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Tunneling in Lesser Himalaya, Jammu and Kashmir, India with Special Emphasis on Tectonic Mélange

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Abstract: Himalayan fold belt has full of geological surprises, 'mélange' is one of them which create difficulties during tunneling. Such mélange completely went unnoticed during surface mapping and geotechnical investigation preceding the construction of the Udhampur railway tunnel (URT). During the construction, the mélange zone has encountered across the tunnel, which occurs along the Tanhal thrust (equivalent to MBT) that separates the Murree Group and the Shiwalik Group. The mélange was characterized by a chaotic, heterogeneous geological mixture of stronger blocks (scale independence) and weaker sheared fine-grained matrix, often termed as "block-in-matrix rocks" or *bimrocks,* which enforced mixed face tunneling. The heterogeneity in a tectonic mélange led to stress concentrations in the rock blocks, and there were relatively high deformations within the matrix also. Release of stress from the blocks due to excavation, with unfavorable joint and thrust orientations enforced brittle failure of the blocks (face and crown collapses) while matrix deformation (time dependent) caused convergence of primary support later. Additionally, the clay minerals with high swelling potential within the matrix swelled and created pressure on the primary support. Due to the geomechanical heterogeneity in mélange, homogenizing the rock-mass by the commonly used quantitative systems might have lead to an inappropriate design and construction. The adopted New Austrian Tunneling Method (NATM) proved to be an useful tool for tunneling.

Keywords: Tectonic mélange, bimrock, Tanhal Thrust, deformation, NATM, Himalaya.

INTRODUCTION

The Himalayan orogenic system has evolved in response to convergence of the Indian and Eurasian plates (DeCelles et al. 2001; An Yin 2006;). This orogenic system consists of mighty mountain chains, deep and narrow gorges and valleys. Northward movement of the Indian plate is an undergoing process, due to which the Himalaya is in up-building state and is under high stress regime (especially compression) (Heidback et al. 2009). Consequently frequent earthquakes and other natural hazards, such as, landslides, etc. are inherent. Therefore, the infrastructural development of the Himalayan regions is very limited. A new railway link project between Udhampur and Baramullahin of Jammu & Kashmir is one of the national projects which may mitigate the issue of poor development in Himalayan region. Construction of long tunnels and high bridges are intrinsic on this project (cf. Kumar et al. 2008; Printzl et al. 2008).Not only had the major hindrances like earthquake and land slides, but also faults, shear zones, weathering, unconsolidated sediments, etc. create problems during construction of tunnels and bridges.

Udhampur Railway Tunnel (URT) of 3.1 km long extending from Umalla (32°56'27.75"N/75°09'30.74"E) to Sambyal (32°56'27.61"N/75°07'34.93"E) is the first tunnel on Udhampur-Katra section (~25 km) of this railway link project. This section is the gateway to the famous Shri Mata Vaishno Devi shrine. The tunnel aligned E-W, earlier was constructed by conventional method in D-shape (cf. Goel and Swarup, 2006). This was not able to withstand the developing stress and abnormal ground behavior with subsequent invert heaving, lateral displacement and deformation (convergence up to 300-1000 mm), partial and total collapse at different locations (Table 1) (Fig. 1). Realignment of the tunnel has been done and decided to build the new tunnel (~1800m) on realigned path by New Austrian Tunneling Method (NATM). The old tunnel, from east-end ~1033.5m to west-end ~351m, built by conventional method have been retained. Overall a poor geology has been encountered during excavation on the realigned path of the tunnel. However, a 90 m thick very weak zone has been crossed, which comprises block-in-matrix (*bimrock*) and is characterized by unusual ground behavior.

Table 1. Brief history of the Udhampur Railway Tunnel, Jammu & Kashmir

Sl. No.	Description			Date	Remarks
1.	Commencement of excavation at Tunnel T-01			June, 2000	1. Tunnel extends from Ch. 2180 to Ch. 5291 (~3.11 km.). 2. Time taken 4 yrs. 4 months.
2.	Breakthrough			September, 2004	
3.	During construction				
		Problems	Remedial measures		
	3.1	Seepage of water			
	3.2	Invert heaving	1. Defected ribs were replaced by new ones. 2. Bottom lifted beams were replaced by curved beams and leveled with concrete up to formation level.		
	3.3	Lateral deformation	1. Spacing of ribs reduced to 50 cm. 2. Cross strut at SPL 3. Double rib 4. Lagging between flanges of ribs. 5 Tunnel dimension increased by 20 cm.		
4.	Lining completed			2005-2006	Lining completed in 1 yr.
5.	After construction				
		Problems			Measures to avoid further collapse: 1. Placing of aggregate bags surrounding collapsed area. 2. Shotcreting, wiremesh fixing done in cavities created by collapse. 3. H-frame to strengthen side walls. 4. 3D monitoring
	5.1	Dripping of water continued			
	5.2	Total collapse			
	5.3	Invert heaving continued			
6.	Diversion of tunnel to avoid highly deformed and collapsed zone				Diversion of 1.735 km
7.	Tender for construction of new tunnel on the realigned path of Tunnel T-01 by NATM (1,735 m, incl. 10 m silent chainages)			2009	Old tunnel remains: 1.UHP end from Ch. 2180 to Ch. 3213.5 (1,033.5 m). 2. KTR end from Ch. 4940 to Ch. 5291 (351 m).
8.	Final lining started			08^{th} November, 2012	
9.	Tunnel completed			Early 2013	Udhampur-Katra train service started July'14

The present work focused on this very weak zone and reports the occurrence of tectonic mélange with emphasis on the hostile geological (ground) condition encountered while crossing the mélange during tunneling. Other aspects dealt are excavation and support systems, effects of poor ground on tunnel and methods of eradication.

GEOLOGICAL SETTING

The Himalayan orogenic system has been divided into (1) Sub-Himalaya (between Main Frontal Thrust or MFT and Main Boundary Thrust or MBT), (2) lower or lesser Himalaya (between MBT and Main Central Thrust or MCT), (3) Higher or greater Himalaya (between MCT and south Tibet detachment or STD), (4) Tethyan Himalaya, (5) Trans Himalaya (beyond the Indus Tsangpo Suture Zone or ITSZ). The detail stratigraphic classifications, geology and tectonics have been given by earlier workers (cf. Fuchs and Gupta, 1971; Fuchs, 1981; GSI, 2005; An Yin, 2006).

URT is located in the Udhampur region (70 km N. of Jammu) which covers some parts of sub-Himalayan and lesser Himalayan belts separated by Tanhal Thrust or TT (equivalent to main boundary thrust or MBT) (Fig.2). Lithologic succession comprises Tertiary (Murree and Siwalik groups) to recent sedimentary deposits (Table 2). The Murree Group composed of brown to reddish, fine to medium grained sandstone, siltstone, claystone and shale alternations, deposited in shallow marine environment and the Siwalik Group encompasses fine to medium grained sandstone, siltstone, claystone, coarse to very coarse grained sandstone, conglomerate and boulder beds deposited in terrestrial environment. The unconsolidated to semi-consolidated sediments of recent to sub-recent origin are present in fair amount. The strike of the beds varies from NW-SE to NNW-SSE and generally dipping NE at angle of 15-35° with local variations. The geology of the region is

Fig.1. The effect of lateral displacement and deformation of ground on tunnel's support system; (**A**) Invert heaving, (**B**) Buckling of cross-strut, (**C**) Buckling of ribs, (**D**) Partial collapse of crown

complicated due to multiple stages of deformation during the Himalayan orogenesis since early Miocene. Several NW-SE trending thrusts, such as Tanhal (TT), Mandili-Kishanpur (MKT), etc. dissected the region into different structural units (GSI 2005) (Fig.2). The rate of movement of thrust blocks is very high, due to which high stresses (generally horizontal stresses) builds-up here, and frequent earthquakes (in zone IV and V of Indian seismic map) occur to release these built-up stresses (Sharma and Shankar, 2001).

Table 2. Stratigraphic classification of the rock at Udhampur region, Jammu & Kashmir

Group/Formation	Lithology	Age
Undifferentiated alluvium, river terraces and glacial deposits	Pebbles, cobbles, sand, silt and clay	Pleistocene-Holocene
Upper Siwalik	Boulder conglomerate, pebble beds, sandstone and mudstone	Pliocene-L. Pleistocene
Middle Siwalik	Sandstone with inter-bedded mudstone	Miocene-Pliocene
Lower Siwalik	Sandstone, mudstone with subordinate conglomerate	Miocene
~~Tanhal Thrust (equivalent to MBT)~~~~~~~~~~		
Murree Group	Sandstone, siltstone, mudstone and claystone	Eocene-Miocene

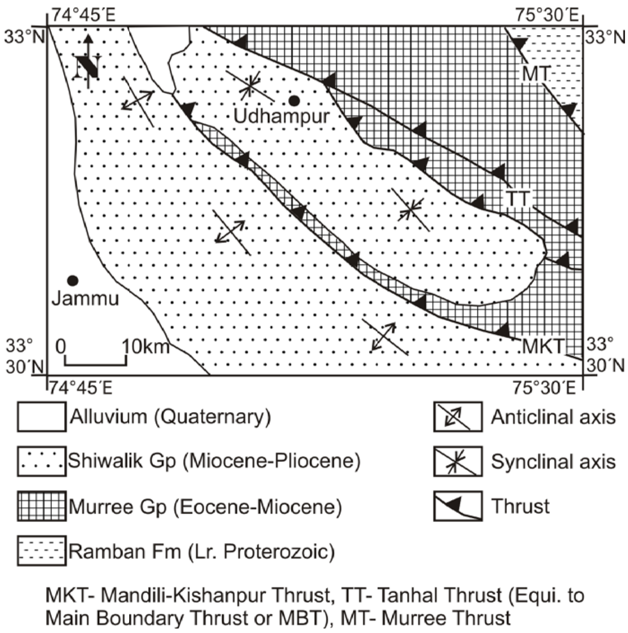

Fig.2. Geological map of the Udhampur region, Jammu & Kashmir (simplified after GSI, 2005).

Geotechnical Information

Preliminary geotechnical investigations before the construction of the tunnel have been carried out (RITES, 2008). The grain size distribution varies from gravel to clay, in which the degree of expansion or swelling pressure of clay is very high and gradually decreases towards coarser grain (Table 3). Free swelling index and Atterberg's Limit (liquid limit, plastic limit and plastic index) also have wide range. The uniaxial compressive strength (UCS) of the rock ranges from 0.52 Mpa to 81.08 Mpa. The density and the specific gravity of the rock mass vary from 2.5-2.7 kg/cm^3 and 2.62-2.92 respectively. XRD of the rock masses shows the occurrence of fair amount of mica, montmorillonite, kaolinite and illite (Goel and Swarup 2006; RITES, 2008) (Table 3). Therefore, the earlier tunnel experienced excessive swelling and squeezing (3-5%) ground conditions around tunnel meter (TM) 1320-1490, consequently invert heaving occurred and steel rib supports have buckled.

METHODOLOGY

Rock mass is rarely continuous, homogeneous and isotropic, and is usually dissected by several discontinuity planes (fractures, cleavage, joints, etc.) and other structures.

Table 3. Generalized geomechanical properties of the rock mass encountered at Udhampur Railway Tunnel

Minerals	Percentage
Quartz	18-30%
Feldspar	13-20%
Mica	18-27%
Montmorillonite	8-15%
Kaolinite	10-17%
Illite	3-10%
Iron Oxide	4-6%
Grain size	
Sand	12-52%
Silt	5-63%
Clay	12-23%
Degree of expansion	
Coarse sand	<10%
Fine to medium sand	30-50%
Silt	20-35%
Clay	50-70%
Atterberg's Limit	
Plastic Limit	18-21%
Plastic Index	8-13%
Free Swell	10-27%
Free Swelling Index	30-50%
Properties of rock mass	
UCS	0.52 to 81.08 Mpa
PLI	1.03 to 6.30 Mpa
Modulus of Elasticity	36.42 to 52.83 Gpa
Poisson Ratio	0.25 to 0.27
Bulk Density	2.5 to 2.7 kg/cm^3
Natural Dry Density	2.4 to 2.68 kg/cm^3
Specific Gravity	2.62 to 2.92
Quantitative analysis of rock mass	
Q-value	0.05 to 2.00
Rock Mass Rating (RMR)	20 to 40
Geological Strength Index (GSI)	30 to 35

Mapping of such elements is an essential component for the design of underground excavation. During construction of tunnel on the realigned path by New Austrian Tunneling Method (NATM) detailed face logging on the scale of 1:100 on an interval of 2-3 m has been done. Competent blocks of varied lithologies embedded in sheared matrix of weaker rocks have been identified and studied systematically. Estimation of block-size distribution and mechanical contrast between blocks and matrix has been done. Different engineering geological parameters, such as, weathering/ alteration, structure, colour, grain size, rock type, groundwater influence, details of discontinuities (number of joint sets, orientation, persistence, spacing, aperture/ thickness, infilling, waviness and unevenness), etc. have been collected (cf. Hoek et al., 1995).

The data collected have been utilized for rock mass classification by the methods like Q-value, rock mass rating (RMR), geological strength index (GSI) and stand-up time (Barton et al., 1974; Bieniawski, 1989; Hoek et al., 1995; Goel et al. 1995; Cai et al, 2004; Russo, 2007). RMR uses compressive strength directly and its calculation also incorporates the joint and tunnel orientations relationship while Q only considers strength as it relates to in-situ stress in competent rock. GSI has also been used here to properly demarcate the geomechanical properties of the rock mass, taking into consideration the discontinuity network and the relative geotechnical characteristics.

Geological cross-section and 3D geological log have also been prepared to show the geological variation along the tunnel path and the distribution of block-poor and block-rich zones of mélange.The structural data (dip amounts and directions of joints and bedding planes and poles) collected were analyzed by using GeOrient software. The poles were contoured for identification of exact number of joint sets, average dip directions, etc. The stereonet plot of the data has been also used to identify the wedges and the stability of the face.The modes of occurrence of the competent blocks within the matrix have been identified and analyzed. Deformation monitoring data of present as well as past have been used to analyze the behavior of the rock masses.

Due to heterogeneity, especially in mélange rock masses, the actual ground conditions at URT varied from the anticipated. Classification schemes of rock masses and determination of support requirements by using Q-value or RMR are inappropriate and simplistic in case of tunneling through mélange. These are founded on the principles of homogenizing the rock mass behavior from generic parameters. Thus selection of the appropriate support and excavation methods in a tectonic mélange need short term predictions during construction, which has to be based on a careful evaluation of deformation monitoring data and continuous updating of geological model by regular face-mapping.

ENCOUNTERED GEOLOGY DURING EXCAVATION

The rock mass conditions predicted on the basis of surface geological mapping, bore-hole data and other exploratory methods have not changed significantly. However, an important geological anomaly has been encountered during excavation.

The Shiwalik Group comprises conglomerate, sand and clay. Strength was low to medium with very short to moderately short stand-up time. In general the deposit was moderately weathered; however, often leached and highly weathered. Dry, poorly interlocked, low to medium compacted and cohesionless situation prevailed throughout. Often raveling, governed by gravitational fall due to loss of internal friction and collapse of natural arching effect has been observed. The Murree Group consists of mudstone (claystone), siltstone, sandstone and subordinate limestone. The rock mass was moderately to highly weathered, dry, sometime damp and wet. The rock mass was characterized by several joints, fractures, faults, thrusts, shears, etc. with variations in dip directions and dip amount of bedding planes. Four to five prominent sets of joints with regular orientation and some haphazardly oriented joints were also present (Table 4). Sometime microfolds and crenulations have developed in the incompetent beds, hence the axial planes of the microfolds and crenulations being parallel to the joint planes, were considered here as discontinuity surfaces. Crenulation cleavage and slaty cleavage were also discernible. The surfaces of joint planes sometime show slickensides.

Two major shear zones have been encountered with the Murree Group. First shear zone (SZ-1) of ~70 m thick was present from TM 1310 to TM 1380, whereas the second shear zone (SZ-2) of~90 m was present from TM 1465 to TM 1555. The SZ-2 was characterised by chaotic, heterogeneous mixture of blocks, with different types and sizes, surrounded by weaker sheared finer grained rocks. The character of the zone as emancipated from actual site condition, laboratory testing of rocks and deformation data inferred as tectonic mélange or *bimrock*. Detail of the tectonic mélange has been given in the following section.

Table 4. Engineering geological properties of the rock masses

1.	Number of joint sets	4 to 5
2.	Uniaxial Compressive Strength (UCS)	25 to 100
3.	Rock Quality Designation (RQD)	10 to 55
4.	Joint roughness	Smooth and undulating, planar, slickensided
5.	Joint infilling	Soft or swelling clay gouge
6.	Joint water or groundwater	Wet to dripping
7.	Stress reduction factor	Squeezing or swelling
8.	Spacing of discontinuities	20 mm to 180 mm
9.	Length of discontinuities	1 m to 10 m
10.	Joint aperture	0.1 mm to 5 mm
11.	Weathering	Moderate to high

CHARACTER AND BEHAVIOUR OF TECTONIC MÉLANGE

Tectonic mélanges are defined as chaotic, heterogeneous geological mixtures of blocks (scale independence), with different types and sizes, surrounded by weaker sheared finer-grained rocks (Raymond and Terranova, 1984). They are often termed as “block-in-matrix rocks” or *bimrocks* (Medley 1994; Medley and Goodman 1994; Medley 1998). Usually, the mélanges pose considerable challenges to the tunneling community because of their considerable spatial, lithological, geohydrological and geomechanical variability (Button et al. 2002). One of the main construction problems in mélanges is working with mixed face conditions. This can have a significant impact on the excavation, for example complicating the construction logistics by forcing the use of different excavation techniques during a single excavation cycle, with attendant delays and cost increments.

Mélange in URT starts from TM 1465 and ends at TM 1555 from the east-end and it occurs along the TT (Fig.3 & 4). It has been classified into ‘block-poor’ and ‘block-rich’ zones (Fig. 4). It comprises of sheared shale or mudstone matrix with blocks (from few centimeters to several meters) of sandstones, siltstones and limestones. Some blocks were elongated and parallel to the sheared plane resembles boudins, whereas others were angular and arranged haphazardly. The small blocks (0.2 to 2.0 m) were surrounded by sheared shale, and the blocks were elongated (boudinaged) suggesting ductile to brittle deformation under extensional regime. There were also large sandstone blocks (5.0 to 8.0 m, may not be the true length) surrounded by sheared shale with small scale faults and thrusts (200/60 E to 220/80 SE) suggesting brittle deformation under compressive and/or extensional regime. Larger blocks stabilized at relatively higher angle (20-25°) in respect to smaller blocks, which stabilized at 5-10° with the shearing direction (NE dipping). Long axes of some blocks became sub parallel to secondary shear planes (E dipping). Most intense shearing (thickness of shear seams varies from 5 to 20 cm) has been observed in block-poor zones adjacent to larger blocks. Shear seams were haphazardly oriented (130/30 NE, 280/85 S, 220/60 NW). The weakest elements in this mélange are the contacts between blocks and matrix,

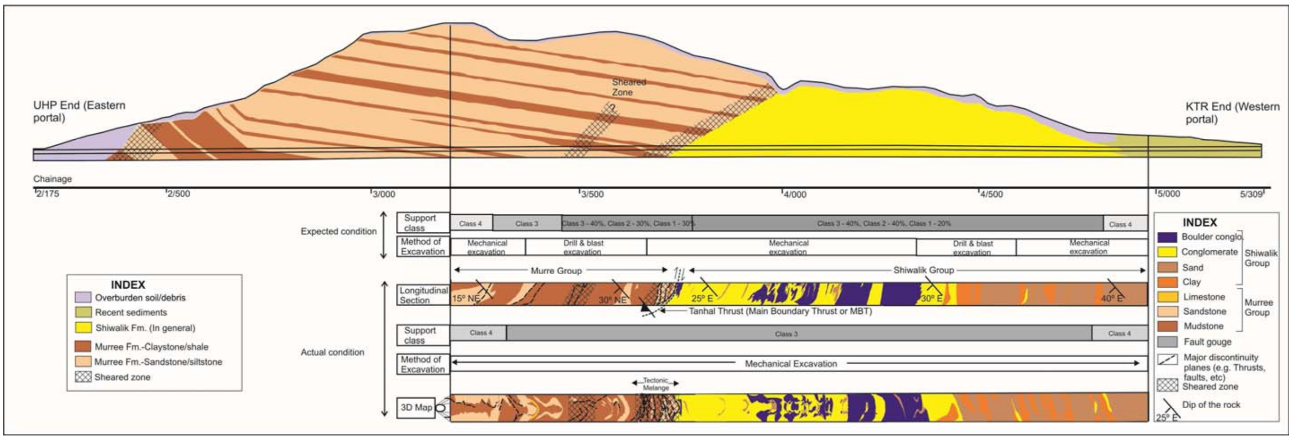

Fig.3. Longitudinal section showing the expected and actual geological condition of the tunnel with emphasis on the realigned path of ~1800 m.

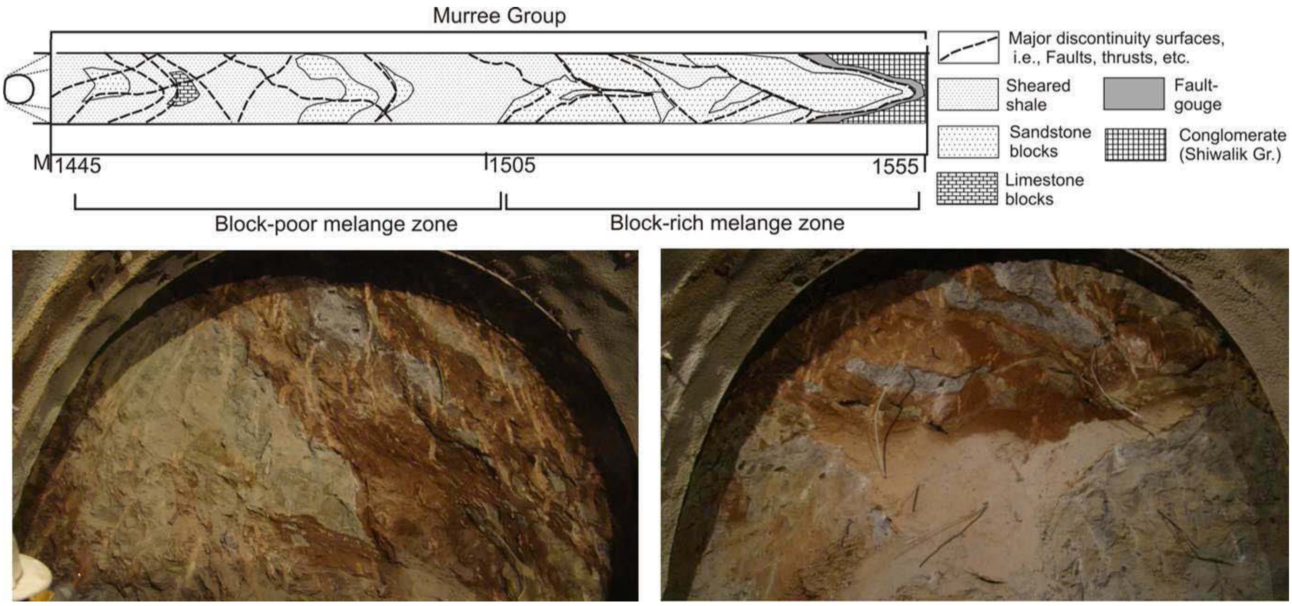

Fig.4. 3D geological map across the tectonic mélange zone with representative photos of block-poor and block-rich sections

which are usually filled-up with soft clay or gouge. Contacts were marked by lustrous surface on the blocks and a thin layer of sheared material that weathers to a shiny film of clay. Sometimes slickensides were observed on the surfaces, which showed preferred orientations (E-W, SE-NW).

The rock mass in block-rich zone was characterized by numerous discontinuity surfaces in the form of joints, fractures, faults, thrusts, shearing, etc. Four to five prominent sets of joints (330/30 NE, 260/vertical, 320/40 SW, 200/60 E) having regular with local haphazard orientations were perceptible. Sometimes microfolds and crenulations (axial plane 320/40 SW) have developed in the incompetent beds, whose axial planes were parallel to the joint planes, and were considered as discontinuity surfaces. Crenulation cleavage and slaty cleavage were also discernible. Due to the formation of wedges by conjugate joints, the rock mass collapses from the face and the crown have been observed. It has been observed that the quantitative rock mass values obtained gradually decreased on proceeding towards the thrust. Due to very blocky/disturbed to blocky or seamy appearance of the rock mass and taking into account of the in-situ stresses, overall a very weak behavioral ground has been encountered.

The character of the mélange is significant, which is controlled by the size, distribution, and the position of the blocks related to the excavation, as well as variations in the strength of the matrix, which in turn influence the appropriate tunnel excavation and support methods.The Q-value varies from 0.05 to 2, RMR being 20-40 and GSI around 30-35 (Table 3 & 4). The low values obtained inferred that the rock mass was poor to very poor. However, the values obtained were considered to be 'average', while tunneling through this mélange showed that the empirical values differed significantly within the same face. This type of mixed face situation presents unpredictable behavior of the ground. Sudden block failures from face, wedge failure from crown, etc. were unavoidable. The sheared, slickensided block/matrix contacts usually control the shear strength of failure surfaces that may subsequently develop around the blocks. The character of the mélange and the ground condition deteriorated as the tunneling progressed from the block-poor zone to block-rich zone in URT (towards the thrust). The character of the mélange in the block-poor zone was usually controlled by dominant sheared matrix. Stress concentrations in the blocks caused sudden brittle failure of the blocks (face and crown collapses) while matrix deformation (time dependent) caused convergence of primary lining later (squeezing effect).

Heterogeneous condition within the mélange arose due to difference in competency of rock masses, strength and stresses.This heterogeneity led to stress concentrations in the blocks, and there were relatively high deformations in the matrix during the excavation. In addition, adverse combination of joints, fractures and shears often caused rock mass to slide from face. Major joints and thrust planes usually dips opposite to the excavation direction and thus created unfavorable condition for tunneling. At the end of this mélange, about a meter to one and half meter thick fault gouge (breccia) consist of angular clasts embedded in muddy matrix was present (Fig. 5).

Fig.5. Fault gouge demarcating the conglomerate of the Siwalik Group in the upper portion and the mudstone of the Murree Group in the lower portion

EXCAVATION AND SUPPORT SYSTEMS

Design of URT and NATM

The design of the tunnel on realigned path of URT is based on NATM, which puts emphasis on the understanding of the behavior of ground as it reacts to the creation of an underground opening and to mobilize the self-supporting capability of the ground (cf. Muller and Fecker, 1978; Karakus and Fowell, 2004). NATM is a tunneling concept starting from the initial design stages of an underground structure until the execution and construction. The surrounding ground is not just the load acting on the tunnel but also the main construction material for the tunnel itself, hence the initial or natural strength of the ground has to be preserved. NATM is also known as sequential excavation method (SEM) works on understanding of the behavior of the ground as it reacts to the creation of an underground opening. The shape of the tunnel cross section has been chosen was curvilinear (or elliptical), which consists of compound curves in both arch and invert, to activate the self-supporting arch in the surrounding ground. Any straight walls and sharp edges in the excavation cross section have been avoided. The curvilinear geometry enables a smooth flow of stresses in the ground around the opening, minimizing loads acting on the tunnel linings (cf. Muller and Fecker, 1978). It also initiated the confinement forces and limited the bending and tension forces. The primary stress in the surrounding ground before any cavity is created depends upon overburden weight and tectonic stresses. During tunnel excavation tangential stresses increased. Induced tangential and radial stresses exceeded strength of the surrounding ground, yielding occurred and consequently a plastic zone around the tunnel formed, which significantly controlled the behavior of the tunnel. The excavation method adopted in URT was mechanical breaking (backhoe). The basic support system consists of dual lining comprising of an initial shotcrete lining and a final, cast-in-place concrete or shotcrete lining, instead of traditional thick and stiff single lining. The dual lining method is based on the principle of controlled deformation for permitting partial stress relaxation (Muller and Fecker, 1978). Initially, the ground structure remained unstable; stabilization achieved by transforming the stresses caused by overburden load into controlled deformation and into the establishment of a new equilibrium stage. Due to predicted poor geology throughout the alignment of the tunnel, the design formulated for URT was appropriate, in spite of the unidentified mélange during design stage. However, while crossing the mélange some modifications in basic design have been done during construction.

General Support System

The basic design contains the following support systems with different components (Fig. 6). Sealing of the excavated area by sprayed concrete (5-10cm thick) has been done regularly, which filled small openings, cracks and fissures

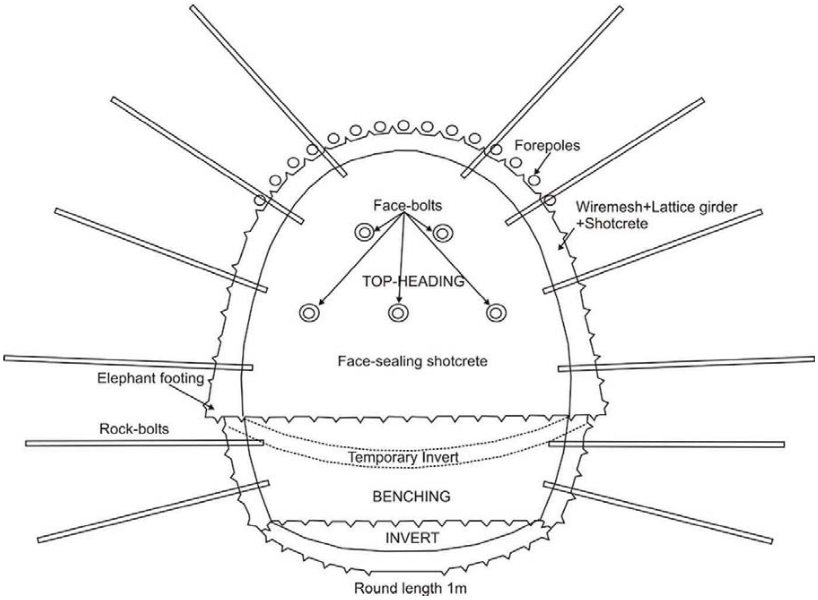

Fig.6. Schematic diagram showing the general support elements following NATM installed at Udhampur Railway Tunnel

and reduced the potential for relative movement of rock bodies or soil particles. Subsequently, wiremesh and lattice girders were fixed. Concrete grade with thickness of 20 to 25cm has been sprayed on the side walls in between two lattice girders. This entire arrangement of concrete, wiremesh and lattice girder formed the initial shotcrete lining or primary support which provided support pressure to the ground. Rock reinforcement in the form of rock bolting has been done. Rock bolting enhanced the strength of the surrounding ground, controls deformation and limits the ground loads acting upon the shotcrete initial lining.

Face and Roof Instability

Stability of the face in tunneling through such weak or heterogeneous medium depends upon the area exposed (Hoek 1999). Thus the partial excavation of face with short round length has been done in URT (by top-heading, benching and invert) such that the area of each face was small enough to control (Fig. 6).

In spite of partial face excavation (only top-heading); the stability of the face remained vulnerable, particularly in mélange zone. Modes of failures were dominantly sliding of blocks in case of block rich zone and caving and/or raveling in block poor zone. The face support has been provided by reinforcing the face with grouted facebolts (Fig.6). Not only the face, but also the roof (crown) stability was another matter of concern. Roof or crown failures were characterized by voluminous overbreaks in the crown section of a tunnel. The failure mode was governed by the gravitational collapse of the tunnel roof due to unfavorable joint orientation with conjugate pattern, increasing stress, exceeding the natural bearing capacity and internal friction of the ground. Then forepoling has been done to reduce the effect of crown failure due to the heterogeneity in ground condition (Fig. 6). These acted as umbrella arch which provided the support and prevented ground material from raveling and limited the overbreak during and after excavation.

Controlling Lateral Displacement

Temporary inverts have been laid simultaneously with the top-heading in the mélange zone, where the ground conditions were adverse. Matrix dominated by clayey gouge contained swelling clay minerals (Table 3). Hence they have the potential for volume increase by osmotic swelling as well as stress-induced shea rfailures with high radial displacements.To reduce such effects temporary invert to temporary close the ring, as soon as excavation has been done became inevitable. It reduced the radial displacements and increased the overall stability of the underground. In addition to temporary ring closure, the length of the rockbolts in the lower part of top heading has been increased (from 6m to 9m), which enhanced the strength of the surrounding ground and controlled displacement (Fig. 6).

Deformation

The deformation has occurred due to the variability in stress condition and high overburden pressure on weak rock masses. To evaluate the deformation, 3D monitoring has been done (throughout the tunnel, not restricted to mélange).

Bi-reflex targets known as deformation monitoring points (DMP) were installed within the concrete lining in the tunnel roof and at selected points along the tunnel walls (5 to 7 in a section). Vertical, horizontal, and longitudinal (in tunnel direction) movements were measured from the displacement of the targets from its installed position (cf. Kontogianni and Stiros, 2003; Barla et al. 2008).

The convergence action or the decrease in the cross-sectional area was very prominent and maximum settlement has been recorded on the crown of the tunnel. The vertical and horizontal displacements of the targets have been measured around 50-150mm, and at some places the displacement reached up to 200-300mm. The deformations have taken place soon after the excavation and installation of primary support, where maximum convergences recorded usually within the 4 to 5 days of lining. Later the rate of inward movement slowed down. This indicates that the rock masses soon released the in-situ stress locked in it and/or tunnel excavation caused a disturbance of the initial state of stress in the ground (elastic rebound). Continuous convergence (for several days) of the cross-section has also been observed at several locations, which caused immense damage (cracks in shotcrete and buckling of lattice girder).

But the observed deformations were not uniform throughout this mélange section. The rates as well as the magnitudes of deformations were high along the soft zones (block-poor or devoid of block zone). It has also been noticed that the deformations in this zones were relatively longer than in comparatively higher strength zone (block-rich zone). The rate of inward movement slowed down or completely stopped after few days (15-20 days) in the block-rich zone of the mélange. When the rate of deformation in the tunnel slowed down (<2 mm/month), final lining (final stage of dual lining) of the tunnel has been done. After final lining the cross-section of the tunnel attend the elliptical shape (Fig. 7).

Fig.7. The rounded shape of the tunnel (after final lining)

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