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# *Tunnelling Through River Borne Material by Pre-Excavation Grouting Technique- A Case Study*

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## ABSTRACT

The river borne material (RBM) zone consists of very loose strata where seepage of water is found to be appreciable in comparison to other types of relatively massive rock strata. Under this condition, the tunnel excavation is not possible by conventional method either by full face drilling & blasting or by heading and bench method with fore-poling or multiple drift methods. These method of tunnelling are time consuming and also uneconomical in this type of strata. Moreover, the possibility of undermining is always there. To overcome these problems and for safer and faster tunnel excavation, pre-excavation grouting method with cement based grouts & chemical grouts (Colloidal silica, polyurethane, epoxy resins, etc.) has proved to be a very effective solution. In this method, the boreholes are suitably drilled from the tunnel excavation face into the virgin rock just in front of the face and the grouts (cementitious and/or chemical materials) are pumped-in and allowed to set. After required setting, excavation of tunnel is started through the injected and sealed rock volume. The excavation of head race tunnel (HRT) beneath the RBM zone of Dhanari Gad in Maneri Bhali Hydroelectric Project Stage-II, located in lower Himalayan region (Uttarakhand, India), has been successfully carried out with the help of pre-excavation grouting technique in comparatively lesser time.

***Keywords:*** Tunnel; RBM; Pre-grouting; Case history

## 1. INTRODUCTION

Tunnel excavation involves a certain risk of encountering unexpected ground conditions like the chances of hitting large quantities of high pressure ground water or ingress of ground water in the tunnel or in the surroundings, stability of soil or rocks for the tunnel excavation etc. To encounter these problems, proper drainage arrangement and grouting (pre-excavation/post-excavation) are, in general, carried out. Poor and unstable ground can be improved by filling discontinuities with a grout material with sufficient strength and adhesion. Grouting may also be used in filling voids to prevent excessive settlement, increasing allowable pressure of the soil both for new structures and/or additions to existing structures, control of groundwater flow, prevention of loose and loose to medium sand densification under adjacent structures due to adjacent excavations or pile driving, ground movement control during tunnel operations, soil strengthening to reduce lateral support requirement, stabilization of loose sands against liquefaction, foundation underpinning, slope stabilization, volume change control of expansive soil up to some extent, soil strengthening to increase lateral and vertical resistance of piles, etc. (Romero, 2002; Button et al., 2003).

Pressure grouting in rock is done by drilling boreholes of suitable diameter, length and direction into the rock material, placing packers near the borehole opening, connecting a grout conveying hose or pipe between a pump and the packer and pumping a prepared grout by overpressure into the cracks and joints of the rock surrounding the boreholes.

## 2. GROUTING MATERIALS

Mainly two parameters viz., particle size (fineness) and particle size distribution ($D_{95}$ max) governs the permeation capability of any grouting materials. However, viscosity, surface tension, etc. of the grout solution are also important. The ability of particulate grouting materials to penetrate a formation is often indicated in terms of groutability ratio. When the groutability ratios of natural soil [$(D_{15})_{Soil}/(D_{95})_{Grout}$], cohesive soil [ $(D_{10})_{Soil}/(D_{95})_{Grout}$] and rock [ Width of fissures/$(D_{95})_{Grout}$] are more than 24, 11 and 5 respectively, grouting is consistently possible and if their values fall below the respective values of 11, 6 and 2, grouting is not possible. The maximum particle size of the grouts should be small to avoid premature blockage of fine openings, caused by jamming of the coarsest particles and filter creation in narrow spots (USACE, 1997).

### 2.1 Types of Grouting Materials

In general, two types of grouting materials are used in grouting which are shown in Fig. 1.

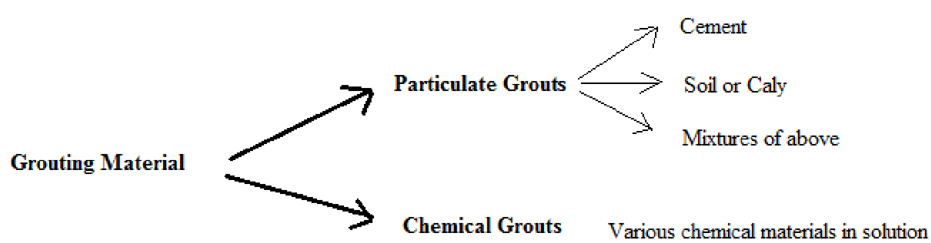


Figure 1: Types of grout material

#### *2.1.1 Particulate grouts*

Any type of cement may be used for injection purposes, but coarse cement with relatively large particle size, can only be used to fill bigger openings. Some typical cement grout materials with fineness and particle size are shown in Table1.

Table 1: Different types of grouting purpose cements

| Sl No. | Cement type | Specific surface Blaine ($m^2$/kg) | Particle size (µm) |
|---|---|---|---|
| 1. | Low Heat Cement for massive structures | 250 | 40-150 |
| 2. | Standard Portland Cement | 300-250 | 40-150 |
| 3. | Rapid Hardening Portland cement | 400-450 | 40-150 |
| 4. | Extra Fine Rapid Hardening Cement (Less availability) | 550 | 40-150 |
| 5. | Super fine /Ultra fine/micro-cement (Specially ordered & more expensive) is nothing but a pure Portland cement | >625<br>625-900 | <40<br>$D_{95}$ < 16 µm<br>and an even & steep particle distribution |

Bentonite is used in combination with cement for dispersing the cement solution and for reducing the bleeding with a standard dosage of 3 to 5% by weight of cement.

*2.1.2 Chemical grouts*

Chemicals grouts consist of only liquid components, which lead to a quite different behaviour than cementitious grouts. They behave like Newtonian fluids, demonstrating viscosity but no adhesion. Therefore, the penetration distance from a borehole and the placement time for a given volume, only depend on the viscosity of the liquid grout and the injection pressure used.

For practical purposes there are three main groups of chemicals grouts available,

- Reactive plastic resins (Polyurethane, etc.)
- Water-rich gels (Colloidal silica, silicates, etc.)
- Others (Epoxy, asphalt, acrylates, acrylamide, etc.)

## 3. PRE-EXCAVATION GROUTING OR PRE-GROUTING

In this method, the boreholes are drilled from the tunnel excavation face into the virgin rock in front of the face and the grout is pumped in and allowed to set, before advancing the tunnel face through the injected and sealed rock volume whereas in post grouting method, the drilling for grout holes and pumping in of the grout material takes place somewhere along the already excavated part of the tunnel, because of unacceptable water ingress or to provide sufficient strength and adhesion to rock (Garshol, 2003). Diagrammatic exhibition of pre-excavation and post excavation grouting is shown in Fig. 2.

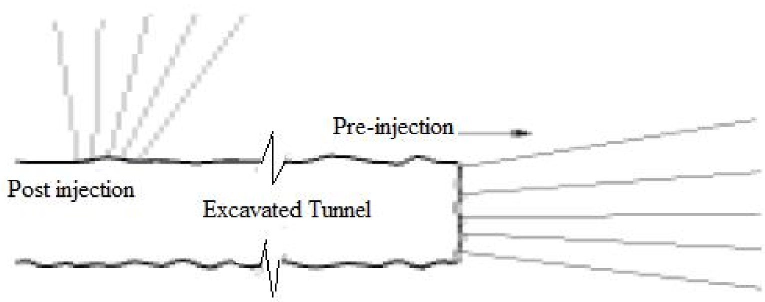


Figure 2: Pre-injection and post-injection Method

### 3.1 Use of Pre-Excavation Grouting in Tunnel Excavation of Maneri Bhali Hydroelectric Project Stage-II - A Case Study

The Maneri Bhali hydroelectric project (Stage-II) is located on Bhagirathi river in lower Himalayan region in Uttarkashi district of Uttarakhand state, India. The hydraulic head available between Tiloth Power House of Maneri Bhali stage - I and Dharasu, the tail end of Tehri Dam reservoir is about 285m, which has been utilised for power generation by Maneri Bhali Hydro electric Project stage-II, constructed and commissioned in 2008. The project includes a 81m long barrage; a 16 km long and 6m diameter horse-shoe shaped head race tunnel (HRT); restricted orifice type, 14m diameter & 174m high surge shaft and 4 numbers of penstocks and surface powerhouse at Dharasu with a capacity to generate 304 MW of electricity (4 x 76). To facilitate a speedy progress of the tunnel, construction was planned from five ends viz. Joshiyara d/s (Face-I), Dhanari Gad u/s (Face-II), Dhanari Gad d/s (Face-III), Dharasu u/s (Face-IV) and Dharasu d/s (Face-V). This has been achieved by creating two intermediate approach adits viz Dhanari Gad adit (780m long) and Dharasu adit (137m long).

### *3.1.1 Geology of project*

Maneri Bhali Hydro Electric Project area lies within the MHB, in the mid-lands of Lesser Himalaya, which is bounded to the north and south by regional tectonic features-the Main Central Thrust (MCT) and the Main Boundary Fault (MBF), respectively. The former separates the meta-sedimentary sequence of Lesser Himalaya to the North from central crystalline rocks of higher Himalaya and the latter disjoins the physiographic unit from the molasse sediments of Frontal Foothill Belt (FFB) in the south.

The Dhanari Gad area comes under the Lesser Himalayan rocks which are well exposed as Garhwal Group in the Bhagirathi valley and extended from village Nalupani (near Dharasu) to Sainj (near Kumalti Gad). These rocks comprise quartzite, limestone, chlorite schist and meta-basics with quartzite being the most prominent litho unit. These rock types form prominent high hill range on either side of the river and along the tributaries/Nalas of Bhagirathi river.

Two major stratigraphic and tectonic units have been identified in the project area. These are referred to as the Jaunsar Group and Garhwal Group. These two stratigraphic units are separated by a major tectonic break i.e. Srinagar thrust, which is the North-west extension of North Almora Thrust (GSI Report, 2007; Dangwal et al., 2014).

The Project area falls in zone V of the seismic zoning map of India, wherein a seismic intensity of VIII and IX on MSK-64 scale is expected. The cover over the tunnel varies from 28m to 1150m. The heterogenic conditions, the sheared litho contacts and crushed rock mass in Garhwal Group posed stability problems in the form of loose fall and over-breaks. Figure 3 shows drainage and HRT alignment.

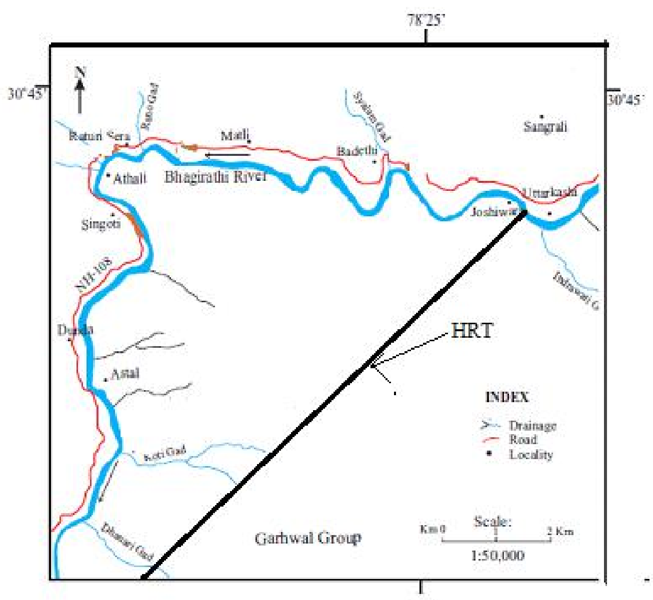


Figure 3: HRT alignment and drainage between Uttarkashi and Dhanari Gad

*3.1.2 Need for special treatment in RBM zone during excavation*

Face-III starts from chainage 8,000 m upto 12,000 m. The tunnel passes beneath the Dhanari Gad with low rock cover and about 40-90 m RBM (river borne material) cover from Ch. 8.5 km to 8.7 km. The old reach, having lesser section was designed for steel liner of 5.4 m diameter. After restarting the construction, NATM (New Austrian Tunnelling Method) was adopted. The face was fully filled with seepage water and tunnel was charged (Fig. 4). After steady and slow dewatering process, the RBM reach was reclaimed at slow pace by lowering down the invert, without touching the crown, as per advice of Geological Survey of India (GSI) and could be reclaimed, even at lesser section of the tunnel which is most difficult to encounter with. In this section, for facilitating the movement of drill boomer, trucks for mucking, transit mixer, etc., further enlarging the tunnel section is not possible by conventional method either full face drilling & blasting or like heading & benching method with fore-poling or multiple drift method etc. as possibility of failure of tunnel is always there, moreover these methods are time-consuming and uneconomical also in this type of strata. The excavation and lining both need special methods and treatment to get through successfully in RBM zone. A by-pass of this RBM reach had been completed in full required section on the advice of Board of Consultants using pre-grouting method (IS 4880 : Part V : 1972; IS 4880 : Part VI : 1971; IS 5878 : Part II : Sec 1 : 1970; IS 5878 : Part III : 1972; IS 5878 : Part IV : 1971; IS 5878 : Part VII : 1972).

(a)

(b)

Figure 4: Water coming from the tunnel face and flooded the tunnel floor

*3.1.3 Procedure of tunnel excavation in RBM zone using pre-excavation grouting method*

In this method, pre-injections of superfine cement and colloidal silica were used in RBM zone for its stabilization before excavation. As per site conditions, typically 15m drilling and grouting were decided. For working purpose at site, a 15 m drilling and grouting cycle is named as one grout fan. The whole procedure of tunnel excavation in RBM zone was completed in different steps as described below:

<u>Step 1 - Shotcreting:</u> First of all a layer of about 15 cm thick SFRS (Steel Fibre Reinforced Shotcrete) of $A_{10}$ $M_{25}$ grade was sprayed on tunnel face, side walls and crown.

<u>Step 2 - Grouting with Micro-cement:</u> After setting of shotcrete, drilling of grouting holes were done at an angle of about 5-7$^o$ outward of tunnel alignment/periphery with multi-boom drilling jumbos with 54 mm drill bit. The grouting holes were kept about 50 cm inside the

tunnel section at spacing of 1.5 to 2m centre-to-centre. One grout fan (drilling & grouting up to 15 m) was completed in four following stages.

In Stage-I, drilling was done as described above up to 4 m around the periphery of the tunnel. After drilling, all boreholes were tightened by 3m long casing pipes of 54 mm (2"Ø) with packer (49 mm Ø) and the tunnel surroundings were sealed by doing grouting with micro-cement having fineness > 625 $m^2$/kg and $D_{95}$ < 16 μm at water/cement ratio 0.53 and dispersing admixture keeping the grouting pressure up to 60 bar.

In Stage-II, all the boreholes were re-drilled up to 8m and grouting of all boreholes was done as per the procedure adopted in stage-I with a change in water-cement ratio ranges from 0.8 to 1.0 in place of 0.53.

In Stage-III and Stage-IV, re-drilling of same boreholes up to 11m and up to15m in stage-III and stage-IV respectively was done. After re-drilling in each stage, grouting of all boreholes were done stage wise as done in stage-II.

Step 3 - Grouting with Colloidal Silica: Grouting with colloidal silica is done only up to 11 m in 3 stages. For this, new boreholes were done about 50 cm inside the earlier boreholes at same spacing and outward directions and were named as 'Control Points' arbitrarily to distinguish them from earlier ones. The control points were kept in staggered positions with earlier boreholes.

In stage-I, all the control points were drilled up to 4 m and were grouted in the same way as done in stage-I of step 2 with micro-cement.

In stage-II, all control points were re-drilled up to 8 m and after fixing packers, all the control points are grouted up to 60 bar pressure with colloidal silica (specific area 80-900 $m^2$/gm & particle size 0.016 μm) and about 15% accelerator (15-20%) maintaining gel time of 15 minutes. After completing Stage-II, drilling was done up to 11 m and then, grouting was done in the same way as done in stage-II.

Step 4 - Blasting up to 1 m: After completing the step-1, 2 & 3, blasting of tunnel was done with a pull of 1 m.

Step 5 - Initial Mucking: After blasting up to 1 m, loose blasted materials were cleaned from the face of the tunnel.

Step 6 - Initial Shotcreting: After initial mucking, SFRS of $A_{10}M_{25}$ grade was sprayed about 15cm thick layer on the excavated portions (face, walls & crown) of the tunnel in order to check the falling of loose materials.

Step 7 - Final Mucking: After initial layer of shotcreting, mucking all the blasted materials was done and site was prepared for the erection of rib.

Step 8 - Erection of Supporting Rib: After final mucking and cleaning waste materials from site, Rib of ISHB 160 was erected at a spacing of 0.75-1.00 m centre-to-centre to strengthen and support the tunnel.

Step 9 - Spilling: After erection of ribs, spilling of roof, i.e. spilling above spring level of tunnel was done with 45 mm dia. bars at spacing of 0.75-1.25m centre-to-centre up to 2-3m inside the virgin rock/RBM in the upward direction at an angle of $5^0$-$7^0$ to support the upper materials by making just like an umbrella.

Step 10 - Final Layer Shotcreting: After spilling, a final layer of 10-15cm thick SFRS of $A_{10}M_{25}$ was sprayed on the side walls and crown to cover the ribs for strengthening and supporting the excavated tunnel section. Fig.5 shows the excavated portion of tunnel.

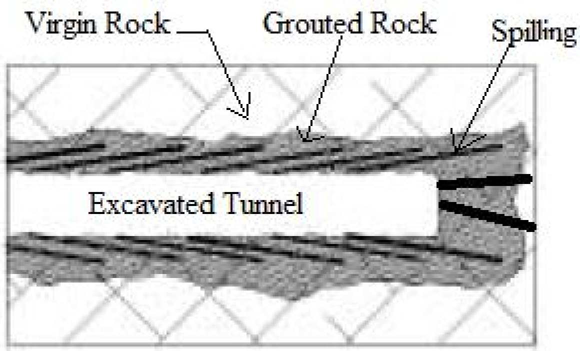


Figure 5: L-Section of excavated tunnel above spring level by pre-grouting method

Step 11 to Step 17 - Excavation of Next 8 m Tunnel: After completing steps 4 to 10, excavation of tunnel up to next 8m was done step by step in 7 steps after completing all the procedure adopted in step 5 to step 10. In this way 8m tunnel was excavated in one grout fan. For further excavation, step-1 to step 17 were recycled.

Step 18 - Final Lining: After completing the excavation of tunnel through RBM zone, lining of tunnel was done by 600mm thick reinforced cement concrete (RCC) of $M_{40}$ grade with silica fume in place of 300mm thick plain cement concrete (PCC) of $M_{20}$ grade as done in normal conditions. After lining, contact and consolidation grouting were also done with OPC, micro-cement and colloidal silica.

## 4. DISCUSSIONS

A comparative study of different parameters of different methods of tunnel excavation in RBM Zone was done which are summarized in Table 2.

Table 2: Comparison between Conventional and pre-excavation methods in RBM zone

| Sl. No. | Description | Conventional Method | Pre-Excavation Grouting Method |
|---|---|---|---|
| 1 | Time cycle | 0.25 to 0.30 m/day | 0.50m/day |
| 2 | Safety factors for workers | Less | More safer than conventional method |
| 3 | Chances of collapsing of the tunnel | More | Very Less |

Timely and safer constructions are the main essence of any hydro project. However, the cost of excavation of tunnel by pre-excavation grouting method in RBM Zone was found about

25% more than that of conventional method. But on the other hand, pre-excavation grouting method is about 40% faster than conventional method. Pre-excavation grouting method is safer for the construction personnel and chances of tunnel collapse.

## 5. CONCLUSION

For the construction of any projects many aspects are considered such as optimal cost, time, safety health & environment of labours, etc. The purpose of tunnel grouting in majority of the cases is to control ground water in-flows and help to improve ground stability. For faster and safer tunnel excavation in RBM zone, pre-excavation grouting method proves very useful tool in the construction industry. The case history presented herein is an excellent example of the use of pre excavation grouting technique.

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