Pre-Grouting in the HRT of Teesta III Hydroelectric Power Project, North Sikkim, India

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Abstract

The geology of the Himalayan region is full of surprises with a large number of big and small geological structures such as folds, faults, joints, thrusts, shears zones occurring in association with heavily fractured, sheared, fragile and weak rocks. Together, they make a challenging task for underground excavation in the ongoing projects in the Himalayas. Most of the problems can be attributed to lack of knowledge of the sub-surface geological condition ahead of the face. In general, poor and unsatisfactory progress in the underground works noticed in the Himalayan region is due to inappropriate planning and excavation methodologies. This leads to several constructional problems such as over break, cave-in formation and high water ingress into the excavation, resulting in causalities to the workers, delays in advancement of project and thereby cost overruns.

The above mentioned problems have also been observed in the Head Race Tunnel (HRT) of Teesta Stage III, HEP in North Sikkim, India. However, to mitigate these problems, a pre-grouting technique has been adopted throughout the tunnel progress. In the present paper, the problems encountered in the project have been detailed, and the techniques involved and remedial measures undertaken are discussed in brief. It has been found that pre-grouting increases the stand-up time, enhancing safety of the workers. Thus it is a viable option for rapid and safe excavation and progress of the project.

1. Introduction:

Water is the most precious nature's gift to the mankind, and present enormously in the rivers originating from the Himalayan region, which makes it a potential region for the development of hydroelectric power (HEP) projects. In general, HEP projects are comparatively much cleaner and greener than the other conventional power projects based on fossil fuels. As such, Teesta Stage III is a 1200 MW run-of-the-river HEP project under Teesta Urja Limited (TUL), located in the North Sikkim Himalaya. The project consists of 60 m high concrete faced rock fill dam near the Chungthang village and the underground power house site near Singhik village of North Sikkim, India (Figure 1). The main feature of the project is high head of about 800 m against 14.6 km long and 7.5 m diameter, horse-shoe shaped, Head Race Tunnel (HRT) having a design discharge of 175 cumecs (Figure1). The HRT of the project lies in the Central Crystalline Group, subdivided into Chungthang Series, Darjeeling Gneiss and Rongli Formation of Pre-Cambrian age and is also well known for the presence of several geological structures such as folds, faults, joints, thrusts, shears zones occurring in association with heavily fractured, sheared, fragile and weak rocks (Raiana, et al., 1965). Therefore, the excavation in this region is a difficult and challenging task. Most of the problems arise due to lack of the knowledge of sub-surface geological condition ahead of the face. Also, it leads to several problems during the construction, such as over break, cave-in formation and high water ingress into the excavation, resulting in causalities to the workers, delays in meeting the project schedules and thereby cost overruns.

Pre-grouting is a viable and rapidly growing technique used in tunneling to increase the stability by improving the rockmass, prevent failure, reduce water ingress and enhance safety of the workers (Singh and Goel, 1999; Ewert, 1985; and Houlsby, 1990). Also, pre-grouting helps in improving the mechanical properties of rock masses (Singh and Goel, 1999) in order to enhance their stability during construction, and during subsequent service (Ewert, 1985). Earlier studies were related to the grouting procedures, where the researchers have carried out rock grouting operations (Bruce, 2007; Björn Stille and, Gunnar Gustafson, 2010; Vervel, J., 1989; Weaver, K., 1991). As such no clear guidelines are available for the practical application on pre-grouting techniques in tunneling.

With this in view, a pre-grouting technique has been adopted throughout the tunnel construction to mitigate the above mentioned problems. Also, the problems encountered in the project, the techniques involved and remedial measures undertaken are discussed in brief in this paper.

2. The study area:

The study area, Teesta stage III, is located in the North Sikkim district of Sikkim, India (Figure1). It lies between latitude 27° 36' N and 27° 31' N and longitude 88° 39’ E and 88° 32' E and is included in Survey of India toposheet no. 78 A/10 on 50000 scale. Seismically, the entire Sikkim region falls in the Zone-IV of seismic zoning map of India, prepared by Indian Bureau of Standards (IS: 1893-2002).

Geologically, the HRT is aligned on the right bank of the Teesta River passing through the rock formations of Central Crystalline Group, subdivided into Chungthang Formation, Darjeeling Gneiss and Rongli Formation of Pre-Cambrian age falls in rockmass class of II to V (Raiana, et al., 1965; Roy and Ray, 1972). The initial portion, 8.5 km of tunnel length is roughly aligned through the Chungthang Formation whereas, the intermediate length of 8.5 to 12 Km is through the intrusive Granitic Gneiss and the terminal portion is through the Darjeeling Gneisses. These units are highly folded, sheared, brecciated & crushed. The Chungthang series consists mainly of quartz biotite gneiss, biotite schist, calc-silicate rocks, and thinner bands of garnetiferous-sillimanite bearing micaceous gneisses.

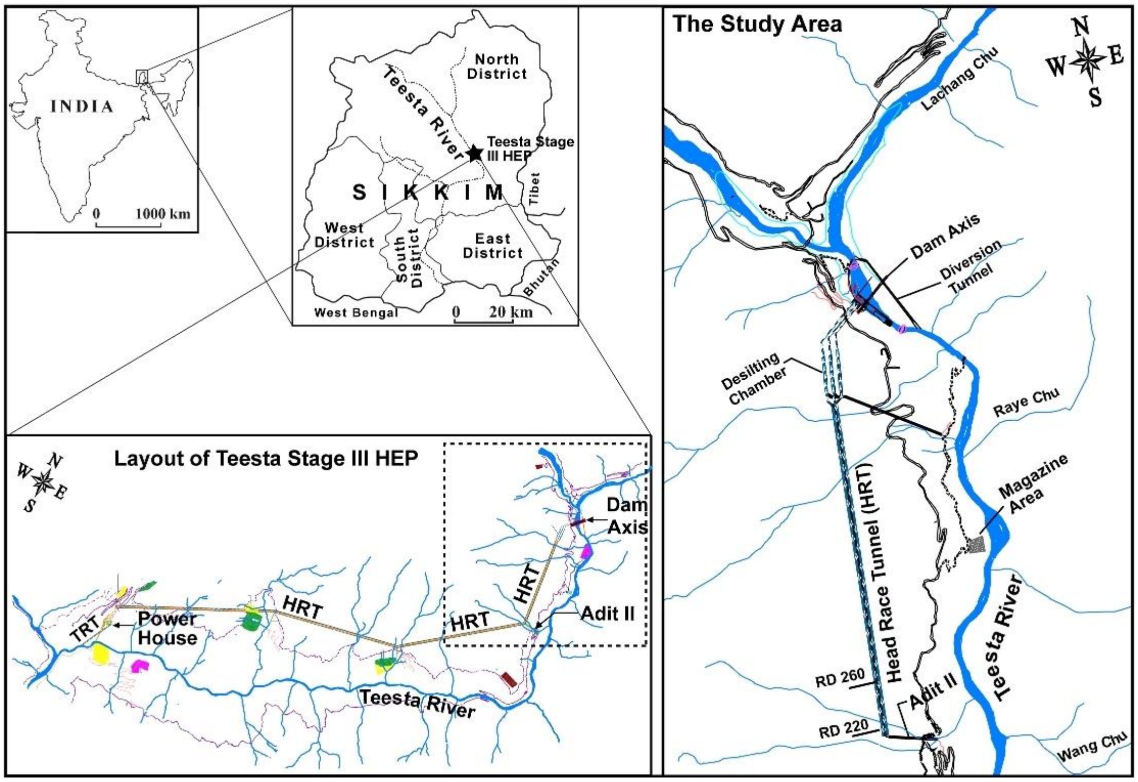


Figure 1 The location map of the study area.

The rocks belonging to Darjeeling Gneiss represent highly metamorphosed argillaceous sediments. They are mostly garnet rich kyanite and sillimanite bearing gneisses. Near its upper contact with the Rongli Schists, the rocks are little schistose in nature. The sillimanite bearing gneisses are intruded by tourmaline rich granites & pegmatite. The intensity of the intrusion increases towards north. Rongli schists are seen in the area between the Teesta and Talung valley. They represent alternate bands of argillaceous and arenaceous rocks. The rocks are mostly garnet rich biotite schist, staurolite bearing garnetiferous schists and quartzites. They lie conformably over each other, having a gradual change in rock types from Darjeeling to Rongli and Rongli to Darjeeling.

Structurally, the Sikkim Himalayas can be divided into three structural zones as, Darjeeling – Tolung Syncline in the southern part, Kanchenjunga – Lama Angdang Antiforms in the central part and Trans Himalayan over fold in the Northern Part. Earlier workers [1, 10] have demarcated thrust surfaces in between the various formations and in some particular cases even in between the individual litho units. The general trend of rocks is NW/SE dipping moderately in NE. Joints criss-cross the rockmass, with steep dip towards SE. Due to complex folding, Gneissic and Schistose bands are intricately folded with the Meta sedimentary units. The folds are generally asymmetric and isoclinal in nature with more or less North Easterly dips. Besides intense folding, the rocks are subjected to extensive shearing, fracturing and folding accompanied by lateral intrusions, both along and across the foliation. The rocks of the Chungthang Formation are separated from the rocks of the Darjeeling Gneiss by a prominent structural feature known as, Ramam Chu Thrust (Raiana, et al., 1965). Localized faulting has been deciphered at several locations on the basis of slicken sides and fault breccias.

3. Methodology:

In this section, procedure of pre-grouting technique, including probe drilling have been discussed in detail. Also face logging with 3D geological mapping of RD 220 m to RD 260 m in the U/S HRT was carried out and procedures have been discussed in brief.

3.1 Face logging and 3D geological mapping:

In the present work, face logging of and 3D geological mapping of the HRT U/S, Adit II alignment from RD 220 m to 260 m were carried out at 1:100 scale, as depicted in Fig. 2. The principal lithological units have been identified and structural breaks were traced. The rock type of Adit -II HRT U/S from RD 220 m to 260 m is composed of calc-silicate rock with two sets of joints. Water seepage is very high in some areas (RD 225, 240, and 241) which affect the strength of the rock, as can be observed in Fig. 2. The rock class falls in Class IV due to flowing water conditions and very thinly spaced (<6cm) foliation. The geotechnical detail of the 3D geological map is depicted in Table.1.

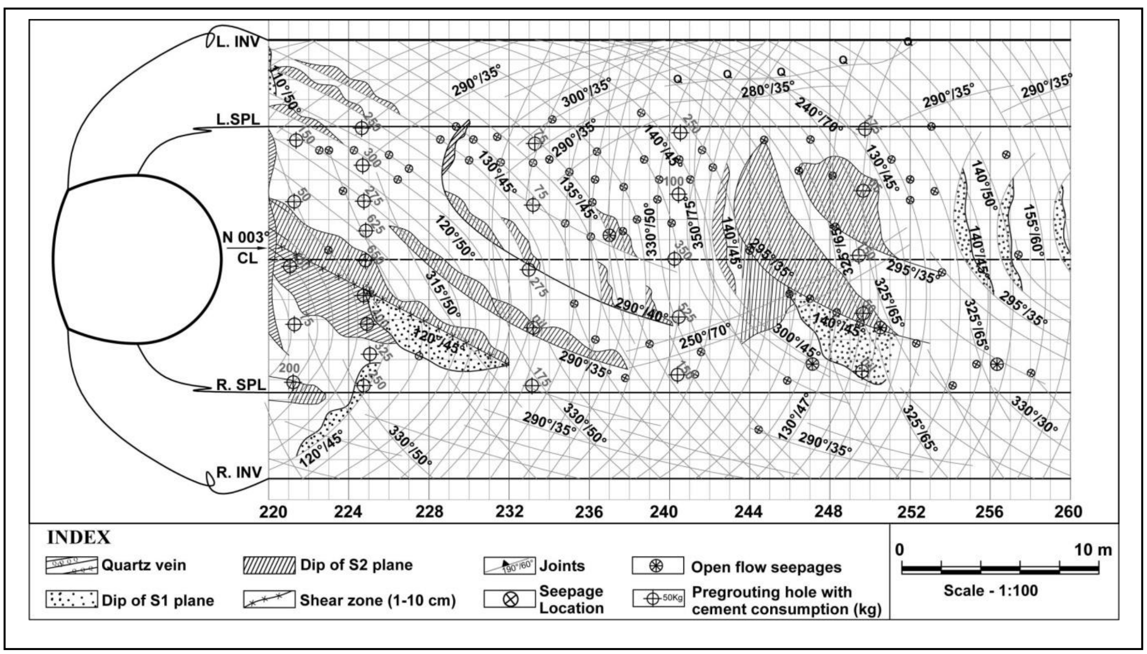


Figure 2 3D geological map of RD 220-260 of the U/S HRT, Adit II, Teesta III HEP.

Table 1
Geotechnical details of the 3D geological map of the U/S HRT.

Joint Plane		Parameters	Details
DISCONTINUITY	**S1**	RD (m)	220-260
		Joint Orientation Range	100-150/45-60
		Persistence (m)	10-20 m
		Spacing (cm)	< 6 cm
		Filling/aperture (mm)	0.1-1 mm/1-5 mm
		Roughness	RP
	S2	Joint Orientation Range	290-340/30-50
		Persistence (m)	3-10
		Spacing (cm)	20-60cm
		Filling/aperture (mm)	< 0.1 mm/1-5 mm
		Roughness	RP
	S3	Av. Orientation	240/70
		Persistence (m)	3-10
		Spacing (cm)	0.6-2 m
		Filling/aperture (mm)	< 0.1 mm/1-5 mm
		Roughness	RP
Rock Mass Description		UCS (MPa)	50-100 MPa
		JV (RQD)	50/17-19
		SRF	2.5
		Rock class*****/RMR/Q **(*as per Q System)**	IV/35-38/3.5
		Supports	Pre-grouting + rock bolts

3.2 Probe hole drilling:

Based on the rockmass classification (Barton et al., 1974) of the face, 3 or more probe holes were drilled before starting the pregrouting at the tunnel face and then pre-grouting hole was decided after finding the rock condition ahead of the tunnel face. Probe holes drilling (percussive method) were carried out using jumbo driller. Probe hole is an easy and safe way to acquire information about the rock conditions ahead of the face. The safety increases proportionally with the number of holes, within a practical range of 5–10 holes. In areas with a high risk, such as encountering water zone, the minimum number of probe holes should be drilled. The length of the holes depends on several factors such as, drilling equipment, shift sequences, round length, etc. In general, a length of about 15 m is kept with an overlap of 4-5 m. If one or more probe holes encounters water (more than accepted), then decisions on pregrouting need to be taken soon. Recorded features are water leakage through the drill holes measured at certain intervals, drilling parameters like penetration rate, quantity of water return, colour of water return and torque, hammering pressure and water pressure given by Atlas Copco Boomer. For the sake of brevity, a schematic illustration of the probe drilling layout with procedure in the HRT at RD 224.7 m from Adit-II is depicted in Fig. 3 (a) and (b).

Table 2
Details of the probe hole

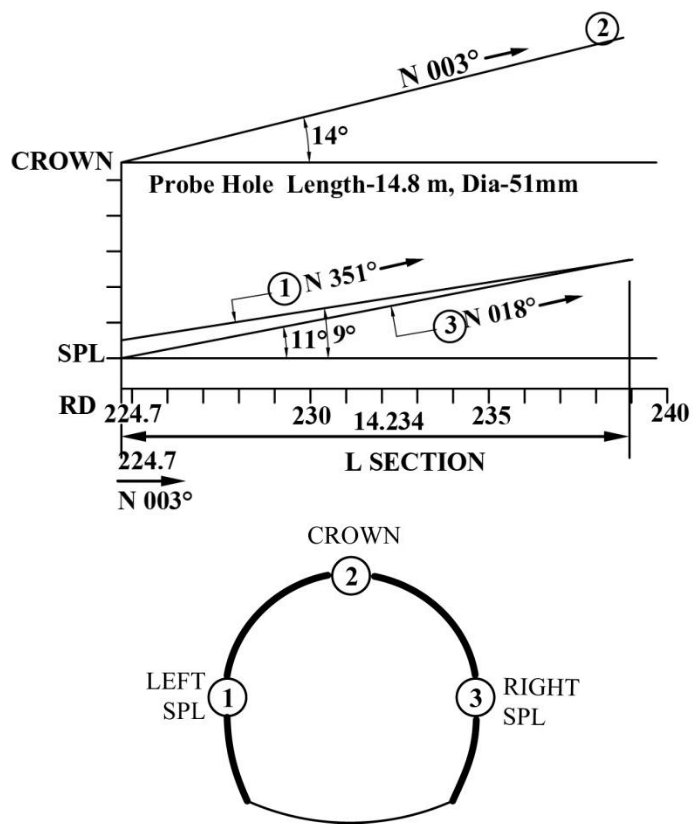
Hole No.	Section Length (m)	Penetration Rate/meter (sec.)	Time of Penetration (sec.)	Remarks
1	0 – 7.4	30.13	223	Between RD 224.7 m to RD 239.5 m, presence of thinly foliated Calc-Silicate rockmass falls in class IV. Due to heavy water seepage, water returns from pre-grout hole in gray to light brown color after removal of drill rod. Hole inclination is 9° – 14° in upward direction. Hammering pressure of 100 bar between 0 – 12 m and in remaining 200 bar with 10 bar of water pressure.
	7.4 – 11.1	29.46	109	
	11.1 – 14.8	32.02	117	
2	0 – 7.4	33.24	246	
	7.4 – 11.1	35.40	131	
	11.1 – 14.8	34.32	127	
3	0 – 7.4	32.56	241	
	7.4 – 11.1	33.51	124	
	11.1 – 14.8	33.10	121	

3.3 Pre-grouting:

Pre-grouting is a process of injecting slurry of cement along with resins or other admixture under pressure (Singh and Goel, 1999) into a rock formation through a borehole to mend fissures and cracks. In most of the cases the purpose of the pre-grouting is to strengthen the rockmass for easier and safer excavation and to stop water ingress (Hansen et al., 2003) into the tunnel.

The excavation of 14.6 km of HRT for Teesta Stage-III is at present under construction through 5 adits. In the present work, pre-grouting with high pressure was being carried out in the HRT of adit II, between RD 220 m to RD 260 m, for safe execution of project after every 10 m of advancement. Pre-grouting was started in May-2009, with 9 to 15 m long and 51 mm dia hole. The pre-grout material used herein was cement admixed with plasticizers and micro-silica. Initially, grouting was started with zero return with concentration of micro fine cement, 8% micro silica, with specified proportion of plasticizer and dispenser, keeping water to cement ratio of 3:1. Depending upon the intake i.e., greater than 20 liters/min. the concentration of the grout mix was decided and increased till the intake was reduces below 4 lit/min. If in two consecutive holes, the intake of cement was less than 50 kg/hole, then one more hole was drilled in the middle and grouting procedure was followed. Also if two successive holes have intake more than 500 kg/ hole, then grouting at these holes was stopped and again one hole was drilled in the middle and same procedure was followed. In case of low intake of grout, pressure at which grouting was done may be increased up to 50kg/cm^2. Grouting was restricted to a maximum intake of 500 kg cement/hole or if the pressure was above 20 kg/cm². The excavation of tunnel then carried out for an approximate length of about 10 m, leaving about 5 m of pre-grouted section.

In general, the pre-grouting was repeated after every 10 m of excavation, but depending upon the circumstances (heavy water ingress, loose rock/soil ahead of face and results of probe holes) at the face, a second layer of pregrouting was conducted at 5 m interval until the encounter of the good stratum. Also pre-grouting was conducted in the holes on both the inverts. At the end, middle holes (between invert and SPL) of both inverts and the hole at the inverts were grouted. The result of pre-grouting has been presented and discussed in the following section.



(a) (b)

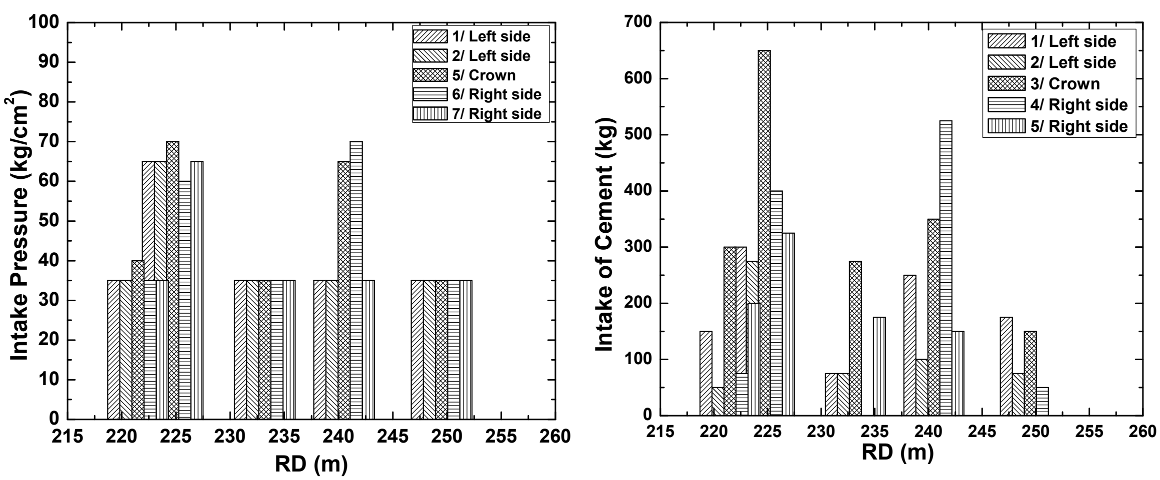
Figure 3 (a) Orientation of probe holes and (b) a schematic layout of probe holes

4. Results and discussion:

This paper presents a detail of the study area, its geology and structure along with a procedure of pre-grouting techniques, which facilitates the tunneling process by strengthening the physico-mechanical properties of the rockmass and reducing the water ingress in the tunnel. The three dimensional map of the study area (Fig. 2) shows that between RD 220 m to RD 260 m, the rocks are highly fractured, foliated with three sets of joints as S1 (foliation), S2 and S3 (Table 1). It can be observed from the Fig. 2, that a shear zone exists between the RD 220 m to RD 232 m starting from the crown to right SPL, having a trend of 120°/45°. Due to presence of this shear zone, planar failure was occurred, which leads to extra over break in the tunnel. With the help of probe hole drilling, the further trend of this shear zone was identified and hence to counter further over break, pre-grouting was started. It can be observed that the over-break was reduced after pre-grouting operation. At RD 224.7 m, 9 pre-grout holes of 51 mm dia were drilled at an angle of about 8°-15° in upward direction with length of 14.8 m. It can be noted here that the intake of cement in these grout holes were more than 500 kg, which is mainly due to the presence of open joints of 1-5 mm apertures.

It has also been observed that during excavation of the upstream end face of HRT through Adit-II, heavy water ingress i.e. more than 425 liter/min at RD 224.7 m was encountered and due to this progress in the advancement of HRT was reduced. To minimize and divert the water ingress form the face, grouting was done. From left side of the tunnel, water ingress was in flowing condition. To reduce the flowing discharge of water from the face, the grout pressure was increased to 70 kg/cm^2. As suggested by (Gustafson and Stille, 2005) the analysis of joints characteristics indicates that grout holes at RD 224.7 m and 240 m, near to the crown and towards left SPL requires some part of micro-cements or other grouting agents to increase penetrability and reduce water ingress. It has been observed that after completion of pregrouting, the heavy water ingress (as jet pressure) at RD 224.7 m in the HRT was reduced to 3-4 liter/min, which is nominal.

For the sake of brevity, the result of total intake pressure and intake of grout plotted against RD for each hole at crown and left and right SPL are depicted in Figure 4 and Figure 5 respectively.



(a) (b)

Figure 4 (a) The bar charts showing the total intake grout pressure (b) intake of cement in each hole between RD 220-260.

It can be observed from the bar diagram that the intake of cement grout was more at the crown as compared to the SPL. It can also be observed that rock type at the left SPL was good and hence the intake of cement was less than the right SPL. The average intake grout pressure has been 35 kg/cm^2 throughout the study area. At RD 240 m, the intake pressure at the crown was increased to 70 kg/cm^2 and intake of cement was about 350 kg. This shows that at RD 240 m, the joints near the crown and towards the right SPL side were open and hence intake of cement is more.

5. Conclusions:

During the construction of HRT (Adit-II) of Teesta HEP, high pressure pre-grouting was carried out inside the HRT for the safe execution of tunneling. Based on observations, it has been found that there are several advantages of high pressure pre-grouting, which are summarized below:

1) Application of efficient fine cement-micro silica with specified proportion of plasticizer-dispenser grout helps in quick sealing of fractured rockmass.
2) Grout hole was used as a probe hole, before the advancement of the tunnel to know the rockmass condition and water seepage ahead of the face.
3) Pre-grouting technique has helped to control the heavy water ingress in the tunnel. It reduces the execution time while driving tunnels through water-bearing fractured zones.
4) Pre-grouting also helped in the reduction of overall excavation tunneling costs. Due to application of this technique, the project operation was ahead of schedule.
5) It has been observed that pre-grouting increases the stand-up time and enhances the safety of the workers.

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