



Study of Roof Collapse in Rohtang Tunnel during Construction

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ABSTRACT

Collapse in tunnels during construction in Himalayan region may occur due to various reasons. High overburden over tunnel causes increase in rock load. High overburden in weak rock mass creates squeezing ground condition while that in hard/brittle rock may cause rock burst problem. Stress flow along foliation under high overburden and deep seated slope movement may also give rise to high deformation in a particular direction in tunnel. In North portal drive of Rohtang tunnel, alternate layers of migmatite and mica schist are being encountered. Mountain is very steep and overburden rises upto 1.9 km. High deformations are being observed in crown area of the tunnel. Problems being faced in North portal drive include bending of lattice girders, cracks in shotcrete layer, fall of shotcrete/rock block etc. In October 2013, a stretch of crown area of approximately 50 m in length suddenly collapsed. Overburden in this area was approximately 1200 m. It took almost two months to repair the collapsed area. To counter the problems and prevent such occurrence in future; a thick layer of shotcrete, longer rock bolts, regular repairing of cracked shotcrete layers and daily 3D deformation monitoring are being followed.

Keywords: Collapse; Squeezing; Overburden; Stress; Foliation; 3D monitoring

1. INTRODUCTION

Rohtang road highway tunnel is being constructed under Pir panjal mountain range of Himalaya near Rohtang pass in Himachal Pradesh, India. It will connect Manali to Lahaul valley. It will reduce the road length of Manali-Sarchu-Leh road axis by 46 km as well as establish throughout the year connectivity of Manali to Lahaul valley. South Portal of Rohtang tunnel is located at a distance of 25 km from Manali at altitude 3060m. North portal of Rohtang tunnel is located near village Teling, Sissu in Lahaul valley at an altitude of 3071m. Total length of Rohtang tunnel is 8.802 km. It is horse-shoe shape, single tube of two lanes alongwith raised footpaths on both sides of the tunnel. It has semi transverse ventilation system at top and egress tunnel at the bottom of the carriageway for maintenance and emergency exit. The tunnel has been designed for maximum speed of 80 km/h. Drill and blast method of tunnelling with NATM philosophy is being used for the construction of Rohtang tunnel.

2. GEOLOGY OF ROHTANG TUNNEL

Rohtang tunnel project is located within 'Central crystalline group' litho-tectonic group of Himalaya. The regional geological succession at the project site comprises of the Tandi formation, Batal formation, Salkhala group and the Rohtang gneiss complex. The Rohtang tunnel alignment is mainly passing through Salkhala group rocks of Precambrian age. Main

rock types along the alignment are phyllites, quartzites, mica schist, migmatite and gneiss. Major geological structures in the area are Seri nala fault, Chandra-Kothi structure, Rohtang ridge structure, Dhundi structure, Palchan structure, Palchan fault, Sundar nagar fault and main central thrust. RITES (1994) and SMEC (2007) give the details of the geology and design of supports.

2.1 Geology of North Portal

Longitudinal section of tunnel is shown in Fig. 1. Chainage of North portal is 9085.0m RL with tunnel control point at North portal is 3071.0m. Rock types being encountered in North portal drive are - migmatite, mica schist, gneiss and phyllite with quartzite intrusions. Mainly migmatite and mica schist are being encountered in alternate layers with dip direction and dip angle being 230-240° and 20-25° respectively. The thickness of migmatite band as well as that of biotite schist band varies from 0.25m to 10 m. Alignment of the tunnel from North portal is N16°E. There is a kink in tunnel alignment near North portal. Figure 2 shows the cross-section of the tunnel. Figure 3 is the geological face log prepared at chainage 7239.0m. Two main shear zones due to 'Chandra Kothi structure' (ch.7600m - 8350m) and 'Rohtang Ridge Structure' (ch.6000m - 6800m) were expected in the tunnel alignment from North portal drive. Tunnelling in shear zone due to Chandra Kothi structure has already been completed without much difficulty. Although two perpendicular joint sets other than foliation joint with persistency more than tunnel heights were causing wedge formation in crown. Dip direction/dip angle of the joint sets were measured (215-240°)/(60-85°) and (130-140°)/(35-70°). But controlled blasting with less charge was used to mitigate the problem. Mountain is very steep from North portal side. High overburden is creating problems for tunnel construction in North portal drive. By August 2014, more than 2.25km of tunnel has been excavated from North portal side and overburden has reached upto 1500m. Problems created by high overburden includes high deformation specially in crown area, bending of lattice girders, cracks in shotcrete, fall of shotcrete/rock blocks etc. Due to low adhesion between shotcrete and mica schist, falling of shotcrete layer is common where mica schist is present.

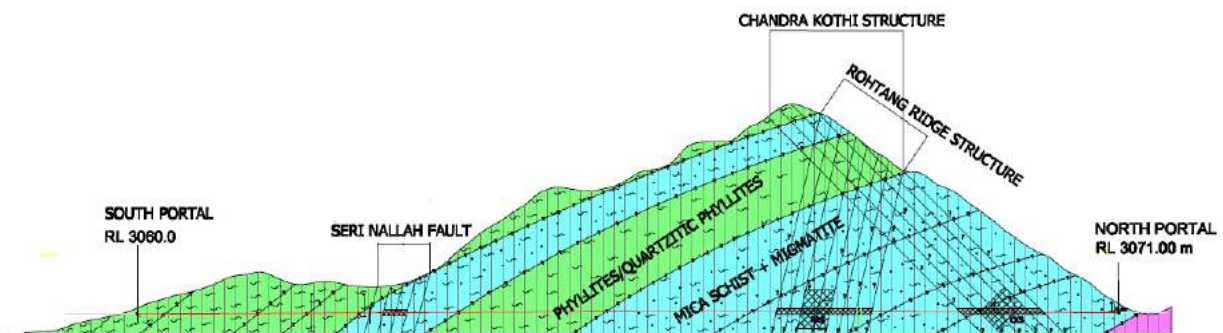


Fig.1 - Geology along the tunnel alignment

3. CONSTRUCTION OF ROHTANG TUNNEL FROM NORTH PORTAL

3.1 Vehicle/Equipments/Plants

Main vehicles/equipments/plants being utilized in North portal side are – two arm boomer, one arm boomer, shotcreting machine (Meyco Potenza, make - BASF), excavator, crusher plant, loader, batching plant etc.

3.2 Progress of Tunnel

By Aug'2014 total tunnel length of 2.25 km has been excavated from North portal side. Tunnel excavation in North portal is in progress from the year 2010 when first blast was taken on 4th Oct

2010. In the year 2013, roof area collapsed on 16th October in between ch.7195m and 7245m. Before the incidence of roof collapse 1.95km of tunnel had already been excavated. Repair of collapsed area was completed upto December 2013. Tunnel excavation from North portal side is carried out from the month of June to December every year. In rest of the period tunnel construction cannot be carried out due to high accumulation of snow on connecting road to North portal from Manali. Rock mass classes being encountered from North portal side are 1, 2, 3, 4 and 5. Rock mass classification is based on rock mass number (N) of Goel (1994). Due consideration has been to account of high overburden. A matrix between rock mass number and overburden has been provided for two main types of rocks being encountered.

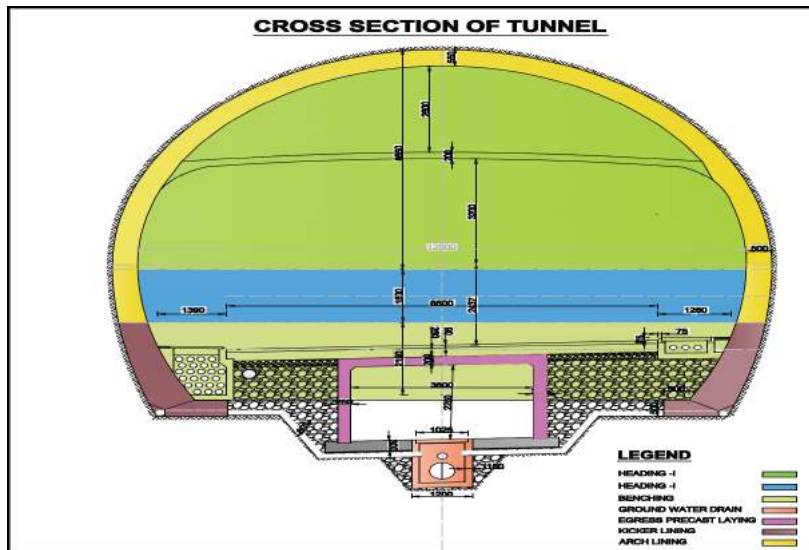


Fig. 2 - Cross-section of tunnel

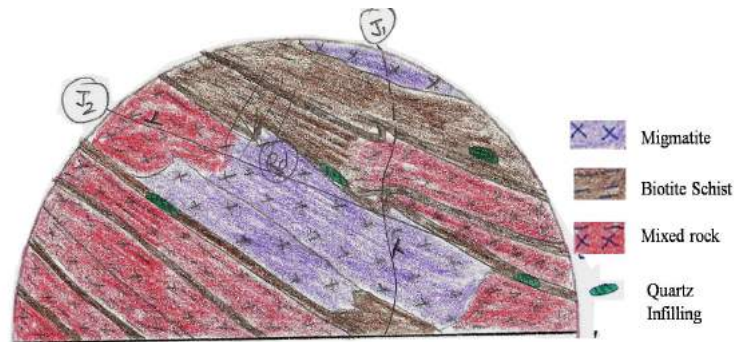


Fig. 3 - Geological face log at ch. 7239.0m

Progress of tunnel from North portal drive is more or less as per method statement, which is as follows:

Rock class 1 – 8.3m/day; rock class 2 - 7.5 m/day; rock class 3 - 6.0 m/day; rock class 3M - 6.1 m/day, rock class 4M – 5.2 m/day, rock class 4S – 3.7 m/day, rock class 5- 3.8 m/day, rock class 6 – 3.0 m/day and rock class7- 2.1 m/d

Rock classes 3M and 4M denote rock class 3 and 4 respectively with moderate chances of rock burst. Rock class 4S denotes rock class 4 with severe chance of rock burst. Moderate and severe chances of rock burst have been estimated considering type of rock mass and overburden. Rock classes have been classified based on range of rock mass number (N) as shown in following Table 1.

Table 1 - Rock mass number N and the rock class in the Rohtang tunnel

Value of N	Rock Class	Value of N	Rock Class
>10	1	0.1-0.4	5
4-10	2	0.01-0.1	6
1-4	3	0.01-0.1	7*
0.4-1	4		

*-Overburden material

4. ROOF COLLAPSE

Incidence of roof collapse on 16th October 2013 occurred in between Ch. 7195m and 7245m. Fortunately there was no loss of life. A stretch of crown of approximately 50 m in length collapsed suddenly. Overburden in this area was approximately 1200m. Heading was 100m ahead of the collapsed area. It took almost two months to repair the collapsed area. Figure 4 is photograph of the tunnel section after collapse as seen from North portal. Figure 5 is the photograph showing the repairing works after clearing the debris from the collapsed area. Figure 6 shows the cross-section of the tunnel at ch. 7235.7m after collapse and as seen from South portal. It may be noted that the debris due to roof collapse was more than 1000 m³.

The apparent dip of joint (θ_1) and the approximate angle of the wedges in the crown from tunnel vertical axis (θ_2) were measured (Fig. 7). It is found that θ_1 is approximately equal to θ_2 .

5. REMEDIAL MEASURES TAKEN FOR REPAIR WORK

After clearing the debris due to collapse, repairing works was started. Repairing works included - reinstallation of lattice girders, reinstallation and extension of rock bolts (SDA type, IBO, Atlas Copco) and shotcreting with wiremesh (Fig. 5). It took almost two months to complete the repairing job. At other locations of the tunnel also, where tunnel deformations were observed to be higher/unsafe, extra rockbolts (swellex and IBO) were installed.

At some of the locations, shotcrete layer was detached from the rock surface. Loose shotcrete was removed and fresh layer of shotcrete was applied on those areas. After completion of repairing works, North portal site was closed for the season anticipating snowfall. North portal was then opened for construction in next season only i.e. in May 2014.



Fig.4 - Roof collapse at Ch. 7+210.89m, North portal



Fig.5 - Installation of extra IBO rock bolts in collapsed area

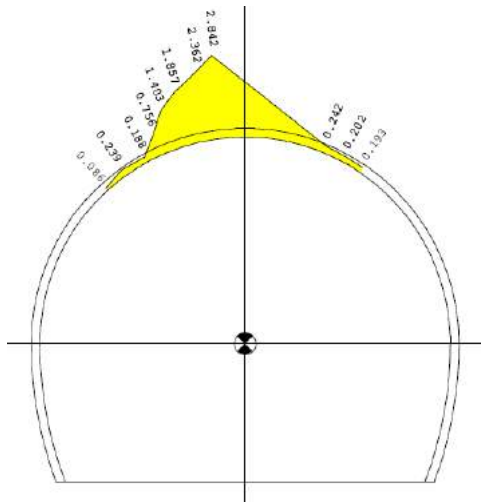


Fig. 6 - Cross-section after collapse at ch. 7235.7m as seen from South portal

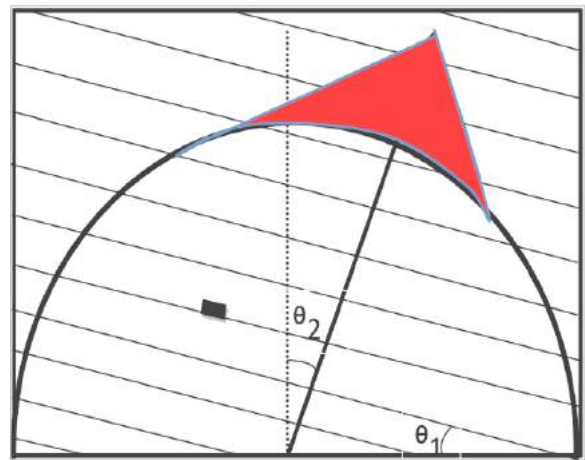


Fig. 7 - Apparent dip of joint and angle of crown of collapsed portion from vertical axis as seen from north portal

6. OBSERVATIONS ON ROOF COLLAPSE

6.1 Deformation Observed before and after Collapse

Figure 8 shows the direction of references inside the tunnel. Optical targets for measuring 3D deformation are generally installed in 10:00, 11:00, 12:00, 1:00 and 2:00 o'clock directions. Optical targets are being installed at distance of 10m but can be reduced to 5m wherever required.

All the wedges formed due to collapse had crown in approximately 1:00 o'clock direction, i.e. height of crowns of wedges measured from the periphery of the tunnel opening are maximum in this direction.

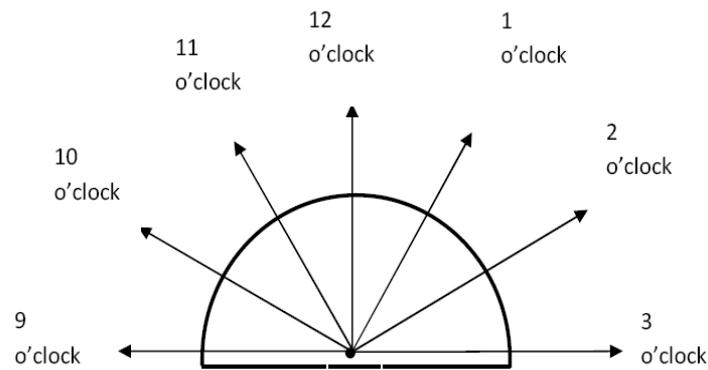


Fig. 8 - Directions for reference as seen from north portal

Figure 9 shows the deformations measured in collapsed section just one day before the collapse. Deformation in approximately one o'clock direction has been maximum throughout the collapsed length of tunnel. Maximum deformation noted in 1:00 o'clock direction was 575mm just one day before of collapse. Maximum longitudinal deformation noted were in the range of 100mm. High deformation at Ch. 7230m is due to detachment of shotcrete surface from the rock. From the figure it is clear that overall deformation of the section increases as the face of excavation moves ahead under increasing overburden.

The 3D deformation monitoring is being carried out in tunnel on daily basis. Figure10 shows the deformation at ch. 7215m just one day before collapse. As overburden is increasing from the North portal drive, squeezing is increasing. Squeezing phenomenon is visible in the form of frequent cracks in shotcrete, bending of lattice girders and high deformations in crown areas.

Steps taken to counter the problem of squeezing includes - thick layer of shotcrete, longer rock bolts, close spacing of lattice girder, installation of rock bolts after some deformation and leaving slots in shotcrete layer. A slot in shotcrete was believed to act as 'Apparent Lining Stress Controller'. This was thought to be useful in countering squeezing condition.

6.2 Geology of Collapsed Area

In North portal drive, tunnelling media consists of alternate layers of quartz-biotite schist and migmatite. Tunnel is being excavated under high overburden. At face ch. 7200m overburden was approximately 1250m.

Tunnel drive direction is $S16^{\circ}4' W$. From the geological face logs (Fig. 3), it is clear that there are two prominent joint sets and one random joint. Dip direction/ dip of foliations joint is $230-240^{\circ}/25-30^{\circ}$. Tunnel is yet to be excavated in shear zone due to Rohtang ridge structure.

6.3 Shape and Size of Cavities Formed

Cavities formed due to collapse in between ch. 7195m and 7245m in North portal drive are in saw-tooth pattern along the tunnel alignment. Crown of wedges formed due to failures were lying in between 11:30 to 1:30 o'clock direction. Depth of wedge crown is upto 3.0 m while width is upto 6.0 m (Fig. 6).

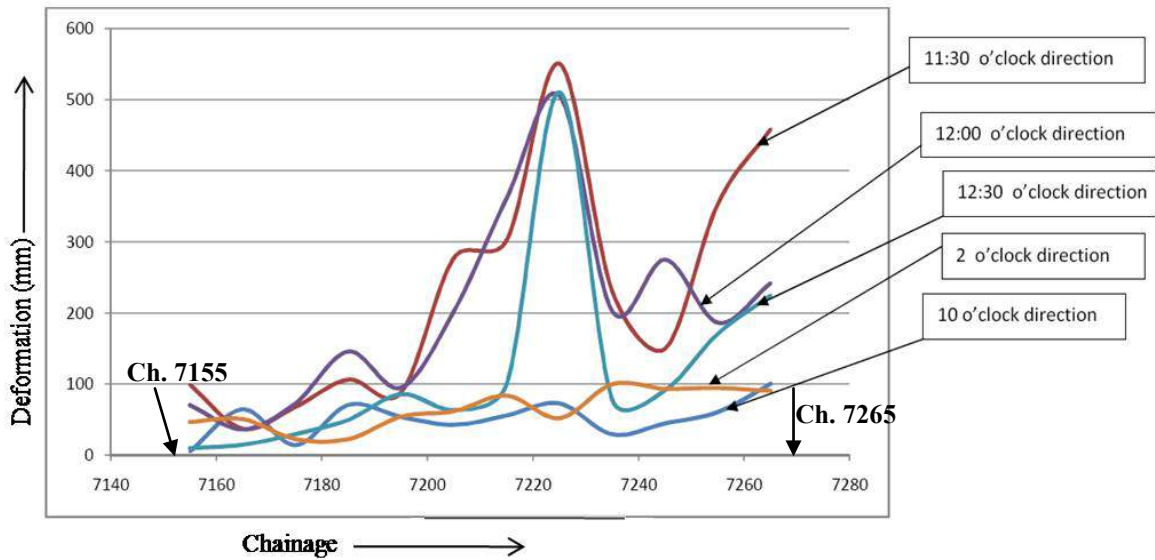


Fig. 9 - Deformation (mm) versus chainage before a day of the collapse

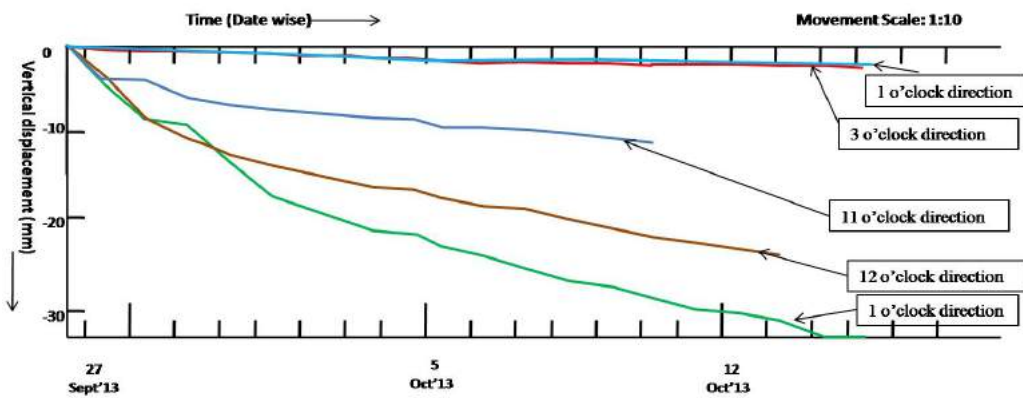


Fig.10 - A sample of deformation measurement at ch. 7215.0m using optical targets

7. REASONS FOR ROOF COLLAPSE

There are different views of experts/consultants regarding cause of roof collapse. Also there is possibility that cause is not a single factor but a combination of two or more factors. Factors being considered for roof collapse are squeezing ground condition, high stress concentration around the tunnel opening, failure due to slots left in shotcrete, late installation of support etc. Different causes of roof collapse are being explored in following paragraphs.

7.1 Squeezing Ground Condition

Table 2 shows the values of different parameters for calculating Q value at ch. 7+214. Squeezing ground condition is observed in weak rock mass with high overburden. In Rohtang tunnel, tunnelling media consists of hard rock (migmatite) and soft rock (mica schist) in alternate layer (Fig. 3). Figure 11 shows the relationship between $HB^{0.1}$ and rock mass number (N) for different squeezing condition as given by Goel (1994). Here, H is tunnel depth and B is tunnel span or diameter in metre.

Table 2 - Parameters of Q at ch. 7214m

RQD	69
J_n	6
J_r	1.5
J_a	3
J_w	1
SRF	5
H (overburden)	1250 m
B(width of tunnel)	13.3 m
J_r/J_a	0.5

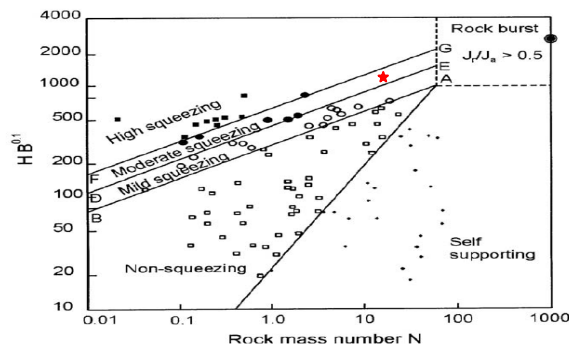


Fig.11 - $HB^{0.1}$ versus rock mass number N (Goel, 1994)

In Rohtang tunnel north portal drive J_r/J_a is equal to 0.5 and hence just the border case for the squeezing condition. Generally, deformation is increasing with increasing overburden. But deformations in 11:00 and 1:00 o'clock directions are very high as compared to values in other directions on the same cross-section of the tunnel. So higher values of deformations in specified direction is not matching with squeezing condition as stress development due to high overburden should not vary much in different direction at one cross-section of tunnel. So it is clear that failure is not due to squeezing condition alone. Other factors also must be contributing.

7.2 Rock Burst Condition

Rock bursting condition is observed in hard rock with high stress which increases generally with increasing overburden. Figure 12 shows the different combinations of J_r and J_a for squeezing and rock burst conditions as given by Kumar (2002). In Rohtang tunnel north portal drive J_r/J_a is always greater than or equal to 0.5, hence mild/ moderate rock burst condition is anticipated. Air blast is being observed after heading blast. But, no rock burst has been observed till now.

7.3 Late Installation of Support

Lattice girder, wiremesh and shotcrete are being applied just after clearing of muck from the heading area. But installation of rock bolts were being delayed upto 6-7 rounds. There was excessive deformation in 1 o'clock direction later on. The deformation could be avoided by timely installation of rock bolts. Also to seize excessive deformation some more number of

rock bolts could be installed in one o'clock direction. So late installation of rock bolts could be one reason for collapse.

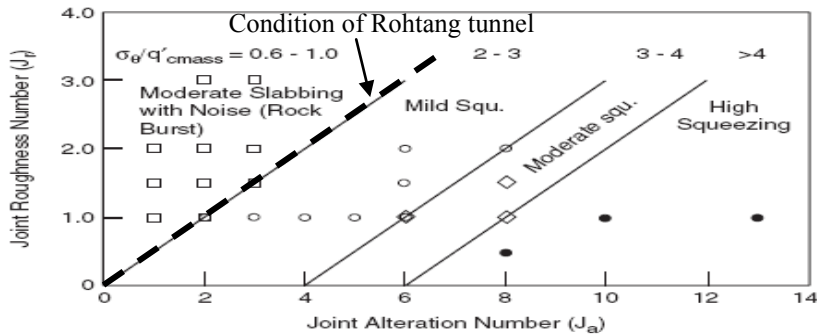


Fig.12 - J_a versus J_r for different squeezing and rock burst conditions (Kumar, 2002)

7.4 Stress Concentration around the Tunnel Opening

Figure 13 shows the typical deformation pattern in the HRT of NJPC project near (Rattanpur adit intersection) due to foliation orientation and deep seated slope movements. Similar conditions are being observed in Rohtang tunnel North Portal drive. Direction of foliation plane with respect to tunnel in the north portal drive of Rohtang tunnel is almost same as that in HRT of NJPC project. Deformation in 1:00 o'clock direction is increasing due to release of high stress under high overburden. Cracks in shotcrete, bending of lattice girders in the specified direction indicate the same condition as shown in Fig. 13. Therefore, it can be concluded that the roof collapse in Rohtang tunnel in between ch. 7245m and 7195m on 16th Oct 2013 is due to high stress concentration/release in specified direction. So rock bolt should have been provided in this part in zigzag pattern in the perpendicular direction of foliation. Later on as part of remedial measure in other location of the tunnel, where cracks were developed, rock bolts were installed in between 11:00 and 2:00 o'clock direction and perpendicular to foliation plane. It proved to be fruitful to stabilize the deformation. Hence not providing more numbers of rock bolts in this part of crown was the major cause of failure. Further this part was weakened by leaving slot in shotcrete.

7.5 Leaving Slot in Shotcrete

A slot in shotcrete was left initially in 1:00 o'clock direction and later in 11:30 o'clock direction. Initially slot in shotcrete acted as apparent lining stress controller (LSC). But, in long term it allowed cracks to expand and when rocks were unable to support itself, it collapsed. So, leaving a slot in shotcrete layer proved detrimental. It has been avoided in further excavation of tunnelling. Leaving slot in shotcrete in 1:00 o'clock found to be a major cause of failure.

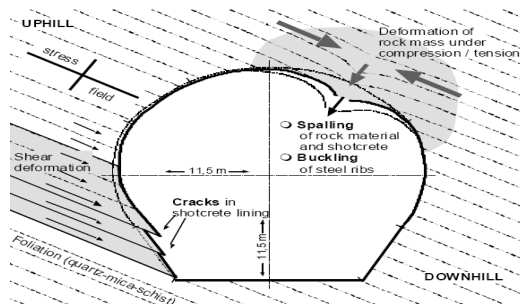


Fig.13 - Deformation/failure of rock mass under compression in approx 1:00 o'clock direction (Thuro and Eberhardt, 2001)

8. CONCLUSIONS

Stress concentration due to particular direction of foliation and effect of high overburden on it, not providing extra rock bolts in between 11:00 and 2:00 o'clock direction and leaving slots in shotcrete are three major reasons for roof collapse in North portal drive of Rohtang tunnel.

Hence following points should be noted in tunnel construction to avoid collapse -

- Slots in shotcrete should not be left to act as apparent lining stress controller.
- Rock bolts perpendicular to foliation plane should be provided especially in location where stress concentration is high. Stress flow along foliation should always be kept in mind while providing the support.
- The 3D monitoring should be always kept in watch to determine excessive deformation at any location and proper and immediate measures should be taken if needed.
- Rock bolts and wiremesh should be properly tied and also shotcrete be provided in monolithic manner so that stress concentration in any direction is distributed properly around the periphery of opening.
- Ground response curve should be developed to determine the allowable deformation before the installation of main support system.

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