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Tunneling in Severe Groundwater Inflow Condition

지하수 과다유입 조건하에서의 터널굴착

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Abstract

For a hydro power plant project, the headrace tunnel having a finished diameter of 3.3 m was constructed in volcanic rocks with well-developed vertical joint and high groundwater table. The intake facility was located 20.3km upstream of the powerhouse and headrace tunnel of 20km in length and penstock of 440m in height connected the intake and the powerhouse. The typical caldera lake, Lake Toba set the geology at the site the caving of the ground caused tension cracks in the vertical direction to be developed and initial stresses at the ground to be released. High groundwater table(the maximum head of 20bar) in the area of well-connected vertical joints delayed the progress of tunnel excavation severely due to the excessive inflow of groundwater.
The excavation of tunnel was made using open-shield type TBM and mucking cars on the rail. High volume of water inflowraised the water level inside tunnel to 70cm, 17% of tunnel diameter (3.9m) and hindered the mucking of spoil under water. To improve the productivity, several adjustments such as modification of TBM and mucking cars and increase in the number of submersible pumps were made forthe excavation of severe water inflow zone. Since the ground condition encountered during excavation turned out to be much worse, it was decided to adopt PC segment lining instead of RC lining. Besides, depending on the conditions of the water inflow, rock mass condition and internal water pressure, one of the invert PC segment lining with in-situ RC lining, RC lining and steel lining was applied to meet the site specific condition. With the adoption of PC segment lining, modification of TBM and other improvement, the excavation of the tunnel under severe groundwater condition was successfully completed.

Keywords : Headrace tunnel, Volcanic area, Vertical joints, TBM, Water inflow, PC segmental lining

요 지

본 논문은 수직 절리가 잘 발달된 지하수위가 높은 화산암질 지반에서 직경 3.3m의 도수터널 굴착을 하는 수력발전소 건설공사 내용이다. 취수시설은 발전소로부터 20.3km 상류에 위치하고 있으며, 20km의 도수터널과 연결되어 있고 440m의 낙차고를 갖는 펜스탁이 발전소와 연결되어 있다. 현장의 지질 조건은 전형적인 칼데라 호수인 토바호에 의해 지반 침식과 수직방향의 인장균열이 발달하였으며 이로 인해 지반의 초기응력이 이완되었다. 높은 지하수위(최대 수두 200m)를 가진 잘 발달된 수직 절리를 터널이 관통하면서 막대한 양의 지하수가 터널내로 유입되었다.
터널 굴착은 개방형 쉴드 TBM과 버럭반출에는 철로와 기관차를 사용하였다. 터널 내로의 유입수가 터널 바닥면에서 70cm 높이에 다다르고 이는 터널 직경(3.9m)의 17%에 해당하였다. 생산성을 향상하기 위해서 TBM과 버럭반출 차량과 같은 몇 가지의 개선과 수중펌프를 증설하는 방안을 사용하였다. 굴착 중에 만난 지반 조건이 설계보다 상당히 불량하여 RC라이닝에서 지하수 유입, 암반조건, 수압 등에 따라 PC 세그먼트 라이닝 또는 PC 세그먼트 라이닝과 현장타설 RC 라이닝, RC 라이닝, 그리고 강재 라이닝이 적용되었다. 이 PC 세그먼트 라이닝의 도입과 TBM과 다른 장비의 개조 및 개선을 통해서 심각한 지하수 조건 하에서 터널 굴착 공사를 성공적으로 완료하였다.

주요어 : 도수터널, 화산지역, 수직 절리, TBM, 지하수 유입, PC 세그먼트 라이닝

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1. INTRODUCTION

The Renun Hydro Power Plant Project has been under construction since 1995 to generate and supply 82MW electricity to northern area of Sumatra, Indonesia. For the generation of power, the generation water is collected from the Intake Structure(located 20.3km upstream of Powerhouse) and tributaries of the Renun River, stored temporarily in the Regulating Pond(located 11.2km upstream of Powerhouse and having a storage capacity of 500,000m^3) and sent to Powerhouse for the generation, as shown in Figure 1. Concrete-lined tunnel of 20km in length and 3.3m in finished diameter was divided into two parts; 8.8km long Upstream Headrace Tunnel(UHT) and 11.2km long Downstream Headrace Tunnel(DHT). The overburden at UHT section is in general in the range of 30m, while that of DHT varies from 30m near Regulating Pond and 350m near Sta. 90. The 440m water head is to be delivered to the Powerhouse via the embedded steel penstock of 853m in length.

Since the project site was located in the dense jungle, the initial geotechnical investigation could not be performed in a full scale. Only the refraction seismic survey and a few drillings were completed. The tender design was carried out based on the insufficient geotechnical information and inappropriate construction method and equipment were adopted accordingly. This paper describes the importance of geotechnical condition at the site, construction difficulties due to insufficient geotechnical informationand design and construction changes to overcome these difficulties.

2. GEOTECHNICAL CONDITION AT THE SITE

2.1 Geology at the site from the tender design

The geology at the site was strongly influenced by the presence of Lake Toba, typical caldera lake. It was understood that, after the volcanic activity, the caving of the ground and forming of lake might have made very high and stiff cliff at the edge of the lake and caused the initial stresses at the ground near the cliff to be relieved. The initial stress relieved near the cliff resulted in the fractured zone of moderately spaced and open vertical joints. This unique feature ingeology at the site was not detected

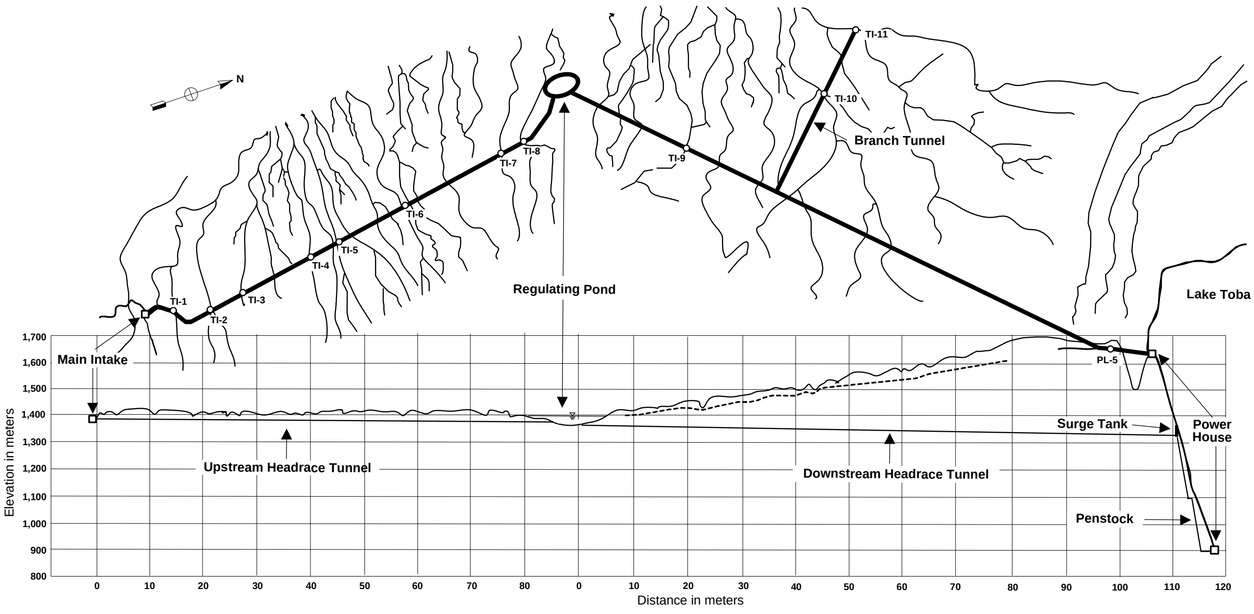


Figure 1. Plan View and Cross Section View of Renun Hydro Power Plant Project

during the tender design and hence the influence of this feature on tunnelling not incorporated in the tender design.

The bedrock at the project area was mainly composed of dacitic tuff of Toba eruptions. The bedrock condition along UHT was in general uniform; loose weak pumiceous tuff or sandy tuff overlay a consolidated and thick dacitic tuff and were eroded along the tributaries. The UHT route passedalmost homogeneous topographic area except some undulations which were formed by depressions along the tributarychannels. A total of 8 drillings(TI1 to TI8) were made along the UHT route at the top of river bank of each tributary intake site.

On 11.2km long DHT side, only two boreholes(TI9 and PL5) on the main tunnel line and two boreholes(TI10 and TI11) on the branch line were drilled. Therefore, major portion of geotechnical information at the site was collected from the observation on outcrops at the Penstock area and the results are summarized in Figure 2. As shown in this Figure, pumice tuff and sandy tuff seemed to be found from the ground surface all along the alignment, overlainby dacitic tuff and ignimbrite and andesite successively. According to the P-wave velocity(V_p) measured, pumice tuff and sandy tuff seemed to be weak, but dacitic tuff and ignimbrite hard.

Using boreholes(TI1 to TI11) drilled at all tributary intakes, Lugeon tests were performed at 5m interval in depth. The coefficient of permeability along UHT section ranged from the order of 10^{-2}cm/s to 10^{-5}cm/s. High permeability was measured at shallow depth, less than 20m in depth, while low permeability encountered at the tunnel level. The hydro- geological condition along DHT assumed to be better than that of UHT side, considering the higher overburden and higher wave velocity of the ground. The groundwater table monitored from two boreholes indicated that the groundwater would be encountered from the ground surface for the section near borehole TI9 and at the deeper depth for a section near the cliff(borehole PL5).

2.2 Geology encountered during construction

On July, 1997, the excavation of DHT started from the downstream side, Surge Tank area and the bedrock condition for 4.4km section of downstream side from the tunnel entrance was carefully monitored during TBM excavation and summarized in Figure 3. A total of 11 horizontal boreholes were also drilled inside the tunnel between Sta. 112 and Sta. 68. At theseboreholes,

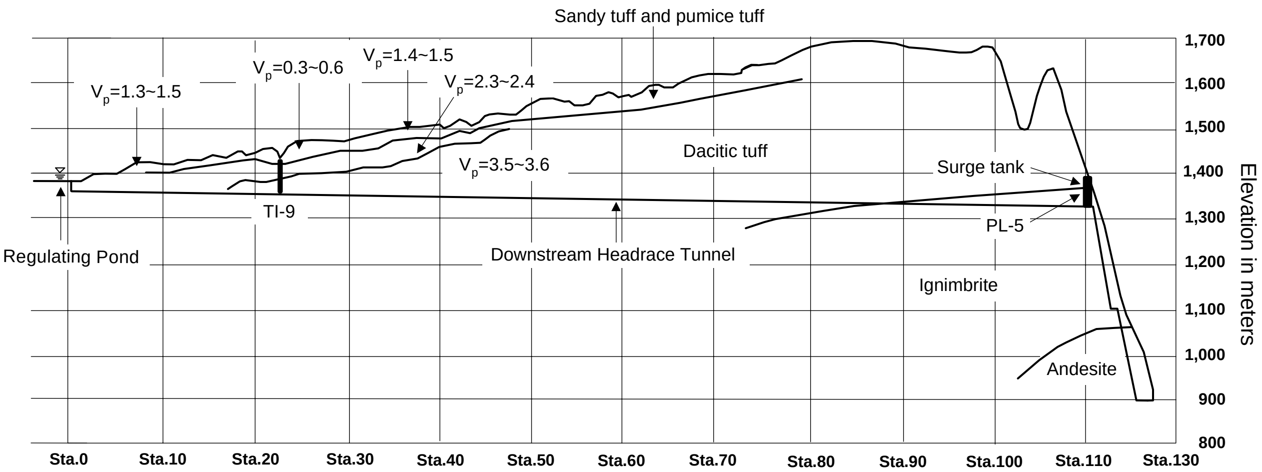


Figure 2. Geological Profile along Downstream Headrace Tunnel Obtained from Tender Design Stage

groundwater pressure measurement were carried out and uni-axial compression test was also performed on rock core samples recovered. The results of this investigation are summarized in Table 1.

As shown in Figure 3, rock formations along the tunnel alignment turned out to be much more complex than those obtained during the tender design stage. However, the mechanical behaviour of rock core sample tested was in general competent and in good agreement with the results of compression wave velocity measurement from the tender design, as can be seen in Table 1. The most important geological feature encountered from tunnel excavation was the presence of vertical fault zone at Sta. 76, found to be a hydro-geological barrier. The hydro-geological condition, 3.5 years after the excavation started, for a section between this fault zone and Surge Tank area may be classified as dry(see Figure 3). The dry condition, that is, the initially-low groundwater table, may have been resulted by the presence of well-developed vertical tension cracks. As shown in Table 1, Lugeon values obtained from horizontal boreholes, in which the influence of vertical joint could be easily measured, were much higher than the design value of 1, measured from vertical boreholes, indicating that bedrock would be much more permeable.

On March, 2002, the tunnel face reached at Sta. 56+00 and the groundwater table for the upstream side of the fault zone at that time was established from the measurement of water table inside vertical boreholes AD1 to AD6, as shown in Figure 3.

The excavation of tunnel from Surge Tank to fault zone took about 1.5 years and that from fault zone to Sta. 56 another 2 years. As shown in Figure 3, the influence of the excavation on the groundwater level is clearly shown in the lowering of groundwater level between the fault zone and Sta. 56 due to the presence of well-connected vertical fractures. The lowering of high groundwater table was made possible by draining high pressure groundwater to the tunnel excavated and resulted in the large inflow of water to the tunnel.

3. EXCAVATION AND SUPPORT PATTERN OF HEADRACE TUNNEL

Based on the fact that UHT side was rather homogeneous and impervious and the length of the tunnel was 20km, it was decided to adopt open-shield type TBM for the excavation.

Table 1. Results of Geotechnical Investigation Performed at Horizontal Boreholes inside Tunnel

Station	Type of Rock	Compr. Strength (MPa)	Young's Modulus (MPa)	Groundwater Pressure (bar)	Lugeon Value	Pcr (kgf/cm^2)	Rock Classification and Groundwater Table predicted at Design Stage
112+00	Ignimbrite	32.9	3200	0	41.5	3.0	Ignimbrite Compressive Strength >500 kgf/cm^2
109+50	Ignimbrite	26.6	3000	0	-	<1.0	
107+17	Dacitic Tuff	14.6	1020	0	6.0	6.2	
104+50	Dacitic Tuff	11.8	750	0	7.0	>12.0	
102+70	Dacitic Tuff	15.0	370	0	>50	3.2	Groundwater Table at -16 m at Sta.112+60 K <10^{-5}cm/sec (no groundwater information available for the rest area)
97+10	Dacitic Tuff	10.6	700	0	7.0	>12.0	
92+04	Pumice Tuff	-	-	1.5	-	-	
90+05	Sandy Tuff	10.6	1000	2.2	12.5	>10.0	
84+45	Ignimbrite	17.3	1920	1.9	>50	>4.5	
76+00	Ignimbrite	44.2	3200	6.6	33.0	>10.0	
68+45	Ignimbrite	32.6	2200	2.3	>50	>12.0	

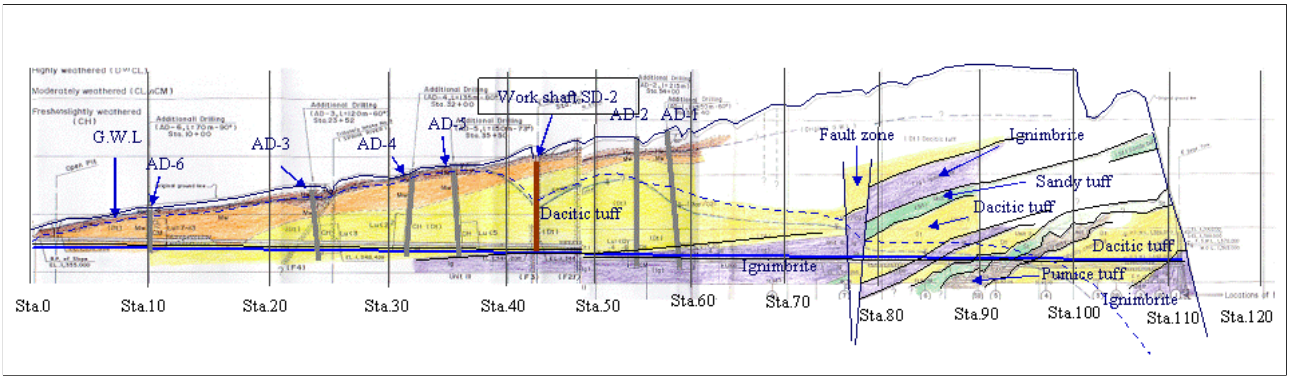


Figure 3. Geological Section of DHT Side Based on the Results of Additional Investigation and TBM Excavation (The dashed line represents GW table when tunnel face was at Sta.56+00)

Table 2. Support Patterns for Downstream Headrace Tunnel

Type	Supprt Pattern	
	Primary Support	Secondary Support
DA	No support	
DB	Rockbolt (L=2.5 m) 5@1.2m	
DC	Wire mesh + Shotcrete (t=70mm) + Rockbolt (L=2.5 m) 5@1.2m	Reinforced Concrete (t=30cm)
DD	Round Steel Rib 100x100 mm @1.2m	
DE	Horseshoe Steel Rib 100x100 mm @1.2m	

The mucking of excavated materials was designed to be made using conveyor belt and mucking cars on rail track. Five different primary support patterns were adopted for DHT(excavated diameter of 3.9m), as shown in Table 2. At the tender design stage, cast-in-situ RC lining of 30cm in thickness was adopted and backfill and consolidation grouting works applied after the installation of concrete lining.

4. DIFFICULTIES ENCOUNTERD DURING CONSTRUCTION

4.1 Excessive inflow of groundwater

At the initial phase of headrace tunnel excavation near the cliff, both rock and groundwater conditions were favorable to the excavation and the progress was rapid, as shown in Figure 4. However, as the tunnel face reached at Sta. 96, the bedrock changed from dacitic tuff to weak sandy tuff, but groundwater condition was still dry, resulting in the slower progress of excavation. As soon as the tunnel face passed through the fault zone at Sta. 76, excessive amount of groundwater poured into the tunnel from the area between crown and springline, as shown in Figure 5. Soon after the tunnel face passed the fault zone, the groundwater having the head of 200m spouted through open vertical joints and the amount of groundwater measured at the tunnel entrance near Surge Tank area was as high as 292liter/sec. As the tunnel face moved away from the fault zone and the excavated length of tunnel became longer, the water head decreased, but not the amount of inflow.

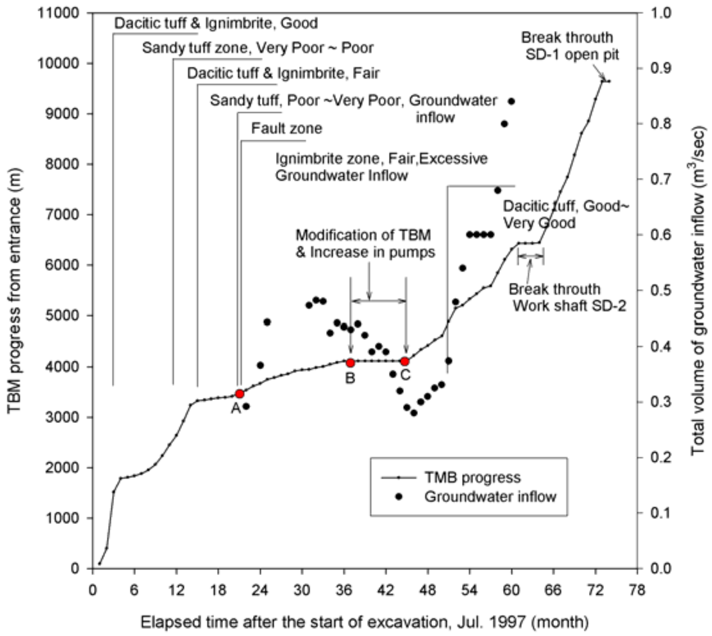
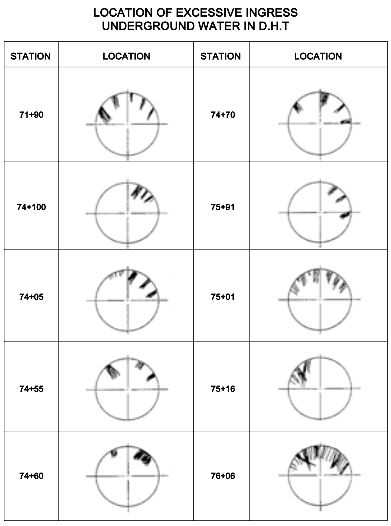


Figure 4. TBM Progress and Volume of Groundwater Inflow with Time



Sketch of Water inflow

STA. 76+06, 1999. 5

STA. 71+90. 2000. 3

Figure 5. Sketches and Photos Showing Excessive Groundwater Inflow

4.2 TBM progress

As shown in Figure 4, TBM progresssignificantly decreased as the inflow of groundwater became excessive after TBM passed through the fault zone at Sta. 76(point A). As the tunnel excavation progressed, groundwater discharge rapidly increased up to 482 liter/sec from 292 liter/sec and decreased slowly when the tunnelling work stopped for the modification of TBM and drainage system(point B). The excessive inflow of groundwater slowed down the progress of TBM due to the slime. The excavated rock fragments broke down to gravel size or smaller when mixed with water and became slime. This material could not be easily removed by the fork-type scraper and hence piled up at the back of TBM head to an approximately 20 to 30cm thickness at the invert. The crew had to remove this material manually with shovels to allow the power unit trailer to pass and permit the installation of the railway tracks on the tunnel invert. Removing the slimetook on average 23 % of the available working time during the period May to October 2001. Due to the excessive water inflow and the material piled up at the invert, the water level inside the tunnel rose to the maximum of 70 cm. Due to the water and dirt conditions, the locomotive and mucking cars derailed frequently. As the rolling locomotive ran on debris-laden tracks, it resulted in high wear of wheels, breaks and bearings. The adverse operating conditions for TBM, power unit trailer and the locomotive in this tunnel led to a high percentage of down time caused by the equipment failure and net boring and re-gripping time on average 10% of the available working time.

5. IMPROVEMET OF EXCAVATION METHOD

5.1 Modification of TBM

Since the initially-adopted fork-type scraper could not remove the slime efficiently, the unremoved slime travelled to the back of TBM head through the space between the rock surface at the invert and the bottom part of TBM head. To improve the efficiency in the removal of slime, the scraper was modified from the fork-type to bucket-type, as shown in Figure 6, and a slime-proof cover was installed at the space between TBM head and rock surface at the invert, as shown in Figure 7. The locomotive was modified for a better operation inside the tunnel by re-locating the engine upward, so that the engine would not be submerged. Besides, all the wheels were sealed and all rotational parts replaced with O-ring and oil seal, resulting in the longer life of these parts.

5.2 Increase in the number of discharge pumps

In order to make the excavation work as dry as possible, it was decided to employ additional pumps, considering the extra amount of groundwater to be discharged from the subsequent tunnel excavation. On April, 2000, electric resistivity survey was performed to investigate the groundwater regime along the 4 km section of tunnel from shaft SD 2 (Sta. 42+40) to downstream side and the results are presented in Figure 8. As can be seen from this Figure, the excessive inflow zone would be encountered at the section between Sta. 59 and Sta. 52. Using Goodman's equation with the measured coefficient of permeability from the area, it was predicted that the groundwater of371 liter/sec would be added from a tunnel

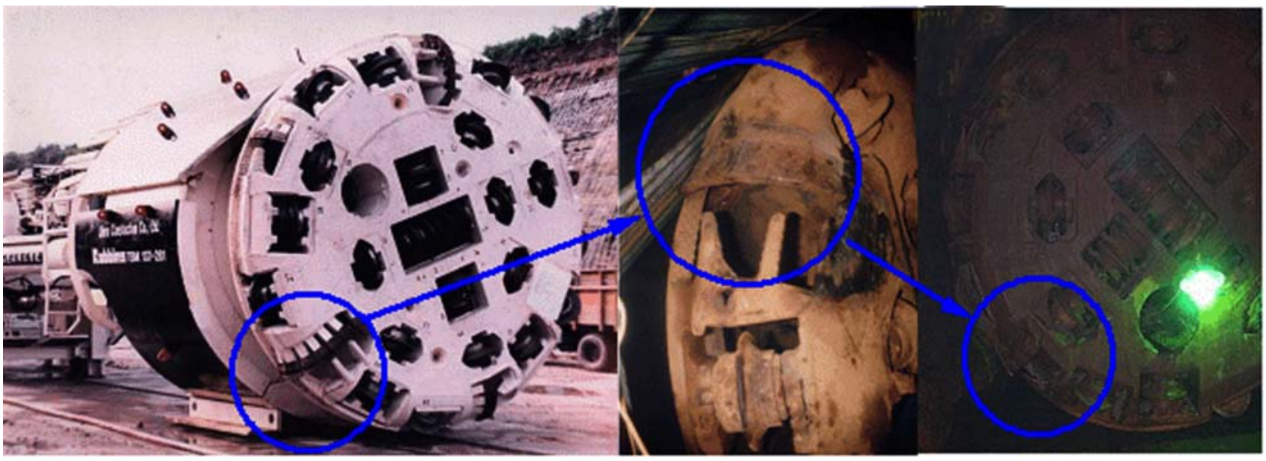


Figure 6. Modification of TBM Scraper; Before Fork-Type(Left), After Bucket-Type(Right)

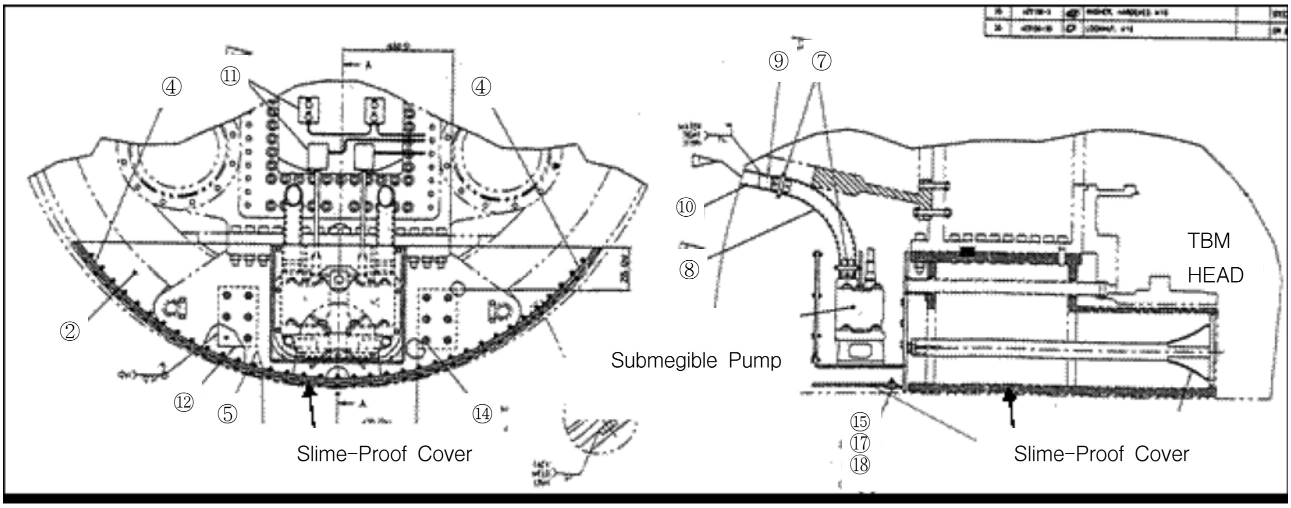


Figure 7. Installation of Slime-Proof Cover

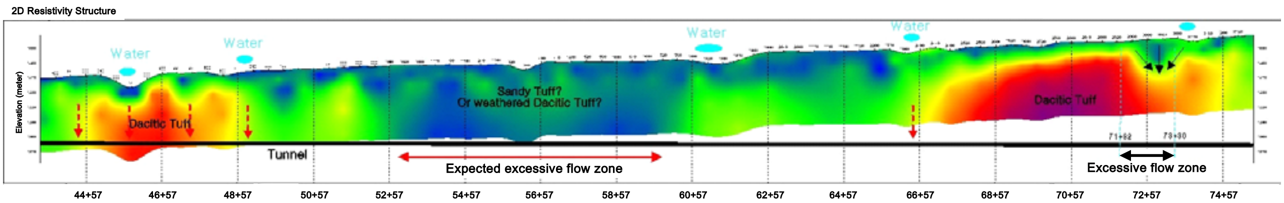


Figure 8. Result of Electrical Resistivity Survey Showing the Excessive Inflow Zone to be Expected

section between Sta. 60+52 and Sta. 45+47. The predicted inflow was in good agreement with the actual groundwater discharge measured during excavation, about 390 liter/sec.

Initially, eleven 55kw pumps were installed to discharge the inflow and then the number was increased to 20 units, when the tunnel face was at Sta. 56. Based on the predicted amount of groundwater inflow from the section beyond Sta. 56, a total of 32 pumps were installed and 7 sump pits were excavated for pump operation.

5.3 Effect of TBM modification and increase in the number of pumps

As can be seen in Figure 4, after the improvement in TBM and discharge capacity(point C), the monthly progress of TBM significantly increased and the excavation of tunnel was completed without further delay. Once the tunnel face passed large water inflow zone between Sta. 59 and Sta. 52 and reached a more competent rock formation, the tunnel excavation was made with a rapid progress.

6. CHANGE OF LINING METHOD

The total amount of groundwater inflow, measured at the completion of excavation, was up to 1,120liter/sec. Under this circumstance, it was almost impossible to meet design requirement with the cast-in-situ RC lining unless the overall grouting for surrounding rock was performed to reduce the inflow substantially. Considering that the grouting of surrounding rock under high pressure would be very difficult, if not possible, and very expansive, it was decided to change the lining methodfrom the cast-in-situ to precast concrete(PC)segment lining. PC segment lining was adopted mainly because the erection of segments would make the construction easier and the quality of work might not be influenced by the excessive groundwater inflow. The other main concern in the selection of lining was for a section where the hydro-fracturing in RC lining could occur in the dry area near Surge Tank where there was no external water pressure, but high internal water pressure. In this area, steel lining was adopted. RC lining in conjunction with the invert PC segment lining was also used fora section where rock condition was poor or there was some groundwater inflow, as summarized in Table 3.

The lining segment adopted had a thickness of 200mm, a width of 1200mm and the strength of 500kgf/cm^2 at 28days. After the break-through of tunnel, segments were installed using an erector of shield. The excessive groundwater inflow was isolated using Bullflex, a fabric bag of doughnut-shape, installed at every 50m interval between the excavated rock surface and segment, as shown in Figures 9 and 10.

Cement milk was injected to Bullflex to block the space between segment and ground and hence make the groundwater for a 50m section isolated and flowed into the tunnel through the drainage hole in the segment at the crown section. For a section between two Bullflexes, pea gravel and mortar were injected at the space between segment and ground to obtain the grouted material having the compressive strength of more than 10MPa.

As shown in Figure 11, void backfill grouting behind the segment ring was injected with the injection hole on the segment open, making the water pressure relieved. At the end of grouting work for each 50m section, the injection was

Table 3. Type of Permanent Linings Adopted

Station	Internal water pressure(kgf/cm^2)		External water Pressure (kgf/cm^2)	Ground Water Inflow	Rock condition	Lining type
	Normal condition	Surging condition				
107 to 112.6	4.0	5.6	0	None	Fair	Steel
96.3 to 107	3.946	5.506	0	None	Fair	RC + Invert Seg.
86.7 to 96.3	3.804	5.244	1.2	Small	Very poor	RC + Invert Seg
77.6 to 86.7	2.904	4.173	1.2	Medium	Very poor	Segment
76.1 to 77.6	2.771	3.904	6	Small	Fault, poor	RC + Invert Seg
31.5 to 76.1	2.754	3.862	15 to 20	Large	Good	Segment
0.0 to 31.5	2.144	2.601	3 to 15	None	Fair to Good	RC

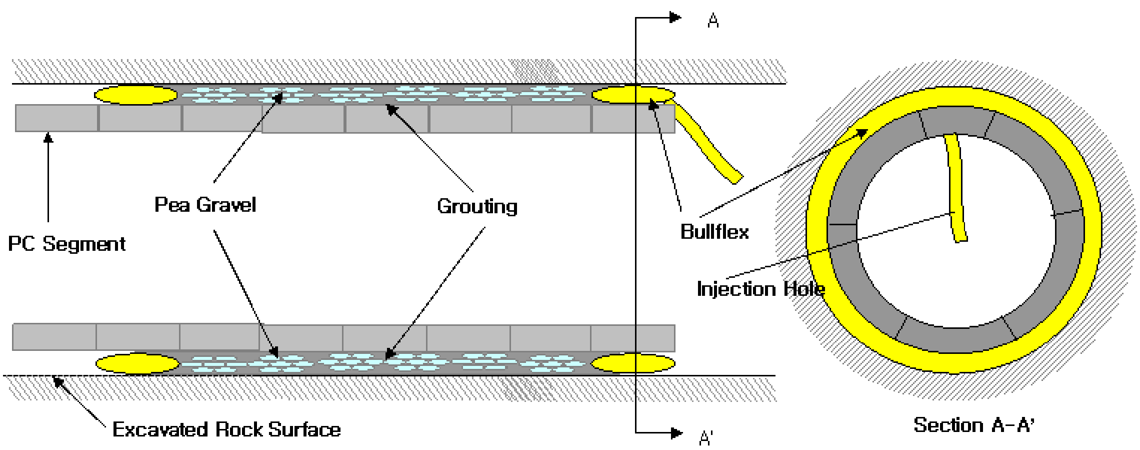


Figure 9. Bulfllex and Backfill Grouting for Cut-Off of Water

Figure 10. Bullflex Installation

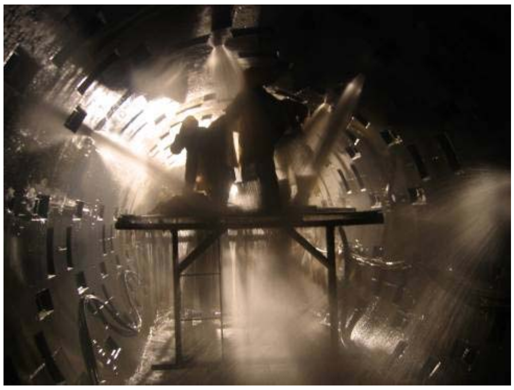


Figure 11. Backfill Grouting

made with the drain hole closed, using the grout pressure higher than the groundwater pressure behind the lining.

7. CONCLUSIONS

A 20km long headrace tunnel of 3.3m in diameter was constructed in the vicinity of caldera lake, covered with the dense jungle which hindered the engineer from carrying out an appropriate geotechnical investigation for the tender design. Based on the tender design, open-shield type TBM was adopted for the excavation and cast-in-situ RC lining as a permanent support. As the excavation of tunnel progressed, a few geotechnical features, undiscovered at the tender design stage, were revealed by the additional geotechnical investigation. These features were a predominant vertical joint set, caused by horizontal stress relief due to the cave-in of Toba Lake, avertical fault zone at Sta. 76, served as a hydro-geologic barrier and high groundwater level due to heavy rainfall throughout the year. Abundant groundwater in the bedrock with vertically well-connected joint was drained as soon as the tunnel pipe hit the groundwater reservoir, causing the excessive inflow of the groundwater to the tunnel.

The excessive inflow of groundwater forced the contractor to modify the initial construction methods, especially TBM equipment and permanent lining method. With this modification, the progress of excavation and lining was expedited and the project could be completed with some delay. This project clearly demonstrates the importance of a proper geotechnical investigation for the design and the selection of construction method.

(접수일자 : 2006년 1월 18일)

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