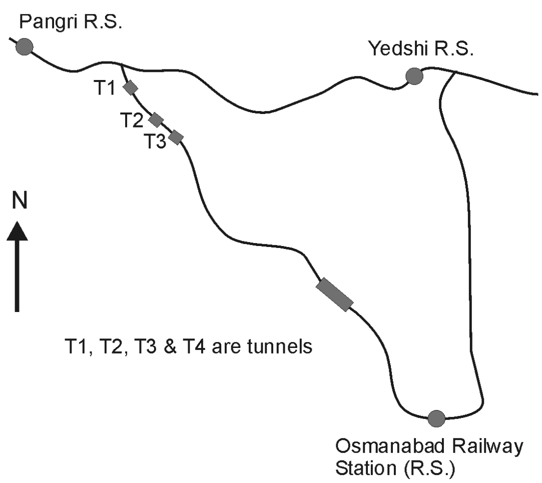

Fig.1 - Layout plan showing the locations of tunnels (Not to scale)

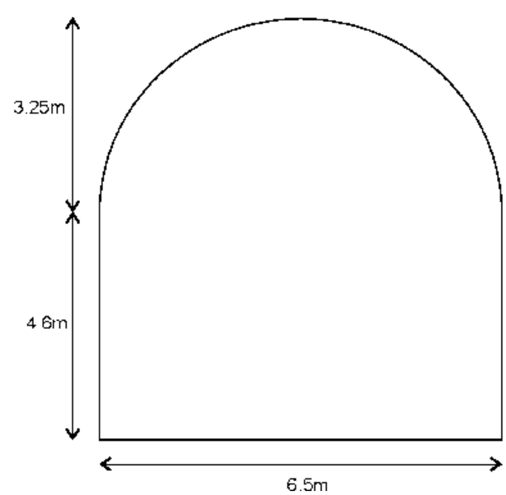

Fig. 2 - A typical excavated cross-section of tunnel (Not to scale)

The longitudinal sections along these tunnels showing the ground profile, tunnel roof and floor profiles are shown in Figs. 3 through 6.

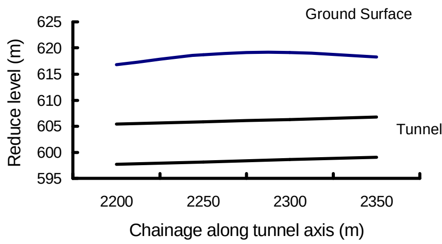

Fig. 3 - Longitudinal section along Tunnel 1

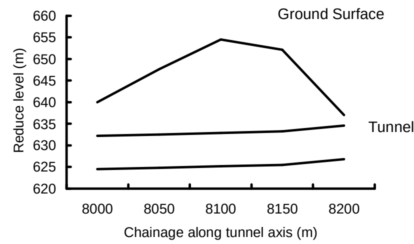

Fig. 4 - Longitudinal section along Tunnel 2

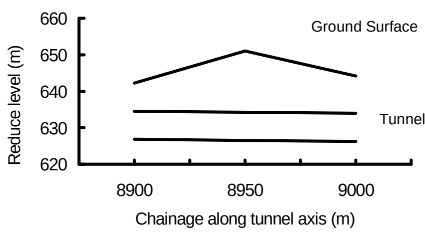

Fig. 5 - Longitudinal section along Tunnel 3

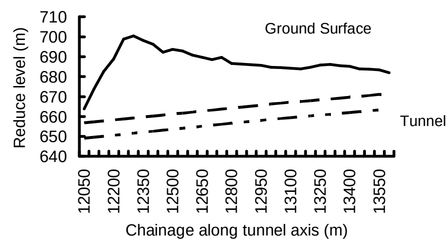

Fig. 6 - Longitudinal section along Tunnel 4

Table 1 - Salient details of different tunnels

Tunnel no.	Length, m	Location	Axis
1	88	Ch. 2231 - Ch. 2319	123^{0} 10' 00" - 87^{0} 30' 00"
2	132	Ch. 8043 - Ch. 8175	64^{0} 00' 00" - 81^{0} 07' 20"
3	61	Ch. 8929 - Ch. 8990	81^{0} 07' 20" - 122^{0} 51' 40"
4	1536	Ch. 12064 - Ch. 13600	107^{0} 45' 40" - 141^{0} 01' 40"

3. GEOLOGY OF THE AREA

The area is dominated by basaltic flows and they tentatively belong to the Deccan Traps of upper cretaceous to lower eocene age. The flows are dark grey to ash grey in colour, hard, compact, mostly non-porphyritic to very sparsely porphyritic. The round to elongated shape gas cavities called vesicles in the rock are also found occasionally filled by secondary minerals such as zeolites, calcite, silica and glass. The basalts with such gas cavities is generally called amygdaloidal basalt. The two successive flows are separated by red boles.

The petrographic study of flows reveals that the basalts are non-porphyritic, very fine grained, with microlites of plagioclase feldspar (labradorite to bytownite composition), clinopyroxene, opaque iron ores and dark brown glass. The flows display a regional gradient of 1 in 550 and 1 in 300 in SSE to SE directions respectively.

The area is devoid of any intrusive and structural deformations. The rock mass in this area is thus predominantly basaltic, a combination of compact basalt and amygdaloidal basalt.

4. GEOTECHNICAL DETAILS

4.1 Joints or Discontinuities

The rock mass is moderately jointed with essentially three joint sets. Two joints are vertical to sub-vertical and the third joint is sub-horizontal. The strike of the vertical joints are N50° and N147°, whereas the strike, dip and dip direction of third joint is N325°, 5° and N235° respectively. Vertical joints are critical from the tunnel stability point of view. The joints are not at a regular spacing. The spacing between joints of one set varies from 20 to 60cm. The joints are undulating and the joint surface is rough or irregular. Generally, the vertical joints are tight but at places joint walls are slightly altered with coating of sandy particles.

4.2 Rock Quality Designation (RQD)

The RQD as determined from bore hole cores varies from 85 to 95 in the cases of compact basalts and from 15 to 50 in the case of amygdaloidal basalts.

4.3 Physico - mechanical Properties

Uniaxial crushing strength (UCS), density and the P-wave velocity of both the rock types were determined in the laboratory. In addition, uniaxial compressive strength of rock masses was also assessed from Schmidt hammer rebound number in the field. The results are presented in Table 2.

Table 2 - Physico-mechanical properties of rocks

Name of the Rock/ Tunnel No.	UCS (MPa)	Specific Gravity (gm/cc)	P-wave Velocity (m/s)
Amygdaloidal Basalt	22.94 - 46.39	2.76	7471
Compact Basalt	62.27 - 120.15	3.04	3879

5. ROCK MASS CLASSIFICATION

The rock mass of the area was classified on the basis of Rock Mass Quality (Q) as proposed by Barton et al. (1974) and Rock Mass Rating (RMR) proposed by Bieniawski (1976). Rock Mass Number (N), defined as stress free Q, i.e. Q with SRF=1, and proposed by Goel et al. (1995) has also been considered for estimating the ground condition and other tunnel design parameters.

5.1 Rock Mass Quality (Q) and Rock Mass Number (N)

Q and N values for basalts and amygdaloidal basalts are given in Table 3. Both the values were used to estimate support pressures.

Table 3 - Rock Mass Quality Q and Rock Mass Number N

Rock Type	RQD	J_n	J_r	J_a	J_w	N	SRF	Q	Q_w
Basalts	60-90%	9.0	3.0	2.0	1.0	10-15	1.0	10 - 15	62.5
Amygdaloidal Basalt	25 %	12	3.0	2.0	1.0	3	1.0	3	7.5

5.2 Rock Mass Rating (Bieniawski, 1976)

RMR was determined from the rock exposures as shown in Table 4.

Table 4 - Ratings of various parameters to obtain RMR_{basic}

Rock Type	RQD	Joint Spacing	Joint Condition	UCS	Water	RMR_{basic}
Basalt	17	10	25	10	15	77
Amygdaloidal Basalt	8	10	25	4	15	62

In view of the tunnel axis being different for different tunnels, the RMR_{basic} has been adjusted for joint orientations to obtain final RMR (Table 5). Out of the two almost vertical joints, one joint set was assumed to be critical with the orientation of tunnel axis. As such final RMR for all the four tunnels is given in Table 5.

Table 5 - Tunnel wise final RMR

Tunnel	Rating for Joint Orientation	RMR
Tunnel 1 (Axis:123^0 10' to 87^{0}30')	-5	72
Tunnel 2 (Axis:64^0 00' to 81^0 7')	-12	65
Tunnel 3 (Axis:81^0 07' 20" to 122^0 51' 40")	-5	72
Tunnel 4 (Axis:107^0 45' 40" to 141^0 01' 40")	-12	65

In the case of amygdaloidal basalts, the final adjusted RMR was worked out at 50. The rock mass was thus classified as poor to good. Amygdaloidal basalt was rated as poor. The study of bore hole core and rock exposure revealed that most of the tunnel lengths would encounter only compact basalt whereas a only section of tunnel 4 would encounter amygdaloidal basalt.

6. ASSESSMENT OF TUNNEL STABILITY

6.1 Estimation of Roof Deformations

Empirical correlation suggested by Goel (1994) was used for estimating the roof and wall deformations.

$$\frac{u_a}{a} = \frac{H^{0.6}}{28 \; N^{0.4} \; K^{0.35}} \; \% \qquad (1)$$

where

u_a/a = Normalised roof tunnel closure in per cent,

u_a = Radial roof tunnel closure,

a = Half of tunnel width or height for roof and wall deformations respectively,

H = Tunnel depth in meters,

N = Rock mass number (i.e. Barton's Q with SRF = 1), and

K = Effective support stiffness in MPa (assumed as 1).

The maximum radial deformation/closure value obtained for tunnel depth 50m and rock mass number 3.0 is 0.24 per cent which is far less than one per cent and thus indicates non-squeezing conditions.

6.2 Estimation of Roof Support Pressure

Empirical approaches of Barton et al. (1974) and Goel et al. (1995) have been used to estimate the support pressures (p) for various rock types using Q and N respectively.

(A) Using Rock Mass Quality Q [Barton et al., 1974]

$$p = \frac{0.2}{J_r}(Q)^{-(1/3)} \qquad \text{MPa} \qquad (2)$$

where

p = Support pressure in MPa,

Q = Barton's Q value, and

Jr = Barton's joint roughness number.

(B) Using Rock Mass Number N [Goel et al., 1995]

$$p = \frac{0.12 \, H^{0.1} \, a^{0.1}}{N^{0.33}} - 0.038, \qquad \text{MPa} \qquad (3)$$

where

p = Support pressure in an underground opening in MPa,

a = Half of tunnel width or height for roof and wall support pressure respectively in metres (= 3.25m),

B = Tunnel width in metres,
H = Tunnel depth in metres, and
N = Rock mass number, i.e. Barton's Q with SRF = 1.

Table 6 - Estimated values of support pressures

S.No.	Rock Type, Q and N	Support Pressure in MPa Using	
		Eq. 2	Eq. 3
1.	*Basalts* Q=10 to 15; Q(Avg.) = 12.5 & N(Avg.) = 12.5	0.03	0.048 (for 50m depth)
2.	Q_{Portal} = N = 6.25	0.036	0.065 (for 20m depth)
3.	*Amygdaloidal Basalts* Q = N = 3.0	0.053*	0.09

6.3 Estimation of Wall Support Pressure

Wall support pressure has been estimated using wall Q (Q_w) as given below:

- Basalts p_w = 0.017MPa
- Amygdaloidal basalts p_w = 0.034MPa

The support pressure thus varies between 0.017 and 0.09 MPa for rock mass from good basalts to weak amygdaloidal basalts.

7. Design of Supports

The maximum support pressure in portal area was estimated as 0.546 MPa [H = 19.5m (3 times the tunnel width)]. Thus, the portal supports have been designed for support pressure value of 0.546MPa.

Supports were designed separately for (i) the portal and portal region where the cover is less than 3 times the tunnel width, (ii) the regions of poor rock mass and (iii) other locations as given in Table 7.

Support designs given in Table 7 were made using empirical approach of Grimstad and Barton (1993) in all cases except for portal supports.

To keep the finished diameter or span of the tunnel same all along the tunnel length, the excavation size shall be increased in portal areas for accommodating the steel ribs.

Table 7 – Tunnel support design

Sl. No.	Location	Support	Remarks
1.	Portal region	Steel ribs (ISMB 200), Backfill concrete (M-20/25) and 25 mm plain shotcrete to cover the exposed steel ribs	• Diameter of anchor /bolt = 22mm • Bolt length = 3m • Size of base plate = 250 x 250 x 12.5 (mm)
2.	6m to 20 m from tunnel mouth	Rock bolts (1.5 m center to center) & 50 mm plain shotcrete	
3.	Poor rock mass (wherever encountered)	Rock bolts (1.3 m center to center) & 50 mm SFRS	
4.	Other locations	50 mm plain shotcrete along with spot bolting as and when required	

8. DESIGN OF BLASTING

The blast designs have been evaluated on the basis of empirical relationships (Chakraborty, 2002).

8.1 General Details

The following points were suggested in respect of blast designs.

- Heading and benching method for tunnel excavation (heading about 3.5m high).
- Wedge cut blast pattern for heading and vertical drilling for benching.
- Jack hammer [hole diameter (d_h) = 33 mm] with 3.5 m deep holes for drilling. In the portal region, 1.5 m deep drilling is suggested to limit the charge per delay so as to control over break.
- Emulsion explosive of 25 mm diameter in conjunction with ANFO (in cartridge form) in blasting.
- Contour blasting in heading.

8.2 Blast Design for Heading

The various blast design and performance parameters were evaluated for blasting (Table 8) in the heading section using different correlations (Eqs. 4 through 8).

$$\mathrm{TBI} = \frac{c_p + n + (\mathrm{RQD}/10)}{(A - r)\,(C_\theta + R_c)} \tag{4}$$

Specific charge and specific drilling drilling before blasting is obtained by Eqs. 5 & 6 respectively as follows.

$$\text{Specific charge (kg/m}^3\text{)} = 1.1 + 0.24\ \text{TBI} \quad (5)$$

$$\text{Specific drilling (m/m}^3\text{)} = \frac{4.79(\text{TBI})^{0.6}}{md_c^{0.5}} + s_h \quad (6)$$

$$\text{Pull (\%)} = \frac{[2.38 - 0.7\ln(\text{TBI})(\sin\theta)^2 + J_{OAp}]}{\text{Cut depth in metres}}.100 \quad (7)$$

$$\text{Roof overbreak (m)} = \frac{0.57 - 0.52\ln(\text{TBI}_c) - 0.5s_h - J_{OAr}}{md_c} \quad (8)$$

where

TBI = Tunnel Blasting Index,
c_p = P-wave velocity (km/s),
n = No. of mixed faces,
A = Tunnel (heading) size, m^2,
r = Tunnel direction in respect to vertical (expressed in radian),
C_θ = Wedge angle (65°, assumed based on usual practice) at the apex expressed in cotangent value,
R_c = 0.76 coupling ratio (ratio of explosive to hole diameter), and
= 0.5 in case of contour blasting because of axial separation by spacers.
TBI_c = Tunnel Blasting Index of the roof rock mass
md_c = Spacing to burden ratio of periphery holes,
= 1.25 in case of compact basalt and 1 in case of amygdaloidal basalt,
s_h = Shape factor (ratio of width to diameter) = 0.8125,
J_{OAr} = Joint orientation rating for roof overbreak = 0.3 to 0.6, and
J_{OAp} = Joint orientation rating for pull = -0.3 to -0.6.

Table 8 - Blast design and performance parameters

Formations	TBI	Specific charge (kg/m^3)	Specific drilling (m/m^3) including relief holes	Pull (%)	Roof overbreak (m)
Compact basalt	0.54	1.23	3.77	73	Negligible
Amygdaloidal basalt	0.3	1.17	3.11	86	0.2

To control wall over break, contour blasting was suggested in the perimeter holes. Long delay detonation was suggested in the heading portion for providing sufficient time for the burden to move in confined condition. The layout of holes in the heading section was suggested as shown in Fig. 7.

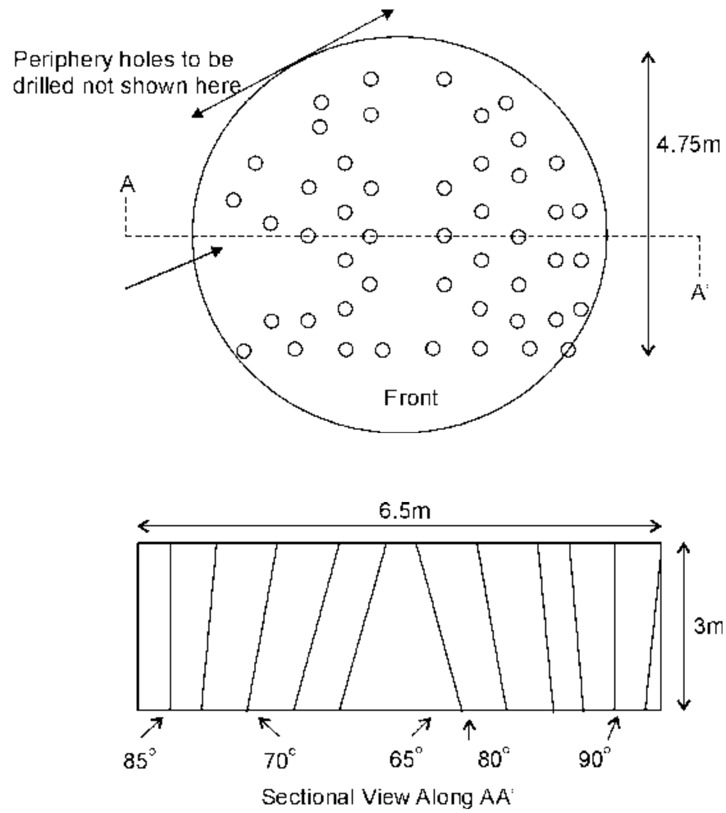

Fig. 7 - Layout of holes in the heading section

8.3 Blast Design for Benching

In benching, hole depth was proposed at 3.5 m for a final pull of 3.0m. Different blast designs were proposed for amygdaloidal basalt and for compact basalt (Table 9).

The mean fragment size (MFS) was estimated using Eq. 9.

Table 9 - Blast design parameters in benching section

Type of rock	Burden (Bd), m	Spacing (Sd), m	Specific charge (q), kg/m3	Charge per hole, (kg)	Subgrade drilling (m)	Stemming length (ls), (m)
Amygda-loidal basalt	(25 d_h) 0.8	(1.25xB_d)1	0.45	1.1	0.3-0.5	0.8 -1
Compact basalt	0.75	0.90	0.6	1.44	0.3-0.5	0.8 -1

$$\text{MFS} = 0.07\,(I_s)^{0.54}\,\left(\frac{A}{q}\right)^{0.172} \tag{9}$$

where

I_s = Stemming length in metres,
Q = Specific charge in kg/m^3,
A = Rock mass factor,
= 3 for RQD 50, and
= 30 for RQD greater than 80.

The MFS was estimated to vary between 0.10 and 0.13 m.

9. CONCLUSIONS

- The rock mass is moderately jointed and mainly consists of compact basalt. Amygdaloidal basalt with gas cavities is however found at a few locations along one of the tunnels.
- The rock mass could be classified as poor to good. The most of the rock mass to be encountered is categorized as good. The poor category belongs to amygdaloidal basalt. The ground condition was estimated as non-squeezing type.
- The Support designs were formulated on the basis of established empirical approaches for tunnel portals and along the tunnel lengths in both poor and good rock mass conditions. It was suggested to provide steel rib supports in the portal region. The support in the rest of the locations included a combination of systematic bolting and plain shotcrete/SFRS and plain shotcrete and spot bolting.
- Tunnel excavation to be carried out in two steps, i.e. by heading and benching method. The specific charges for heading and benching operations are worked out at 1.1-1.23 kg/m^3 and 0.3 0.6 kg/m^3 respectively. The pull was estimated as satisfactory and to vary between 73 and 86 per cent.
- To control overbreak, contour blasting was proposed in the periphery holes in the heading section.
- In view of the rock mass quality, ground condition and jointing patterns, the tunnel excavation was not expected to encounter any serious stability problems with the implementation of suggested support and blasting patterns.

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