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GROUND WATER CONTROL BY GROUTING IN TUNNELS FOR A PUMPED STORAGE SCHEME

by

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ABSTRACT

The paper presents ground water problems encountered during the driving of tunnels for a pumped storage scheme in a developing country. The storage scheme is briefly outlined together with the geology and hydrogeology of the strata. The method of ground water control was based on the grouting technique incorporating cementacious and bentonite grouts. The scheme of grouting is described together with the ground water monitoring programme. The results indicate that the zone of highly permeable rock can be successfully grouted for ground water control by using the cementation technique.

INTRODUCTION

In recent years, a large number of water and hydro-electric resource development schemes have been planned in countries situated in an arid climate. In most cases these schemes are of great national importance and are carried out with utmost urgency and scant financial and technical resources. Consequently, in many cases, it is not always possible to carry out a detailed field investigation at the site of the proposed tunnel, especially when the tunnels are very long. Therefore, it is not surprising to encounter unexpected strata control and ground water inflow situations during tunnelling. The paper describes the ground water inflow problems encountered during the driving of access and water discharge tunnels associated with a pumped storage scheme.

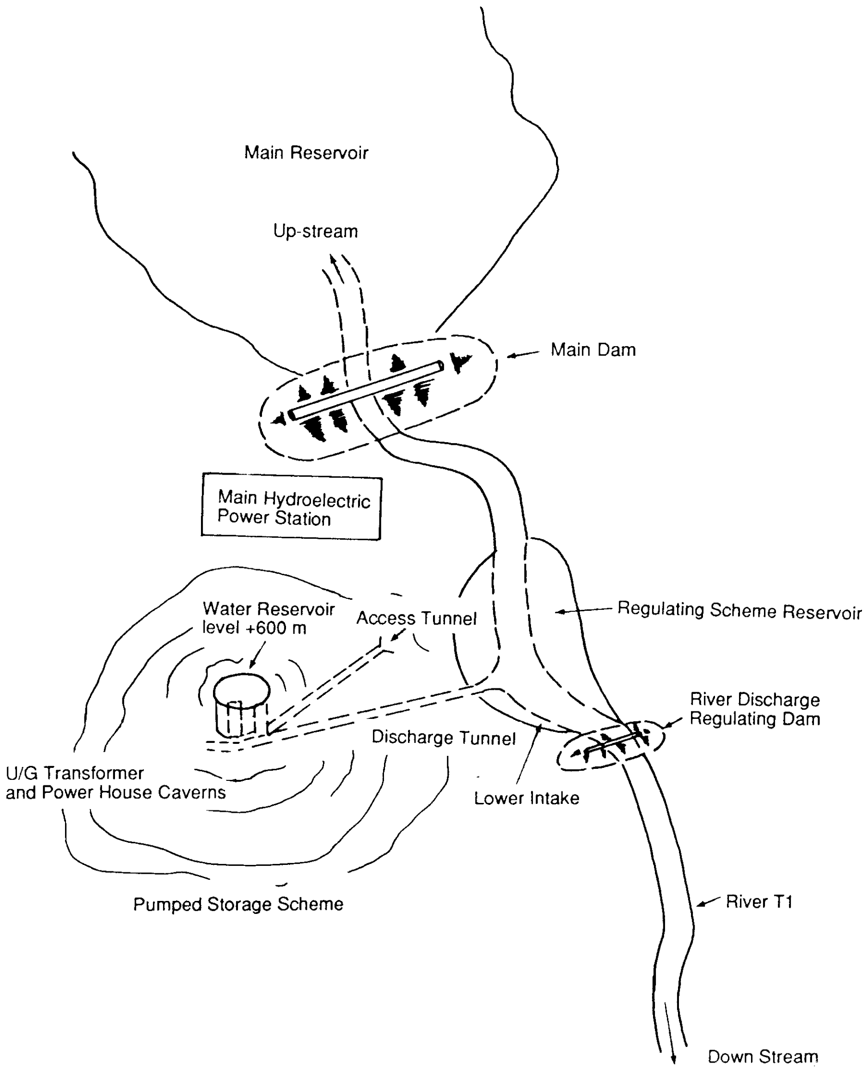


Figure 1. General Layout of the River Development and Pumped Storage Scheme.

PROJECT DESCRIPTION AND ENVIRONMENT

The purpose of this project was to develop water and hydro-electric power resources on river T1 by constructing a dam and associated pumped storage scheme to affect flood control, irrigation and to generate electricity. The dam was to be constructed on the river T1 which originated on snowy mountains and had a mean discharge of 1400 cu m/s. The drainage area of the river was 111400 sq km and yearly sediment yield was 62 M t/y. The climatic conditions of the site can be described as subtropical steppe with the temperature in summer as high as 50°C. Figure 1 shows the general layout of the river development complex together with the location of the dam and dam reservoir. A regulating scheme 2 km down the river stream from the dam formed a small reservoir which controlled the discharge of water in the river. The pump storage scheme was devised to utilise electricity during off peak periods by pumping water from the regulating reservoir intake to a water storage reservoir. The water reservoir was located on the top of the hill at a level some 600m above sea level and had a capacity of 1.6M m^3 . In the peak power demand period the water from the resevoir was transmitted through the shaft to the underground cavern at 240m level, housing turbo-generators to produce electricity. The outlet water from the generator was discharged through a water diversion tunnel to the intake reservoir. An access tunnel was driven to provide a means of ingress to the underground power station. Figure 2 shows the schematic presentation of the project.

The main constituents of the pumped storage schemes were as follows:-

- A water reservoir at the top of the hill at an elevation of 600m above the sea level.
- Communication and pressure shafts between the reservoir and the caverns.
- Caverns to house turbo-generators , transformers and pumps.
- Surge chamber to compensate fluctuating water pressures.
- Access Tunnel for inspection, supervision , supplies , the length being 1800m.
- Water transmission tunnel for the discharge of water to the river after being used for electricity generation. The total length of water diversion tunnel was 2800m.
- Lower intake to facilitate recharge of the water reservoir at the top of the hill.

This paper is concerned with the water problems associated with the drivage of access and water discharge tunnels.

ENGINEERING GEOLOGICAL ENVIRONMENT

Figure 3 shows the general geology of the strata at the lower intake. The overburden is of variable thickness up to 25-35 m comprising boulders and cobbles. These are underlain by gradually finer sediments of gravel, sand, silt and clay respectively. The permeability of the sand bed is in the order of 10^{-5} m/s while that of the unsorted cobbles bed was 10^{-6}. However, sand and gravel beds were highly permeable as indicated by the loss of water during permeability testing. The bed rock comprised variable thicknesses of limestone underlaid by calcareous marl with medium to high strength. This was underlain by medium to strong marl and soft to medium clayey marl respectively. Underlying clayey marl were gypsum and anhydrite beds which were at places highly gypsified. The water

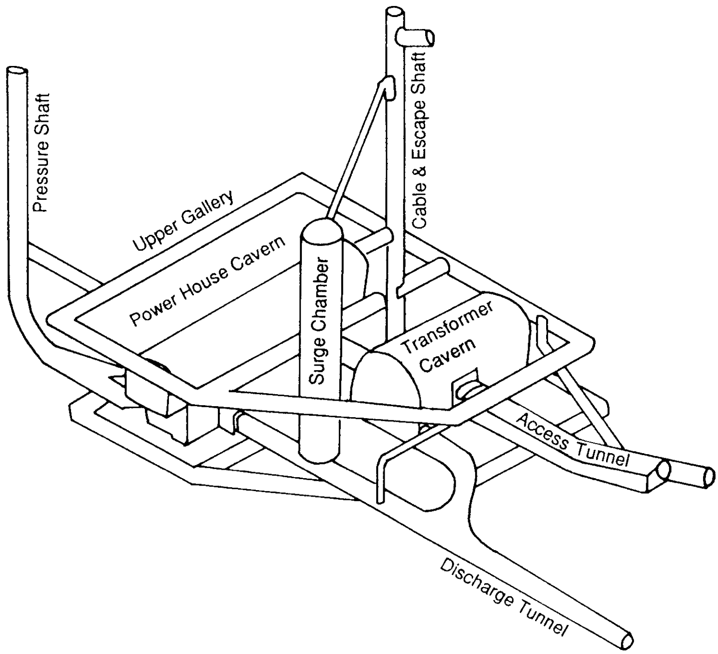


Figure 2. Schematic Presentation of the Project.

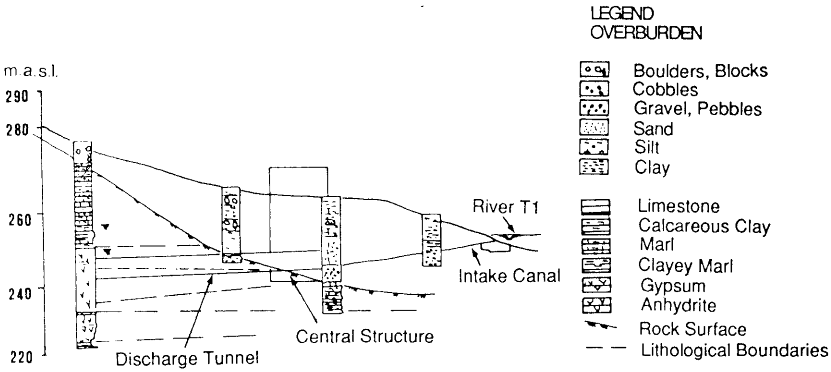


Figure 3. General Geology of the Area at the Lower Intake.

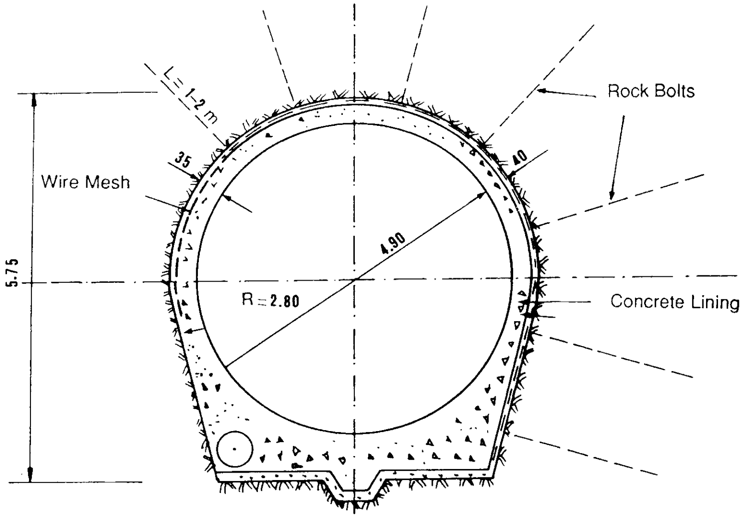
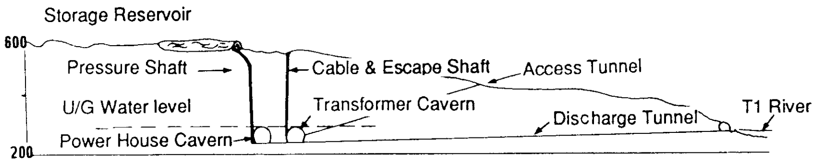


Figure 4. Designed, Finished Profile of Tailrace Tunnel.



Stationing (m) 2700	1800	1400	150	0
Tunnel Elevation (m)	205 217	222		240
Ground Water Elevation (m)	Maleh Aquifer Min 295 Max 330	Aquitard	T1 Aquifer Min 255	

Start Steel Lining ←|

Figure 5. Longitudinal Section of the Pumped Storage Scheme.

diversion tunnel was mainly driven in this horizon. The bedrock near the outcrop was highly weathered, thus displaying very poor rockmass strength characteristics. However, the modulus of deformability of the lower marl series from 150 to 1400 m chainage was between 500-2000 MPa. From the chainage 1400-1800m the tunnel intercepts a partly imperveous zone where the rock mass quality improved. The modulus of deformation of rock mass was in the range of 2500-5000 MPa. Figure 4 shows the designed finished profile of the water discharge tunnel. The new Austrian Tunnelling method was used utilising rockbolts, wire mesh and shotcrete as temporary support. The final support was in-situ cast concrete lining.

HYDROGEOLOGY

Figure 5 indicates the general layout of the access and water diversion tunnels in relation to the hydrogeology. The geology and hydrogeology of the rock mass at the site of these tunnels are highly variable. The first 150m of the tunnel is excavated through gravel and pebbles beds and horizontally dipping lower marl series. The rock mass in this sequence is characterised by weathered medium to hard marl and clayey marl which is highly permeable. In this zone marl is undergoing active karstification. Tunnelling at this zone did not present any problem.

T1 Aquifer :- This aquifer is intersected by the tunnel in the zone between 150-1400m. It is essentially an unconfined aquifer comprising horizontally bedded lower marl series. It is chararcterized by low permeability and water table varying between 251.6 -265.5m above the sea level. Mean ground water table is 255m. Inspection of the rockmass surrounding the tunnel excavation has shown three wet zones along the water tunnel.

- A zone between 710-745m consisting of Gypsum/Marl, brecciated Gypsum and laminated Gypsum/Anhydrite. This zone shows slight drippers emanating from the crown of the tunnel.
- A clayey marl at the chainage 1225-1390m with the intercalcations of up to 200mm thick bands.
- A dolomite/limestone zone at the chainage 1574m incorporating a perveous zone 1.2 m thick consisting of banded Gypsum/anhydrite and gypsum/marl bed. This bed is highly inclined towards the south and outcrops 120m above. This bed provides a major recharge area at the surface to the dolomite/ limestone layers.

Aquitard Zone: The rock mass between chainage 1400-1800m is the highly dipping (near vertical) lower marl series with some identifiable karstifications. The beds have very low permeability and can be considered as aquitard.

M1 Aquifer: M1 aquifer occurs between the chainage 1800-2000m along the Tunnel. The rock mass is limestone and dolomite which is characterised by high permeability. The water table in this unconfined aquifer fluctuates between 245-330m above the sea level.

GROUND WATER PROBLEMS

Ground water seepage problems occurred in the access tunnel when it intercepted the ground water table at the chainage 500m which corresponded to about 1800m off the discharge tunnel . This problem was accentuated in the presence of poor quality rock ahead of the tunnelling face. The major tunnelling problems could be assigned to the lack of preparation of pumping arrangements and associated operational problems. Further, the lack of forward knowledge of ground conditions ahead of the tunnel face did not inspire the confidence of the tunnelling personnel. Further ground water problems at the water discharge tunnel were apprehended when intersecting M1 aquifer about 25m below the ground water table. Actually severe ground water seepage did occur when the water diversion tunnel reached 1942 m chainage mark where the ground water inflow rate exceeded 450 l/s. The trials to select the best grouting procedure and final grouting technique for ground water control is described in subsequent sections.

AVAILABLE GROUTING TECHNIQUES AND SELECTION PROCESS

The most common technique of protecting a tunnel against inflow of water is to construct an umbrella of grouted rock mass surrounding the tunnel. In the past several techniques have been used but most common of them are as follows:-

- o Advancing stage method
- o Retreating stage method
- o Advancing zone method
- o Multi-packer method
- o Multi-packer method utilizing sleeve valved pipe.

In the advancing stage method, the grouting boreholes are drilled in steps of 5m in which permeability tests are performed using the pumping in technique. The borehole is then pressure grouted and the grout is allowed to set over a period of 8 hours. The grouting borehole is then deepened in the stages of 5m and then the pressure testing and grouting operations were resumed. This technique is particularly valid in the presence of incompetent ground where it is impossible to install a packer in the bore hole due to the danger of borehole collapse.

Retreating Stage Method:- The retreating stage method is applicable in a relatively stable ground where it is possible to drill a long borehole in one operation, without lining. The grouting is done in retreating steps of three metres using a packer at the outbye end of each stages. This method has been successfully used in homogeneous rock masses.

Advancing Zone Method:- The advancing zone method of grouting is used for improving the rock mass quality and to render it impermeable in the presence of some abnormal conditions. This method of ground treatment is applicable to treat cavities, open cracks and fissures, large flow of artesian water and caving of borehole walls. As soon as an anomaly is detected, the borehole is washed and a packer is installed at the outby end of the anomaly for grouting. The packer is then installed at the collar of the borehole and the rest of the borehole grouted. After permitting the grout to cure for a period of eight hours the drilling is resumed up to a desired depth or until another anomaly is encountered. Thus, in this method, the grouted zones advances while within the zone the grouting is carried out in retreating stages.

Multipacker Method:- This method is designed on the basis of laboratory and field trials to improve water pressure tests and subsequent grouting operations in the weathered or weak rock mass susceptible to partial or complete collapse of borehole. Attributes of multipacker systems are as follows:-

- o Borehole walls are least disturbed as all the packers are installed in one operation in contrast to the repeated installation of single a packer.

- o It permits permeability testing of various test cavities along the predetermined length of the borehole in the undisturbed rock mass.

- o Permits systematic grouting of rockmass by changing grout type, grout volumes and pressures to suit variable ground conditions.

MULTIPACKER SLEEVED PIPE SYSTEM

This technique is a variant of multipacker system and consists essentially of installation inside the borehole of a plastic pipe which is fitted at regular intervals with rubber grouting sleeves. The permeability testing and grouting operations are carried out by means of simple or double removable packers set inside the grouting pipe and lined with the required length of rubber sleeve. The grouting operations are carried our either by advancing or retreating method to suit the ground conditions. The packer bags are fastened to the grouting pipe at regular intervals to seal off the borehole between two predetermined positions to form a test cavity in which grouting is carried out. The sealing action of the packer is achieved by injecting grout in the packer bags and expanding them against the borehole walls. Figure 6 shows the sequence of operation of the multipacker sleeved pipe grouting system.

The packer assembly essentially consists of the following components:-

- o Sleeved pipe of fixed lengths and 37 to 50 mm in diameter are jointed together by steel sockets to withstand sufficient grouting pressures with adequate factor of safety.

- o Grouting valve comprising rubber sleeve, 4mm in thickness, fitted with the plastic pipe at the perforated section. The sleeve expands under pressure and allows grouting operations and closes to the original position when adequate refusal pressure develops. The sleeve can withstand the pressures up to 7 MPa.

- o The packer bag comprises polypropelene continuous fibre, 120mm in diameter and 400- 500mm in length fastened to the grooved sockets of the pipe by means of steel strapping. The fabric of the bag ruptures at pressure of 2.5 MPa and the injection grout will break through in radial directions to penetrate into the rock mass.

- o The sleeved pipe had small holes at 1.50 m intervals along its length which were covered with the rubber sleeve which opened under pressure, thus formulating one way valve. In order to

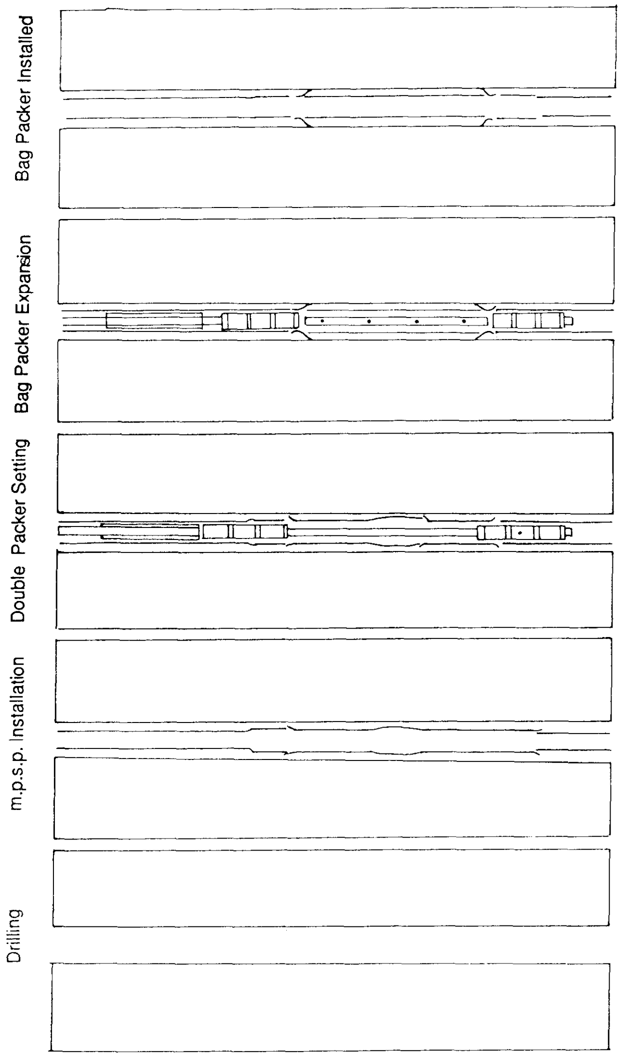


Figure 6. M.P.S.P. System Sequence of Operations.

inject grout through the sleeved pipe , a double packer mounted on the sleeved pipe was inserted in position. Small diameter injection pipes were connected to the packer assembly, in order to inflate the packers with grout, thus forming a closed injection chamber incorporating an outlet valve. This facilitated the injection of grout through the sleeved pipe.

This method of grouting presents many advantages, but the most important feature is its ability to inject repeatedly various grouts of reducing viscosities. Coarser grouts can seal larger discontinuties and fissures while more expensive, low viscosity grout can be used to infill finer cracks. Thus grouting in a variety of ground conditions can be carried out economically. Further, this method permits grouting to be carried out independently of drilling.

Method of Grouting Using Multipacker Sleeved Pipe Technique:-

Grouting boreholes are drilled by using a rotary drilling rig., the outer diameter of the drilling bit was 116 mm. The speed of the drill could be continuously changed between o to 660 r.p.m. and torque between 0 to 420 kgm. Normally water was used as a flushing medium but occasionally bentonite or other gels could be used to meet a variety of ground conditions. Once a borehole is drilled , it is lined with a casing 105 mm OD and 80 mm ID. A multipacker and sleeved valve assembly is inserted in the borehole, and the casing is withdrawn. The Multipackers are then inflated in position by injecting grout and are allowed to set in position for a period of 48 hours.

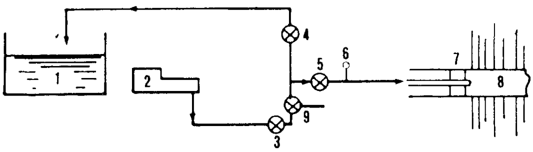
The grouting equipment comprises of primary mixing plants, supply and return pipe circuits, control valves and monitoring instrumentation. The equipment is sited as near the grouting holes as possible. The storage of water, cement, bentonite and clay should be available nearby. A high speed colloidal mixer fitted with an automatic batcher to dispense basic cement and bentonite was available. The mix was collected in a suitable agitator tank and delivered by alternating pumps to the grouting circuit. A series of tanks and agitators are required to mix various slurries. The layout of the grouting circuit is shown in Figure 7.

GROUTING TESTS AND SELECTION PROCESS

Water problems during the drivage of access tunnel suggested that similar water problems were likely to occur in the water discharge tunnel. The main problems expected were inflow of excessive quantities of water and related poor ground conditions. It was therefore decided to carry out grouting trials in the water dicharge tunnel to select the most appropriate grouting technique. The purpose of the grouting trials were as follows :-

- o Gain experience of drillling grout holes.
- o Installation of Multipacker
- o Selection of various grouts
- o Grouting of Check holes and monitoring

The depth of the excavation for the grouting trial was selected to be 15m and the length of the corresponding grouting holes were 20m. The drilling and grouting was decided to be done in two stages, the first



1- Batcher
2- Pump
3- Supply Line Valve
4- Return Line Valve
5- Packer Valve
6- Pressure Gauge
7- Packer Sealing
8- Stage in Grout
9- Discharge Valve

Figure 7. Layout of the Grouting Circuit

stage being 0-9m and the second stage 9 to 20m. The grouting trials arrangements are shown in Figure 8. The diameter of the grouting boreholes was 40mm and if sand was required the boreholes to be redrilled to 76mm and packers placed at 3m. The compositions of various grouting mixes used are given in Table 1 and the pumping pressure requirements in the Table 2.

Table 1. Grouting Mix for Trials

Mix no.	Cement Part	Water Part	Bentonite %	Cement / Stage Kg	Sand Part
1	1	1	2	2000	-
2	1.6	1	2	8000	-
3	1.5	1	2		1

The check holes were drilled to monitor grouting performance and proportion was made to be grouted by mix 2 if water was encountered in quantities in excess of 5 l/s.

Table 2. Pumping Pressure

Stage	Without sand MPa	with sand MPa
1	10.00	15.00
2	20.00	25.00

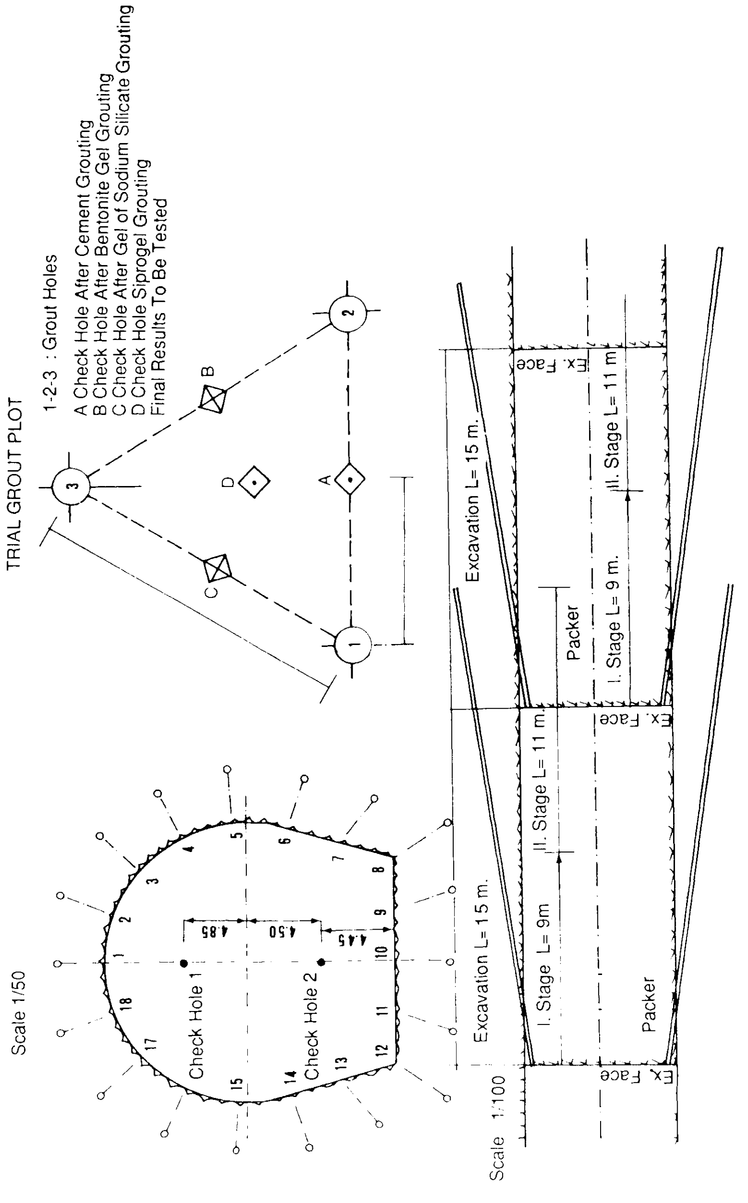


Figure 8. Arrangement for Grouting Trials.

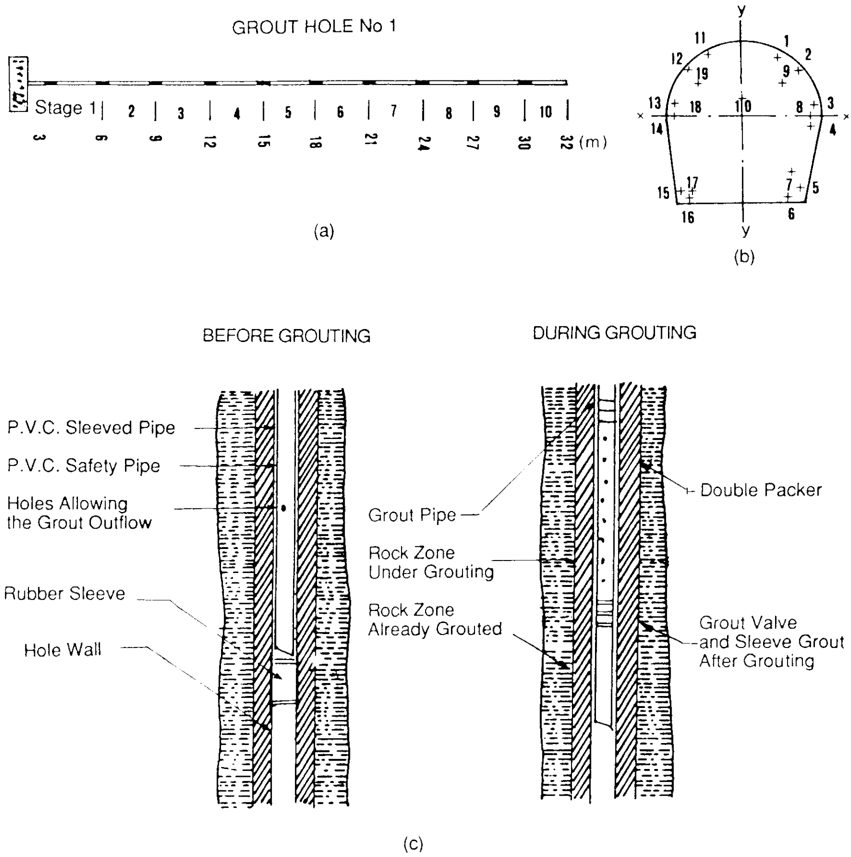


Figure 9. Layout of Grouting Arrangement (a) Details of Multiple Packers in the Boreholes (b) Borehole Layout (c) Details of Grouting.

PRESSURE GROUTING OF WATER DISCHARGE TUNNEL

The pressure grouting of the water discharge tunnel was started at the chainage 1947m. Figure 9 shows the layout of the two stage advance grouting at 1947 and 1972 chainage together with the details of seals. During both stages of grouting the length of each borehole was 32m and diameter 113mm. It can be seen that at each stage, 18 grouting boreholes were used. In each borehole a Multipacker incorporating 10 seals was installed. After allowing it to set for a period of 48 hours water preesure tests were conducted. If the borehole was discharging water under pressure, pumping out tests were performed and the pressure loss in the borehole for a constant rate of outflow was recorded. If a limited amount of water was available in the rock mass then the pumping-in test was the obvious choice. Thus the data regarding the pressure test in each of the test cavities were recorded and the permeability of rock mass surrounding the cavity calculated. The pressure grouting was carried out by the retreating stage method, monitoring the flow pattern at each stage.

The sequence of the borehole grouting started at the right hand floor level and proceed in the clockwise direction. It took about 2 days to drill 18 boreholes and from 2-3 hours to grout 10 sections of each borehole and upto 40 days to grout one complete stage of the tunnel. Figure 10 shows the water inflow quantity from the borehole number 1 for various stages of grouting. It can be seen that the inflow rate from the borehole was 42 l/s which reduced to 1 l/s after ten stages of grouting. After grouting all the boreholes the inflow from the tunnel was reduced from 250 l/s to less than 6 l/s. After each stage of grouting the residual water inflow was measured. It can be seen that each stage of grouting effectively reduced the water seepage.

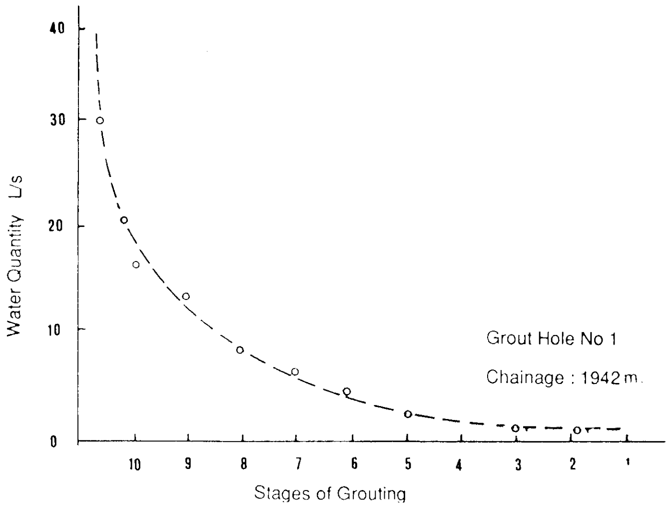


Figure 10. Typical Grouting Effect in Water Quantity Decrease.

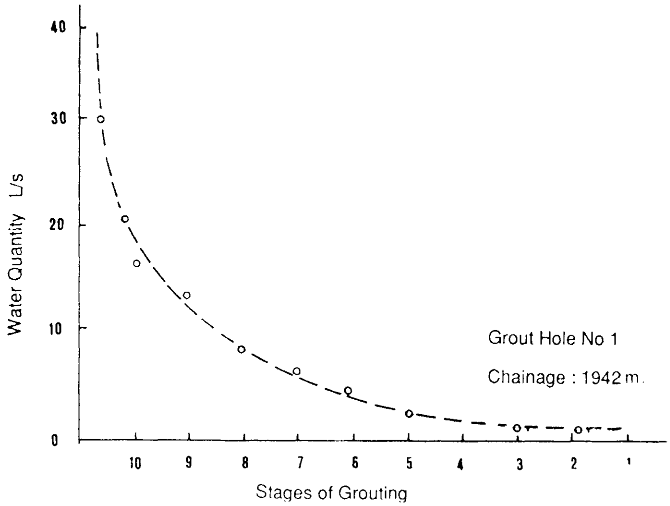


Figure 10. Typical Grouting Effect in Water Quantity Decrease.

CONCLUSIONS

One of the practical methods of ground water control during tunnelling was the use of cement or clayey grout. The appropriate method of ground water control is the use of grouting. One of the most effective ways of grout design is the assessment of ground characteristics and the evaluation of grout behaviour followed by the grouting trial. It has been shown that the multipacker technique of tunnel grouting can be adopted to control ground water inflow problems of a tunnel in water bearing and weak ground.

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