

## **Construction of a tunnel under thin rock overburden in Middle Marsyangdi Hydroelectric Project, Central Nepal**

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### **ABSTRACT**

This paper deals with an application of New Australian Tunnelling Method (NATM) in low cover tunnelling in Lesser Himalaya of Nepal. The length of the tunnel is 365.8 m with a 8.2 m finished diameter. The average thickness of the rock overburden is 16-18 m with a maximum of 30 m, whereas average side cover is 40 m. Top heading and multiple benching methods were applied for tunnelling work. The rational support design techniques were conceived together with Bieniawski's Support Guideline for each standard support classes.

Standard initial support system was designed according to NATM, to provide complete stabilization of excavation. It consisted of a combination of systematic rock bolts and shotcrete. The smooth blasting technique was adopted for the tunnel excavation. The specific charge was 1.39-1.47 kg/m<sup>3</sup>. A special emphasis was given in the collection of discontinuity data so that the rock mass could be evaluated effectively. Geomechanics classification for rock mass was used for the rock mass evaluation. The rock mass was also back evaluated by using Q and GSI classification on the basis of installed support. After the careful assessment of the data, the rock mass in the tunnel was classified into fair to poor according to RMR and Q and blocky / disturbed to very blocky / fair according to GSI. The rock mass parameters collected during the construction stage agree with the data collected at surface during feasibility and tendering stages. The rock mass classification based on the surface outcrop survey and drillings was a considerable success and found to be very close to the actual condition. The effectiveness of revised support system with steel rib was found to be negligible or minimum for tunnel support. Rock support deformation monitoring in the tunnel was regularly carried out to determine the efficiency and adequacy of the installed support.

### **INTRODUCTION**

#### **Project Description**

Nepal Electricity Authority (NEA) identified the Middle Marsyangdi Hydroelectric Project as a feasible project in 1994 (Fig. 1). Latter it made a detailed investigation and identified a suitable site for the construction of a dam at Phaliasangu (Lahmeyer 1997, 1998 and Fichtner 2000). The catchments area of the Marsyangdi River at the dam site is about 2729 km<sup>2</sup>. The altitude within the Marsyangdi River basin varies between 200 m and over 8,000 m. The gradient of the main river varies from 0.6 to 2% and the valley slope gradient varies between 20% and nearly vertical. Within the project area the river gradient is about 1.75%.

The project consists basically of a 68 m high diversion dam, with 5 hours regulating reservoir located in the Marsyangdi gorge, at Phaliasangu, and provided with an underground Bierl type three desanders at right bank of the river. The maximum and minimum operation levels of the dam are 626 and 621 amsl respectively. With a gross head of 110 m at a flow of 80 m<sup>3</sup>/ sec the project offers a potential of 70 MW power generation and annual energy production of 400 Gwh (Lahmeyer 1997 and Fichtner 2000).

The 8.2 m wide modified horseshoe shaped 370 m long diversion tunnel (Fig. 2) was proposed at the right bank slopes for temporary diversion of the river for the construction of spillway foundation. This tunnel was designed for the dry season flood of 20 years return period. The design discharge of the tunnel is 415 m<sup>3</sup>/ sec.

Tunnel excavation was made by smooth blasting, with top heading and multiple benching. Rock support consists of SN bolts, wiremesh reinforced shotcrete and steel ribs and partly additional shotcrete layer was also provided for smoothing shotcrete surface as additional support at the weak sections.

Diversion tunnel was proposed at the toe of sub-vertical cliff. It lies in a very close to closely jointed variably weathered quartzite rock. Rock cover above the crown ranges from less than one meter to 30 m. About 18% of the tunnel has rock cover of less than 16 m. The lateral cover of the tunnel is about 35-40 m. New Austrian Tunnelling Method (NATM) was chosen for design and construction of the tunnel. This paper provides details of the design and construction of low cover tunnel in the middle mountain of Himalayan region.

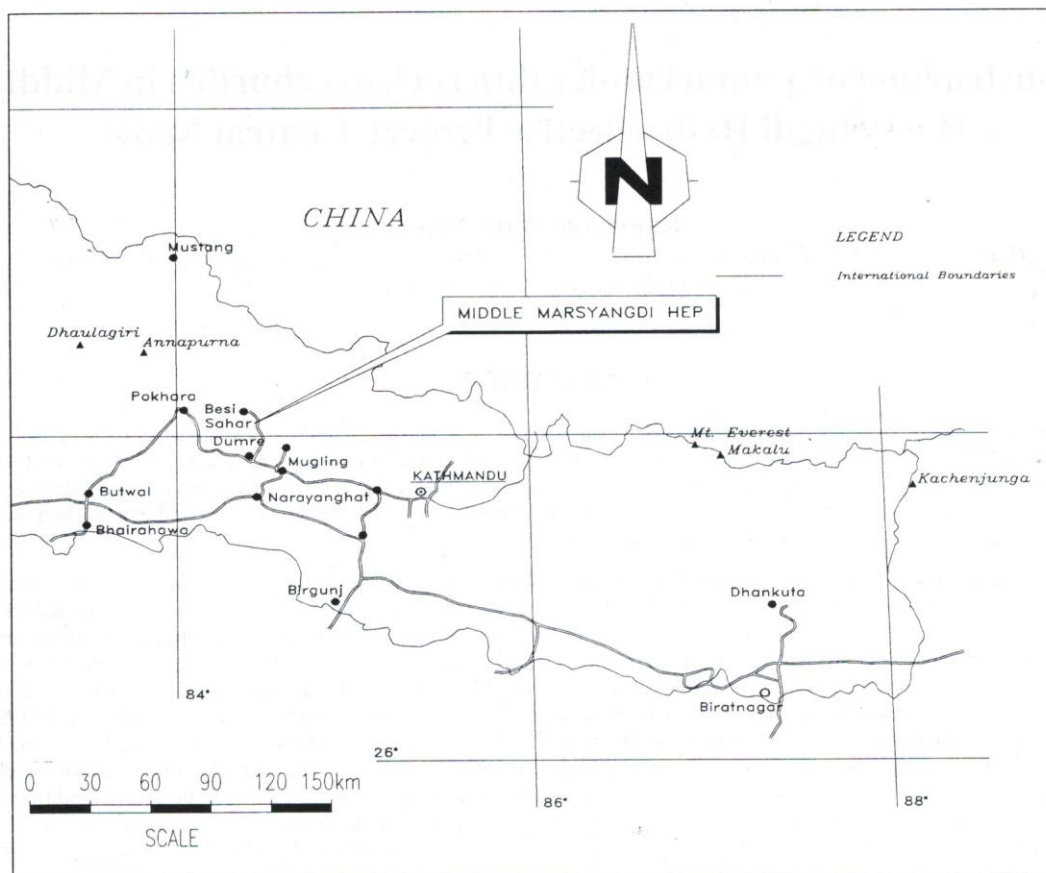


Fig. 1: Location of Middle Marsyangdi Hydroelectric Project (MMHEP), Central Nepal

## SITE INVESTIGATION

### Site Geology

The project site is located on the southern slopes of the Lesser Himalayan Range. The Marsyangdi River has eroded a deep and partly canyon like valley in the dam site area. The predominant rocks in the dam site is slightly to moderately weathered, medium to very strong fractured quartzite, the basal quartzite, member of Kuncha Formation. Glacio-fluvial deposits, colluvium, and talus deposits are also present in this area (Yamanaka et al. 1982). Two paleo-channels are traced at the left abutment of the dam, which may act as seepage paths after the reservoir impoundment. The nearest major geological structure is the Main Central Thrust (MCT) which is a north dipping thrust, located about 15 km north of the project area. The Madi Fault crosses the Marsyangdi River just upstream from the Dordi - Marsyangdi confluence and cuts across the power tunnel of the project.

### Pre-construction investigation

The site investigations performed at the diversion tunnel area includes the usual geotechnical field investigation methods such as:

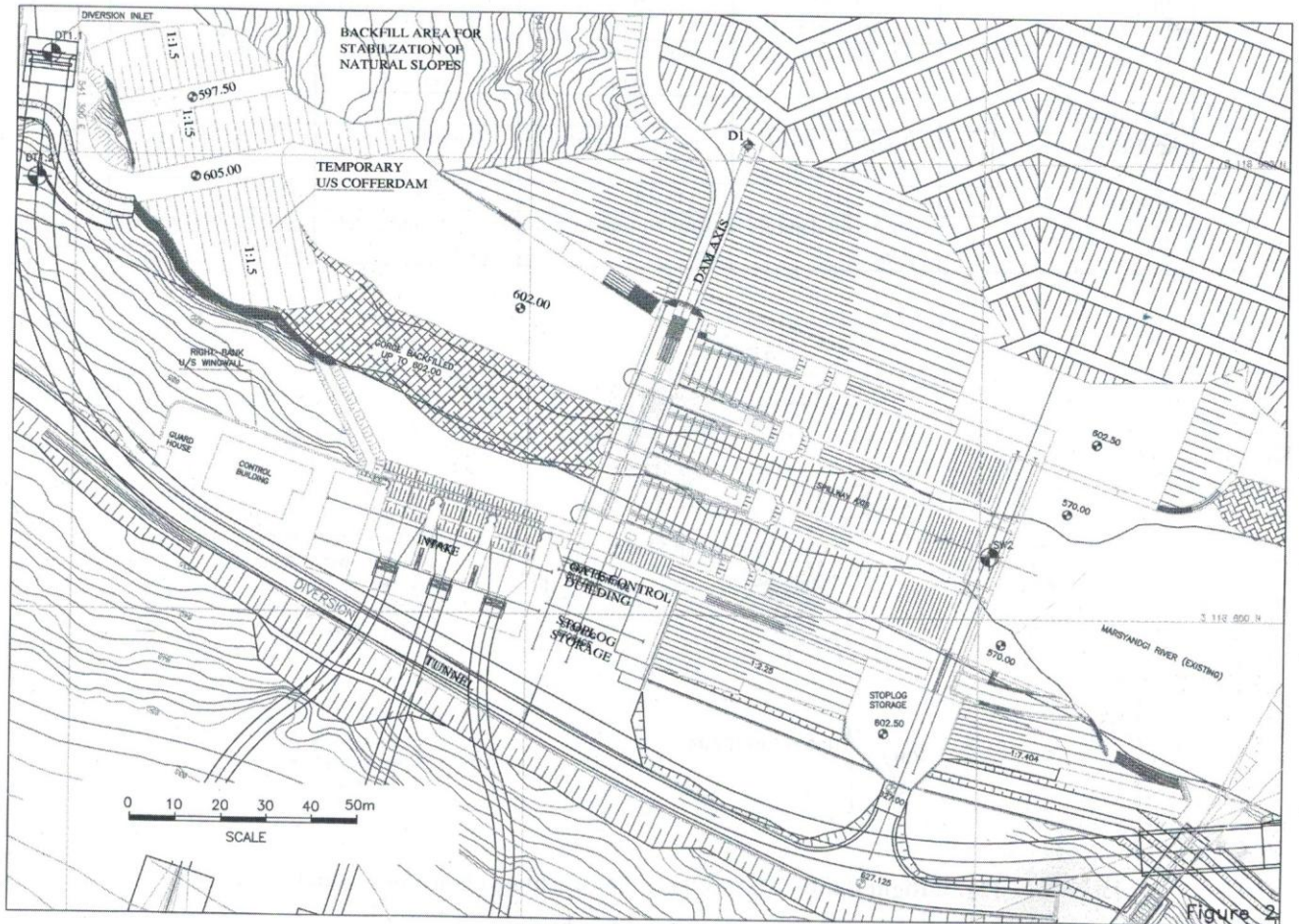
- \* Surface geological mapping on the proposed diversion tunnel area;
- \* Exploratory boreholes;
- \* Lugeon tests in all the boreholes;
- \* Installation of stand pipe piezometer in boreholes, for groundwater monitoring;
- \* About 100 m of geo-electrical profiles distributed in 2 lines.

### Interpretation of pre-construction investigation

Based on rock mass exposed around the diversion tunnel area and core drillings, a fair to poor rock was expected for the portals after removal of the first 3 m to 5 m thick surface deposits.

Approximately maximum 3-5 m of overburden consisting of colluvium and glacio fluvial deposit was estimated on the top of the bedrock. The rock cover above the tunnel alignment is 18 m in average. On the right bank slope, the rock is exposed irregularly up to 640 amsl and then the upslope is covered by colluvium. The quartzite is well exposed at the right-bank of the Marsyangdi Valley. The engineering





**Fig. 2: Layout of Diversion Tunnel and other project structures**

geological mapping of the area and borehole investigation show a presence of 100 m thick fair to poor quality quartzite. The rock is moderately thick to thinly bedded, gently dipping toward N to NNE with some local intersections of faults and minor shear zones. Minor interbeddings of phyllite are also observed at a higher elevation. A total of six boreholes were drilled at the right side of the gorge. Discontinuity parameters of rock exposures were carefully mapped and recorded on ISRM format to facilitate rock mass classification (Table 1 and Fig. 3). Some relatively thick to thin fractured zones were also logged during the core drilling. The fill material in joints consists mainly of thin mica schist layer and iron oxide. Locally some clayey material was found in some joints. Surprisingly, in one of the boreholes thin layer of loam was also found. The gently dipping rocks with nearly vertical joint set, parallel and orthogonal to the tunnel alignment,

raise potential risk of block failure especially in the presence of water.

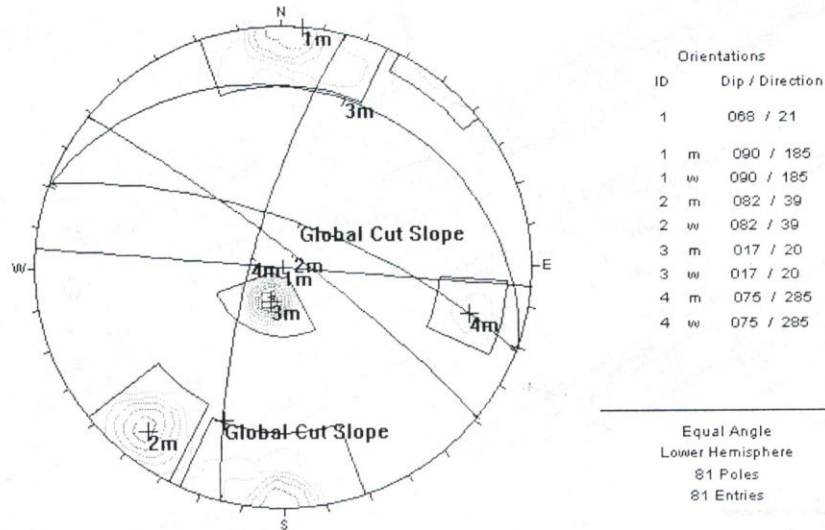
No major geological problems were expected during the excavation of the tunnel. However, it was expected that the portals may need a reinforced concrete support. Extensive lineaments splitting locally into 2-3 branches, were identified and one of them passes straight along the gorge, while the second one passes about 150 m further north. At the diversion tunnel the range of RQD varies between 50-70% and the Schmidt Hammer Index varies from 40 to 60 MPa.

The ground water level along the tunnel alignment was found to be at about 610 amsl and gradually lowers to the river water lever. However, heavy groundwater inflow from right side of the tunnel totaling up to 500l/s was expected during the monsoon periods.



**Table 1: Engineering geological parameters collected during pre construction investigation**

Discontinuity Features	Joint set J <sub>1</sub>	Joint set J <sub>3</sub>	Joint set J <sub>2</sub>	Joint set J <sub>4</sub>
Dip	20°-16°	90°	82°	75°
Dip Direction	350°-018°	185°	039°	285°
Discontinuity Type	Bedding planes	Sub vertical to vertical	Sub vertical to vertical	Sub-vertical
Persistence (m)	> 10	> 6	> 3	<10
Joint Roughness	Rough to smooth with some mica	Rough, irregular	Smooth, planar	Slightly rough
Spacing (m)	0.2 to 1.0	0.5 to 2.0	0.5 to 2.0	0.5 to 2.0
Aperture (mm)	tight to moderately open	tight to moderately open	tight to moderately Open	tight to moderately open



**Fig. 3: Right Bank – Major Plane Plot (Bedding planes and joints)**

On surface, quartzite bedrock was exposed corresponding to initial 20 m (approximate) stretch of the tunnel. Subsequently, up to chainage (Ch) 320 m the surface area covered with colluvial as well as fluvial deposits. The bedrock is exposed again between Ch. 320 to Ch. 365.80 m beyond which glacio-fluvial deposit was found. Based on the drill hole data, 2D electrical profiling and surface exposures, a geological section was prepared which was updated during the tunnelling. The RMR value was estimated from Bieniawski's (1989) revised classification (Nilsen et al. 2000; Bieniawski 1989; Bickel et al. 1996 and Wu 2001) on the basis of average parameters given in Table 2.

The rational support design was proposed also considering the Bieniawski's Support Guideline for the recommendation of each standard support classes. The proposed support classes were based on:

- Rock bolt design to support a dead load of rock.
- Rock bolt and shotcrete design based upon creation of a reinforced arch capable of supporting the arch self-weight plus an additional dead load.

- Shotcrete design based on supporting rock blocks between rock bolts.

- Rock bolts design based on the forces needed to stabilize specific rock blocks expected in the walls and roof of the tunnel excavation.

### CONSTRUCTION INVESTIGATION

During construction between chainages Ch. 26. 8 –Ch. 32.5 one block failure occurred at the crown. The block failure was due to the formation of wedge by the three major rock joints and direction of tunnelling. The estimated volume of the block was more than 1.5 -2 m<sup>3</sup>. Point load tests of the lump samples from rock face were carried out to know the intact strength of the rock according to ISRM standard. The point load strength values converted into compressive strength was ~150 MPa. This value of the intact strength of rock was considered as very strong. RQD of rock mass in the tunnel as measured ranges from 25-50% (in general), but went up to 65% at Ch. 20. 8 to 26.8. The lowest RQD percent (<10-19%) was measured at Ch.101. 4 to Ch. 112. 8. The



**Table 2: Rock mass parameters used for rock classification**

S.No	Parameter	Description	Rating	
1	Strength	4-10 (Point load)	V. Strong	12
2	RQD	40-60%	40-60%	8-13
3	Spacing of joint set	0.5-200		8
4	Condition of discontinuities			17
5	Ground water	0.4/1.0 l/s (expected) or none	Dry - Dripping	7-15
6	Total rating			52-65
7	Joint Orientation Adjustment	Fair		-5
8	RMR value			47-60
9	Rock Class	Fair		

direction of tunnelling change from N 186° (at the inlet) to N 80° (outlet). About 30cm of fractured zone was encountered at Ch.185.8. And, a notable flow of ground water was encountered from Ch. 270. 4 to 276. 2. But in general ground water condition in the tunnel was dry to damp. At some chainages the faces were wet to dripping.

The dip direction and dip of bedding plane range from 15-20°/ 10-25°. The joint sets measured at the area were 15-20°/10-20°, 75-80°/75°-80°, 185°/75-80° and random joints were 15-20°/75-80° and 270°/65°. One joint set 039°/82° was not observed during the construction instead of this; a new joint set 75-80°/75°-80° was identified. Most of the joints having 0.1-5 mm of opening were filled with silt and silty clay and at some places hard crushed materials were also observed. A minor variation of infilling materials from soft to partially crushed with some angular particles have been observed. This typical infilling material in discontinuities originates from the crushing or weathering of the minor phyllite lamina in quartzite beds. The crushed and infilling material with insignificant plasticity found within the joint openings has no major role in tunnel stability. The persistence of bedding joint was 3-10 m whereas other joints have 1-3 m. The degree of weathering of joint surfaces was slight to moderate. The joint surfaces were slightly rough to smooth. A 3 cm thick inclined shear joint (270°/65°) was found to pass through the tunnel face. The rock mass was raveling into slabs due to presence of bedding joints. Mostly the tunnel follows strike of the foliation of beds. The tabular rock mass were raveled from the crown due to very close to close foliation planes. Several dislocations, minor raveling and falling of rock slabs from crown and sidewalls occurred at different chainages. These phenomena were the result of gravitational action and the development of tensile stresses in the rock mass above the crown due to the presence of minor shear zones along the bedding planes (Singh and Goel 1999).

The rock mass parameters collected during the construction stage agree with the data collected at surface mapping during feasibility and tendering stage. The calculated RMR value ranges from 30 to 53. The rocks encountered in the tunnel were 56% fair and 44% poor according to Bieniawski's rating. The variations in RMR

value were not remarkable. Besides Geomechanics classification RMR, the rock quality was also back evaluated by NGI "Q" and GSI system according to the installed primary support for comparison purpose. The Q value ranges from 0.3 to 1.5, which indicates that the rock mass was very poor to poor whereas GSI system described the rock mass as blocky/disturbed poor (BD/P) to very blocky fair (VB/F) (Hoek and Marinos 2000; Cai and Kaiser 2003). The RMR = GSI was considered for this tunnel due to low stress environment for the tunnel. The low stress environment is due to low overburden and low lateral confinement. The spacing, length and surface of discontinuities did not show much variation through out tunnel length.

## TUNNEL SUPPORT DESIGN

Basically the purpose and service period of the tunnel was taken into consideration during initial support design. The horseshoe shaped diversion tunnel was designed for a discharge of the dry season flood with 20 years return period. The area of cross section of the tunnel is 56 m<sup>2</sup>. The design discharge of the tunnel is 415 m<sup>3</sup>/s. The main purpose of the tunnel is to pass the dry season flood for 2.5 years without structural failure. According to the basic principle of NATM, some standard initial support system was designed to provide complete stabilization of excavation. It consisted of a combination of systematic rock bolts and shotcrete. Thickness of the shotcrete varies according to support class. Three different classes of rock support was proposed which is given in Table 3.

The behavior of the rock mass at diversion tunnel inlet was primarily governed by the orientation and persistence of joint sets. One of the joint sets (bedding) was particularly unfavorably oriented with respect to the alignment of the tunnel and closely to very closely spaced from the beginning up to Ch 12. 7. After the excavation of some stretche of the tunnel it was necessary to revise some elements of the proposed support system. This requirement was to cope with these unforeseen conditions, the proposed three excavations and support classes were revised and one new support and excavation class 4a was added. The support class 4a consists of steel ribs in 1.5 m spacing with 7 rock bolts with 100 m of wire mesh shotcrete.



Table 3: Showing the support of different classes

Excavation Class	Support Class	Rock Bolt No (per m)	Length of rock bolt (m)	Shotcrete Thickness (cm)	Reinforcement (Wire mesh 3-4.5kg/cm <sup>2</sup> )
1	1	locally	2	<5 (locally)	N. A
2	2	3	3	3-5	1 layer
3	3	4	3	5-10	1 layer

## TUNNEL CONSTRUCTION

### Open cut excavation for portal preparation

The open cut excavation for the portal structure of the tunnel inlet was started from an Elev. 619 m and completed at Elev. 591 m. The excavation of slope was completed in three different stages such as Elev. 619 – 604 m first, 604-597 m second and 597-591 m third.

The rock found in the area is yellowish white, medium grained, thinly bedded, slightly to moderately weathered rock with medium to high strength. The rock mass is fractured and locally highly fractured. The major joints measured at the area were 15-20°/15-20°, 90/50°-65°, 185°/75-80° and random joints were 15-20°/70-80°, 260°/50-60°, 50/70°. Joints having 0.2-50 mm of opening were filled with clay, sand and small fragments of rock at some places. The trace of the joints were found throughout in the open cut. The degree of weathering of joint surfaces was moderate to slight. The joint surface was slightly rough. The spacing of the joints was very close to moderately close and separation of the joints was moderately open to very tight. Between the elevations 591-594 m at the right side of the portal the rock mass was fractured into small fragments. The ground water condition of the slope was damp to dry. The RMR rating ranges from 26 to 31.

The slope was cut parallel to the dip direction of the joint bedding. Smooth blasting method was adopted for the excavation of the slope. At the crown level of the tunnel, slope blasting was stopped due to difficulty to keep the berm width of 3 m due to the presence of gently dipping bedding joints and sub-vertical joints (185°/75-80°) and orientation of slope face. Due to these reasons the rock face from where the portal was to be constructed shifted toward south (into the mountain) and the tunnel length became shorter by 3.0 m. So, hydraulic hammer, ripper and excavator preceded the excavation instead of blasting. The slope was supported by 5 m long fully cement grouted SN bolts and at places 8 m long bolts were also used. The 8 m long bolts were installed with steeper inclination than that of 5 m long bolts. The spacing of the SN bolts was 2.0 m c/c. The open cut slope was supported by 50 mm of plain and 100 mm wire mesh shotcrete depending up on the site conditions. The open cut excavation for tunnel outlet was from an Elev. 598 to 593 m. The height of the slope was 5 m. The rock conditions were same as that of inlet portal. Due to the presence of three prominent joints in rock mass different sized tabular

blocks were produced. The calculated RMR value of the slope area was 28-31.

For preparation of the temporary access ramp to the diversion tunnel outlet portal, riverside rock block was supported by two rows of 8 m long SN bolts. They were installed vertically at 2 m spacing and 1.5 m laterally. Left slope face of temporary access ramp was supported by SN bolts and reinforced shotcrete. To avoid the backwater pressure on shotcrete surface, during monsoon season, a number of 50 cm long weep holes were drilled on the shotcrete surface.

### Excavation and Rock support

The portal area of the already excavated tunnel was defined by the measure of rock cover in the range of 0 m and 1.5 m times the excavation width. Only false portals were erected temporarily at the both ends of the tunnel. They comprised of steel ribs and wire mesh reinforced shotcrete. Tunnelling from inlet was started by blasting lower right part 2.45 m from the invert and half the width and tunnelling work was stopped at Ch. 16.2 due to flooding at the portal. The length of round was 1-2 m. The average rate of production was 3.1 m / working day. The NATM was developed for tunnelling in weak / squeezing ground (Sauer 1994; Barton et al. 1994; O'Rourke and Varshney 1986). As the present project area also consists of weak rocks, this method was proposed

The tunnelling was done by heading and multiple benching. The cross sectional area of the heading was 29 m<sup>2</sup>. The smooth blasting technique was adopted for the excavation. It was wedge cut and total numbers of blast holes were ± 96. Each perimeter holes were charged with a 200 gm, 32 mm diameter gelatin and about 2 m long detonating cord of 80g / m and specific charge was 1.39-1.47kg / m<sup>3</sup>. The main purpose of this type of charging in the perimeter hole was to minimize the possible geological as well as other over breaks. The depth of round was in general 1.5 – 2 m depending on rock condition. Other activities like intake tunnels and spillway excavation were also simultaneously in progress immediately above the tunnel. As the excavation preceded the rock cover of the tunnel was gradually decreased so that the tunnel stability and excavation became critical. The excavation of heading from outlet was started from an Elev. 593.8 m (invert level) sloping with 10% negative slope. The negative slope was extended up to 30 m from portal. This stretch was excavated and supported by 1.5 m spaced ISMB



steel rib with of 250 mm of reinforced shotcrete and 5 m long, 7 SN bolts / meter around the periphery of heading. Along this stretch, the diameter of tunnel varies from 10 m to 8.5 m. The reason for this method of excavation was to achieve some cover above the arch because immediately above this stretch there was crossing of Adit 2 (Fig. 4). The excavation of heading was supported by shotcreting the exposed ground. This initial application of ~ 20-30 mm thick shotcrete was applied partly at tunnel face, sidewalls and on the crown. The main purpose of initial shotcrete lining was to provide instantaneous support to hold the raveling rock pieces from crown and sidewalls and to make intimate contact with wire mesh and ground and deform with it. The tunnel face was subsequently excavated as the tunnel advanced. At the sides and crown of the tunnel, single layer of (3.01kg / m<sup>2</sup>) wire mesh was applied and the steel arch was installed at spacing of 1.5 m. It was erected as per requirement of site condition. The size of arch was 150 x 80 mm, ISMB. Steel arch together with wire mesh was installed to protect the raveling rock blocks and slabs from crown and right wall. At places where the ground condition evaluated as very poor, and the excavation was followed by 15 – 25 nos., 3 m long, 25 – 28 mm diameter fore poling bars at central part of crown, of steel quality Fe 415/500. The arch was then subsequently covered with shotcrete so that the total average thickness of shotcrete lining was roughly 200-250 mm. This type of initial support was installed up to 50 cm before the advancing face. Subsequently after few hours the shotcrete area was bolted by SN bolts according to the given revised standard support classes. Steel quality used for the bolting purposes was thermo mechanically treated (TMT) bar of Bst 500.

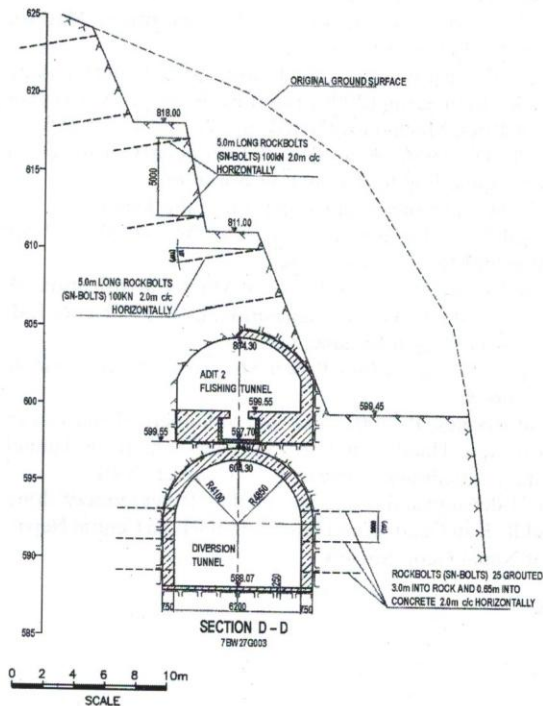


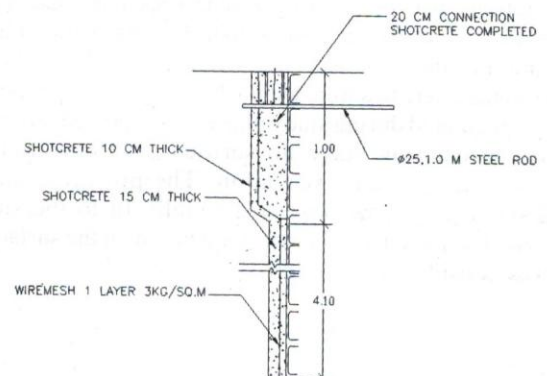
Fig. 4: Cross section showing the position of Diversion tunnel and Adit 2 at crossing.

After completion of top heading through out the tunnel, benching was started with multiple benching method. The cross sectional area of the bench part was 28m<sup>2</sup>. Right half of bench was excavated up to ten meter forward and supported by shotcrete of 20 – 30 mm thick as an initial support, after that one layer of wire mesh was installed up to invert level and immediately covered by shotcrete of thickness 100 -150 mm. Subsequently left half of the bench was excavated and supported as before at the right half. After completion of shotcrete SN bolts were installed according to the revised standard support classes. The complete steel arch was only installed in the accepted inlet portal length and 30 m transition zone from outlet portal.

Finally geometry of the tunnel was also changed from modified horseshoe shaped to inverted “D” and the steel ribs were not continued in the rest of the tunnel length i.e. lower half of tunnel was excavated straight instead of being curved in horseshoe shape. Lower portion of the tunnel was supported by new revised support for benching (Fig. 5).

#### Support deformation monitoring

Five convergence stations for support deformation monitoring were installed at different chainages of the tunnel. The five stations were Ch 48.5, 112.8, 162, 267.0 and 276.8. The monitoring was carried out by both tape extensometer and optical reading with the help of Electrical Distance Meter (EDM) theodolite. The rock mass evaluated at the stations was poor to fair according to RMR rating. At poor rock area rock mass was supported by steel arch embedded in reinforced shotcrete of 200 mm thick and in conjunction with 4 nos of SN bolts where as in fair rock area was supported by 100 -150 mm reinforced shotcrete with 8.8 nos / m of SN bolts. The maximum settlement at crown was found to be ~16 to 20 mm in 25 days after that settlement was normalized in poor rock area (Fig. 6). No signs of distress have been observed along the tunnel of the installed support. At one station where the shear zone was encountered the rock mass was moved towards the riverside by 5 mm and remaining all the other stations show no remarkable deformations.



NEW SUPPORT ADDED IN DIVERSION TUNNEL BENCHING.

Fig. 5: New revised support for Diversion Tunnel benching



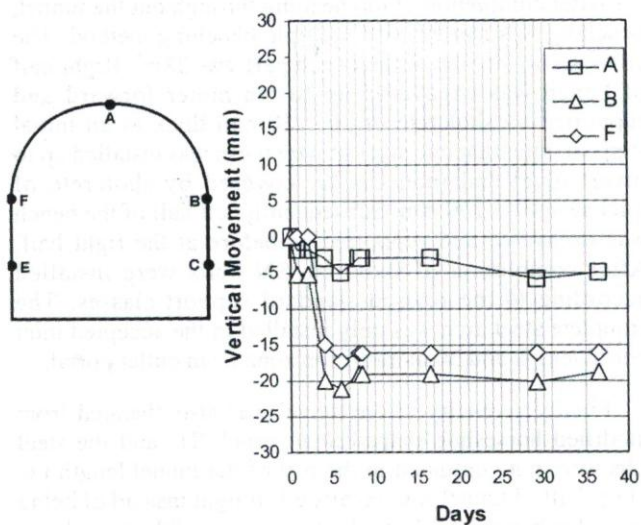


Fig. 6: Layout of deformation monitoring points at Ch. Station 0+162 and deformation results

Considering the convergence measurement of different stations in the tunnel, the elastic deformation of rock mass was quite dominating, which shows that the support installed in the tunnel was adequate. And if one compares the deformation of installed support both in steel arch and rock bolt supported area it could be concluded that support provided by the steel arch to the rock was minimum or negligible.

### CONCLUSIONS

New Australian Tunnelling Method (NATM) was used in a diversion tunnel constructed in the Lesser Himalayan rock of Nepal where several excavation activities were in progress simultaneously above the tunnel. Due to which gradually rock cover was decreased and the tunnelling and tunnel stability was being critical. The rock mass did not show signs of distress and unusual behaviors. The deformation of the rock mass was within elastic limit. The rock mass characteristics were within the limits of expectation as it was speculated during study phase. As expected during the pre-construction stage no serious problems were encountered during the excavation. The projected and revised support systems were successfully fit to the site conditions. The rock mass classification based on the surface study was considerably accurate.

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