Abrasion and Corrective Measures of a Sediment Bypass System at Asahi Dam

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ABSTRACT:
At the Asahi Dam of Kansai Electric Power Co., Inc., a sediment bypass system was built and in operation in 1998 to take a fundamental measure to mitigate impacts by prolonged water turbidity and sedimentation ascribable to collapse of mountain slopes in the upstream caused by a great flood in 1990. The effectiveness of the bypass system has been verified and reported in the past. In the meantime, the tunnel invert has been worn notably by a significant sediment transport at high velocity and how to improve the efficiency of periodical maintenance is an issue.
This paper describes the abrasion of the tunnel invert of the sediment bypass system at the Asahi Dam that has been in operation for approximately 15 years, and evaluates possible tunnel maintenance methods using the life-cycle cost based on monitored data of abrasion.

Keywords: sediment, bypass system, abrasion, life-cycle cost

1. OUTLINE OF ASAHI DAM

The Asahi Dam is on the lower regulating reservoir of the Oku-yoshino Power Plant, which is a pure pumped-storage power plant and has been in operation since 1978. The location of the power plant is shown in Fig. 1 and the specifications are listed in Table 1.

Figure 1. Location of Asahi Dam

Table 1. Technical features of Asahi Dam

Catchment area	39.2km^2	
Design flood	1,200m^3/s	
Power plant	Max. output	1,206MW
	Max. discharge	288m^3/s
	Effective head	505m
Dam	Type	Arch dam
	Height	86.1m
	Crest length	199.41m
Reservoir	Gross storage	15.47 x 10^{6}m^3
	Effective Storage	12.63 x 10^{6}m^3
	Available depth	32m

2. PLAN AND DESIGN OF SEDIMENT BYPASS SYSTEM

At the Asahi Dam, a sediment bypass system was built and in operation in 1998 to take a fundamental measure to mitigate impacts by prolonged water turbidity and sedimentation ascribable to collapse of mountain slopes in the upstream caused by a great flood in 1990.

The sediment bypass system at the Asahi Dam was designed to divert bed load and suspended load besides wash load for the purposes of mitigating both prolonged turbidity and sedimentation. The capacity of the bypass tunnel is designed as 140m^3/sec, which can practically diminish the number of prolonged turbidity days induced by a flood peak of 200m^3/s, which equals to the discharge of the return period of 1-year, and also can sluice almost all of bed load transported by the design flood of 1,200m^3/s.

Technical features of the bypass system is shown in Table 2 and Fig. 2. Based on a uniform flow calculation, the cross-section of the bypass tunnel was designed so that the tunnel could pass the flow with the water depth of 80% of the tunnel height. D-shape cross section was adopted because of cost effectiveness and ease of maintenance. The entrance of the tunnel was composed of a diversion weir and an orifice intake, by which the volume of river water and sediment into the tunnel could be naturally regulated.

Table 2. Technical features of sediment bypass system

Diversion weir	Height	13.5m
	Crest length	45.0m
Orifice intake	Height	14.5m
	Width	3.8m
	Length	18.5m
	Type	Reinforced concrete and steel lining
	Gate	1
Bypass tunnel	Length	2,350m
	Height x Width	3.8m x 3.8m
	Shape	D-shape
	Gradient	Approx. 1/35
	Max. discharge	140m^3
Outlet	Width	8.0 ~ 5.0m
	Length	15.0m
	Type	Reinforced concrete

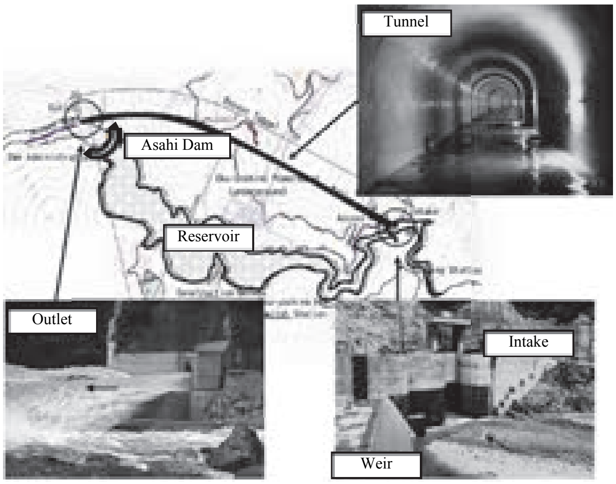


Figure 2. Plan view of the bypass system at Asahi Dam

3. PERFORMANCE AND EFFECT OF SEDIMENT BYPASS SYSTEM

3.1. Performance of Sediment Bypass System

Fig. 3 shows annual total inflow into the reservoir and bypass tunnel since the start of operation. 50% to 80% of total inflow were flowed downstream via the bypass tunnel every year. As shown in Fig. 4, a peak flow of 895m^3/s, which is the highest flow since the start of operation, was recorded at the Asahi Dam in September 2011.

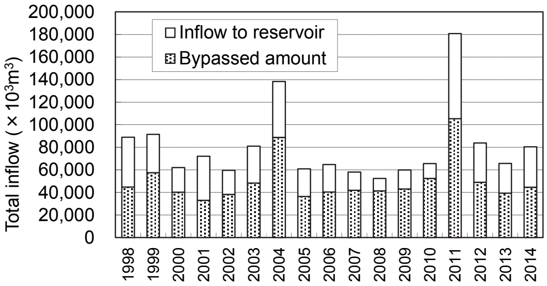


Figure 3. Annual total inflow to the reservoir

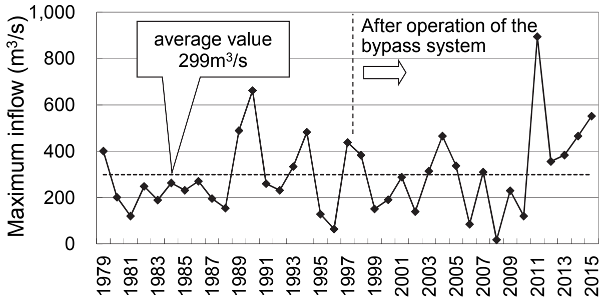


Figure 4. Maximum inflow at the Asahi Dam

3.2. Effects of Sediment Bypass System

The data on suspended solids concentration (SS) have been collected once a day at the two gauging stations: 4.3km upstream and 1.6km downstream of the Asahi Dam. Water in the reservoir was turbid for 50 to 130 days during a year on average depending on scale of floods before the sediment bypass operation. The average number of days of prolonged turbidity was reduced to about 10 days after the start of operation. It shows the bypass system was effective for mitigating water turbidity on a long-term basis.

Also, with regard to deduction of sedimentation in the reservoir, it was estimated that 80% of the total sediment, which would have deposited in case that there was not the bypass system, would be bypassed downstream. The estimate was made by calculating the volume of bed load that reached the diversion weir using a bed load equation (Ashida-Michiue Formula) and assuming that the total volume would be bypassed. Details of effects reducing sedimentation would be referred to relevant reports i.e. Harada, M., et al. 1997, Kataoka, K. 2000, Doi, H. 2005 and Fukuroi, H. 2012.

Fig. 5 shows changes in sedimentation volume (bed elevation) and Fig. 6 shows changes in bed material grain size distribution in downstream course. They indicate that changes owe to the operation of the bypass system.

The changes in downstream sedimentation volume (bed elevation) show that bed elevations in downstream channels had repeatedly increased or decreased and basically stabilized but that sedimentation volume increased to approximately 200,000 m^3 due to landslides caused by a flood of 2011.

Significant changes were made in bed material grain size distribution in downstream course. Fine grained fractions have been increased since the commencement of bypass operation. It is thus found that sediment was supplied to the downstream of the reservoir by the bypass system and the river condition is getting close to that before the construction of the dam.

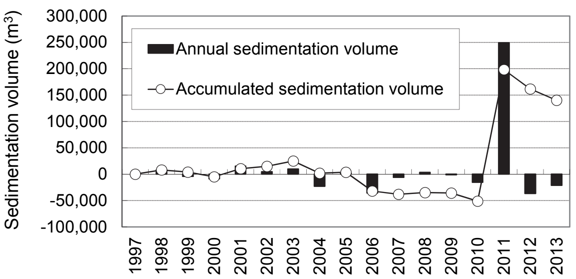


Figure 5. Changes in sedimentation volume in the downstream course

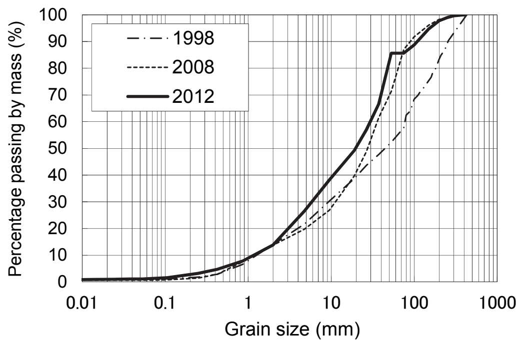


Figure 6. Changes in bed material gain size distribution in the downstream course

4. ABRASION OF SEDIMENT BYPASS

When designing the bypass tunnel, a study on concrete strength and abrasion allowance at invert section was made in order to determine its specification and allowance considering scale of repair.

Based on the study for the design, the following measures were applied. (1) an intake where severe abrasion was evidently expected to occur was reinforced with steel plates; (2) the invert section of the tunnel was constructed by high strength concrete of 36N/mm^2 except the part of the downstream end where the invert was constructed by higher strength concrete of 70N/mm^2.

It has been known through repeated repair during 15 years of operation that the distribution of abrasion tends to show a particular pattern. In addition, knowledge important to the control of abrasion was also obtained based on the results of monitoring of reinforcing materials in trial constructions. These were used to develop efficient measures to control the abrasion of the bypass tunnel.

4.1. Prediction of Abrasion in the Design Phase

In the area upstream of the Asahi Dam, granites are distributed and granite boulders have deposited on the river bed. The bed materials at the bypass intake had a mean grain diameter of 50 mm and a maximum diameter of 300 mm. It was assumed that the sediment of the size flowed through the tunnel.

For predicting abrasion in the design phase, an Ishibashi's formula shown below (Ishibashi, T. 1983) was used.

$$V = C_1 \times E_t \times C_2 \times W_t \tag{1}$$

Where, V is the volume of abrasion (damage) (m^3), C_1 is the coefficient of damage by impact (m^2/N), C_2 is the coefficient of abrasion due to friction (m^2/N), E_t is the total kinetic energy of gravel acting on the channel bed (N x m), and W_t is the total work done by abrasive force. (N x m).

The mean thicknesses of annual abrasion of concrete and steel were estimated at 40 to 50 mm/year and 0.2 mm/year, respectively, which were repairable level considering period of maintenance. As a result, the tunnel invert was therefore lined with concrete (design strength: 36 N/mm^2 and lining thickness: 40 cm).

4.2. Actual Abrasion and Abrasion Pattern

Described below are the results of monitoring of actual abrasion at the intake lined with steel-plate and at other section constructed by concrete since the commencement of bypass system operation.

4.2.1. Steel plates at intake

The intake of the bypass tunnel was lined with steel plates because the severest abrasion was expected. Damage was made predominantly on the right bank side in the steel-plate-applied area where the slope is changed from sharp to moderate (Fig. 7) and sediment likely flowed on the right bank side. Repair has been made three times in this area since the commencement of operation. After the flood of 2011, repair was made again in a wider area mainly on the right bank side too. Frequency of repair has, however, been much less than in the tunnel section.

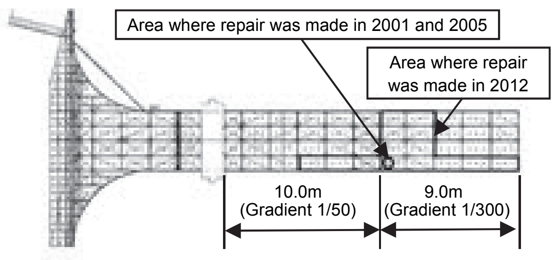


Figure 7. Areas repaired with steel plates at the intake

4.2.2. Tunnel section

Abrasion on concrete section was observed nearly throughout the invert in the tunnel section. Repair was made periodically using higher-strength concrete (design strength: 70 N/mm^2) instead of concrete of 36N/mm^2. From a viewpoint of tunnel structure, fracture of reinforcement in the invert was allowed, however, the invert was repaired before the rock was exposed to prevent adverse effects on the sidewalls and arch section. In actual, repair was made on a priority basis before the fracture of reinforcement over a length of approximately 100 m from the intake which was an important part to tunnel structure. Sidewalls were worn slightly in areas at low elevations. In the arch, long-term cracking and exposure of reinforcement were observed in some areas. Because it was unlikely that sediment of the grain size contributing to abrasion reached the arch, these phenomena were attributed to the effects of ordinary deterioration.

As abrasion in the invert was predominant, the past records of distribution and volume of abrasion in the invert were focused and summarized. The cumulative abrasion since the commencement of bypass operation is shown using contours to verify the tendency of planar distribution of abrasion (Fig. 8). The cumulative abrasion is calculated by annual measurement results. The depth of abrasion is locally larger near the tunnel outlet than near the tunnel intake. In the transverse direction, abrasion was predominant on the right bank side at the intake and on the left bank side at the outlet. The maximum cumulative abrasion depth on the left bank side at the outlet was largest at 1,272 mm. The tendency shows that the sediment does not flow uniformly in the cross section but flows in a certain pattern due to the tunnel alignment that bends leftward from the midpoint. In other words, bed load concentrate on the right bank side at the intake and then on the left bank side downstream of the bend owing to the secondary flow created by bending, resulting in increase in abrasion volume. Abrasion depth was larger at the outlet than at the intake also because secondary flow was more likely to develop in the bend than at the intake and consequently the points of passing sediment flow were concentrated more.

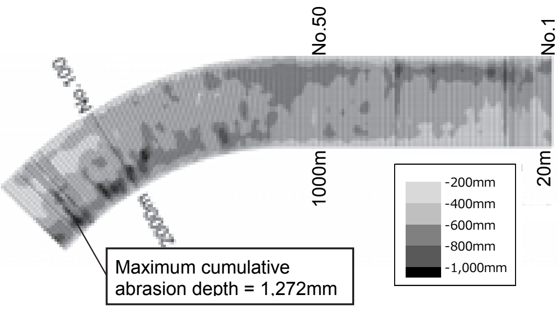


Figure 8. Cumulative abrasion in the tunnel invert (sum of abrasion between the commencement of bypass operation and November 2011)

The results of measurement of the volume of sediment that passed the bypass tunnel, volume of abrasion in the tunnel invert and the volume of materials used for repair are shown in Fig. 9 in order to quantitatively evaluate abrasion. The changes in abrasion volume affected by the volume of sediment passing the bypass tunnel were indicated by the relation between annual volume of abrasion and sediment load in the bypass tunnel (annual abrasion volume (m^3/year)/sediment load in the bypass tunnel (m^3/year) (Fig. 10). The volume of abrasion tends to increase nearly in proportion to the volume of sediment that passed the bypass tunnel. The volume of abrasion was highest in 2011 when the maximum flow was recorded. The percentage of high-strength concrete increased on the invert surface of the bypass tunnel owing to periodical repair (Fig. 11). Fig. 10 shows that the volume of abrasion relative to the volume of sediment decreased from 2003 when the percentage of high-strength concrete exceeded 70% and in subsequent years. In addition, the planar distribution of abrasion was not affected by the volume of passing sediment, and abrasion was predominant in the convex bend.

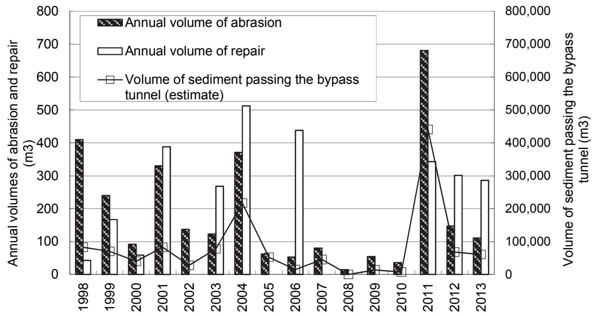


Figure 9. Annual volumes of abrasion and repair

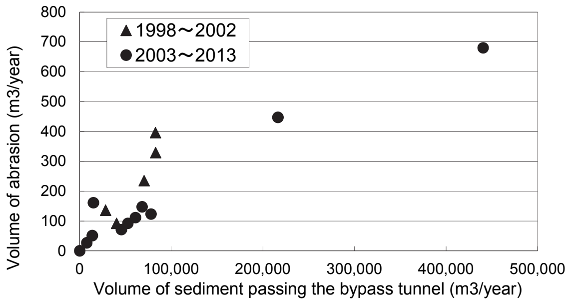


Figure 10. Relationship between the volumes of abrasion and sediment passing the bypass tunnel

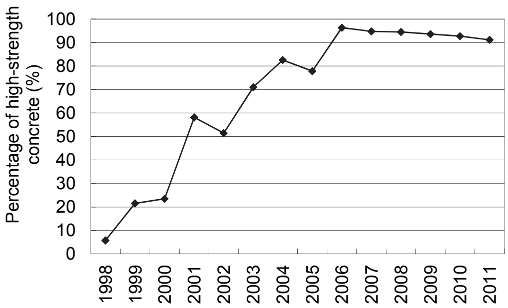


Figure 11. Percentage of high-strength concrete

4.3. Trial Construction

Other materials than concrete have also been applied in the invert on a trial basis in order to confirm effectiveness of the original design since 1999. Fig. 12 shows how they were placed. As reinforcing materials, higher-strength precast panels, steel plates and stones which had a good record as materials highly resistant to abrasion, and resins with a low modulus of elasticity were selected. The abrasion resistance of respective materials was tested and monitored through actual bypassing. Nearly all of the materials were greatly worn or delaminated in a few years after its construction. It was found that abrasion was likely to progress from the joint of the material, and created a weak point and that the reinforcing material was

delaminated rapidly at once owing to the impact of gravel or for other reasons. In 2013, an abrasion-resistant protective material made of rubber, which has been applied in many sabo dams, was placed as one of trial materials additionally.
The condition was verified ten months after its installation. As a result, it was found that the volume of abrasion was greater in the bypass tunnel than in sabo dams as the mean abrasion volume was 0.7 to 1.1 mm/year in the bypass tunnel while actual mean abrasion volume in sabo dams was 0.1 to 0.3 mm/year.

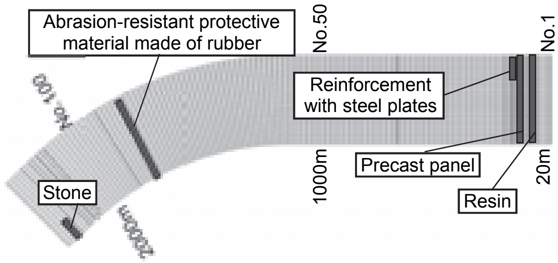


Figure12. Locations of materials applied in trial construction

5. DISCUSSIONS ON OPTIMUM MATERIAL AGAINST ABRASION

High-strength concrete (70N/mm^2) has been adopted to resist abrasion because of the availability of repair materials and ease of construction. In order to improve the efficiency of repair cost, new repair materials were examined based on the results of trial construction. The materials compared with one another are described in detail in Table 3. The materials were applied in locations with high volume of abrasion. In other locations, high-strength concrete was applied for repair as conventionally practiced. The annual volume of abrasion of each repair material was expressed as a ratio to the annual volume of abrasion of high-strength concrete based on the results of trial construction and published data. Unit costs are expressed by relative value to a unit cost for high-strength concrete based on the actual results and published costs. Table 4 shows the area and the percentage to the total area of invert which corresponds to various amounts of annual abrasion volumes of high-strength concrete. The service life of high-strength concrete was obtained by dividing a marginal volume of abrasion of 200 mm by the annual volume of abrasion.

Table 3. Annual volume of abrasion and unit cost of repair materials

No.	Repair material	Annual volume of abrasion	Unit cost
1	High-strength concrete	1.0000	1.0
2	Resin (t=15mm)	0.2430	2.5
3	Precast panels (t=50mm)	0.2500	4.0
4	Reinforcing steel plates (t=16mm)	0.0028	6.4
5	Abrasion-resistant rubber protectors (t=50mm)	0.0075	8.5
6	Stones (t=150mm)	0.1000	22.6

Table 4. Annual volume of abrasion of high-strength concrete in invert (2006 to 2011) and service life

Annual volume of abrasion (mm)	Area (m^2)	Ratio to total area of invert (%)	Service life (years)
160 or higher	0.0	0	1
150 or higher	4.0	0.05	
140 or higher	14.0	0.16	
130 or higher	21.0	0.24	
120 or higher	27.0	0.30	
110 or higher	35.0	0.39	
100 or higher	40.0	0.45	
90 or higher	56.0	0.63	2
80 or higher	81.5	0.91	
70 or higher	128.1	1.44	
60 or higher	200.7	2.25	3
50 or higher	375.3	4.20	
40 or higher	704.8	7.89	4
30 or higher	1,539.2	17.24	5
20 or higher	3,190.6	35.73	7
Less than 20	5,739.4	64.27	10

Life-cycle cost of each materials were calculated by the formula shown as below (Eq. 2). Fig. 13 shows the relationship between the location where the repair material was applied and life-cycle cost. For comparing the life-cycle cost, the study period was set at 50 years and the social discount rate was set at 4.0%. If the concerned facility has a residual value at the end of the 50-year period, the residual value was deducted.

$$LCC = \sum_{t=0}^{49}\left\{\frac{C_c}{(1+r_s)^t}\right\} - R_v \qquad (2)$$

Where, LCC is life-cycle cost, C_c is construction cost in the t th year, t is passed years, r_s is social discount rate and R_v is residual value at the end of the 50-year period.

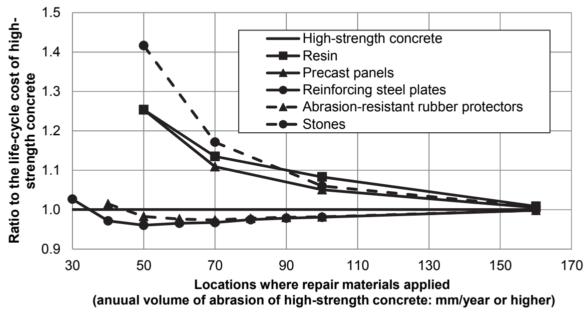


Figure 13. Relationship between the location of repair and life-cycle cost

Fig. 13 shows that the life-cycle cost in cases where resin, precast panels and stones were used were higher than in the case where higher-strength concrete was applied throughout the invert. The life-cycle cost of the case which reinforcing steel plates and abrasion-resistant rubber protectors were used were at a minimum level in case that the material were applied at the location with annual volume of abrasion of high-strength concrete were 50~70mm/year. Applying the materials at a location with smaller annual volume of abrasion led to the increase of the area where the material should be applied, and increased the life-cycle cost. When the materials were applied in locations with large annual volume of abrasion, life-cycle cost also increased because the area of high-strength concrete that requires frequent repair increased although the area where the material was applied decreased. Life-cycle cost was the lowest in the case where reinforcing steel plates were applied in locations with an annual volume of abrasion of 50 mm or higher. Life-cycle cost was reduced compared to the case where high-strength concrete was applied throughout the invert by approximately 4% in 50 years. Thus, the effect of cost reduction was very small. In the case where abrasion-resistant rubber protectors were adopted, the life-cycle cost was estimated approximately 3% below the high-strength concrete case.

Reinforcing steel plates were selected as the most economical repair material based on the results of comparison in life-cycle cost. Reinforcing steel plates, however, have several drawbacks. Uplifting or delamination occurred for unknown reasons in trial construction. There is a possibility of abrasion advancing from the point of uplifting. Such defects, once they occur, involve large-scale repair work. The life-cycle cost shown in Fig. 13 takes no effects of steel plate delamination into consideration. If the effects are considered, life-cycle cost is likely to exceed the cost in the case where high-strength concrete is applied throughout the invert. It was assumed that a certain percentage of reinforcing steel plates are delaminated every five years according to the results of the trial construction, and life-cycle cost including the cost of repair of delaminated steel plates were compared (Fig. 14). It was found that life-cycle cost of reinforcing steel plates exceeded the cost in the case where high-strength concrete was applied throughout the invert when the percentage of delaminated plates exceeded 12%. As the area where reinforcing steel plates were applied in this study (Fig. 15) involve uncertainty, making repairs in a large area bundling scattered locations where reinforcing steel plates should be applied is practical, and then applying reinforcing steel plates is no longer advantageous in terms of cost because high unit cost is required.

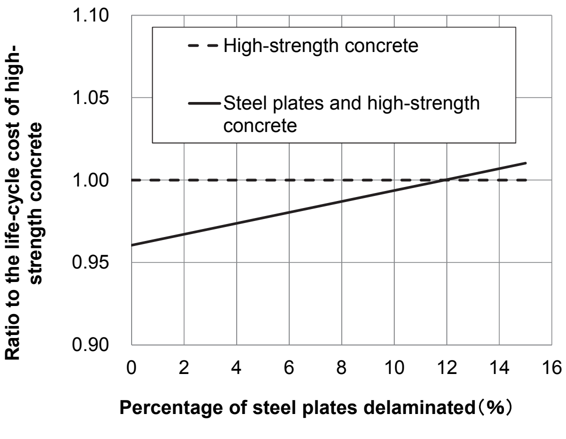
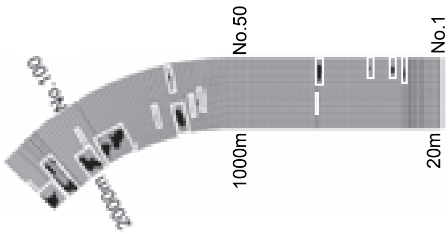


Figure 14. Relationship between the percentage of steel plates delaminated and life-cycle cost



Shown in black: Locations with an annual volume of abrasion of 50 mm/year or higher
Shown in white: Locations where reinforcing steel plates were applied

Figure 15. Locations where reinforcing steel plates were applied

In view of the above discussions, the repair method using high-strength concrete as currently practiced is considered efficient and valid because it requires no large-scale equipment, unlike the method employing reinforcing steel plates, and because it is applicable to the points of repair of varying shapes that are scattered in the invert.

6. CONCLUSIONS

This paper described the distribution and volume of abrasion in a bypass tunnel. The results of trial construction could serve as basic data for determining types of repair materials and the frequency of repair as they showed the importance of treatment at the joint, a weak point in abrasion control, and that the bypass tunnel had larger volume of abrasion than sabo dams and was placed under a severe condition. As the result of life-cycle cost evaluation, it was found that the repair method using high-strength concrete as currently practiced is considered efficient and valid.

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