International Symposium on
Sediment Management and Dams
(For the 2nd EADC Symposium)

Proceedings

October 25-26, 2005

Organizing Committee for the International Symposium on Sediment Management and Dams

Co-Sponsored by
Japan Commission on Large Dams (JCOLD)
Korean National Committee on Large Dams (KNCOLD)
Chinese National Committee on Large Dams (CHINCOLD)
Japan Society of Hydrology and Water Resources
Ecology and Civil Engineering Society
Japan Society of Dam Engineers
Asia-Pacific Group of ICOLD

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Contents of Proceedings

Papers of Contribution

I. Sediment Management

Opening Remarks

Upon opening this symposium, it is my great honor and pleasure to offer my remarks as chair of the Organizing Committee for the International Symposium on Sediment Management and Dams.

We have with us today distinguished guests, Mr. Alessandro Palmieri (Senior Dams Specialist, The World Bank), Prof. Dr. Abdalla Abdelsalam Ahmed (Director of UNESCO Chair in Water Resources) and Mr. Lin Chuxue (Vice President, China Yangtze Three Gorges Project Development Corp.), who will later provide guest speeches.

In addition, a keynote speech will be made by Mrs. Ayako Sono, who is a Japanese novelist known for her active involvement in social issues such as "poverty and development" both at home and abroad.

Moreover, I find it very happy and grateful that some 44 participants from China, 28 participants from Korea, 3 participants from Thailand and 150 participants from Japan have been able to join us.

This symposium also encompasses the 2nd EADC Symposium. As you may well know, the East Asia Area Dam Conference was established in May of last year in Seoul as a joint endeavor of the respective Commission of Large Dams in Japan, China and Korea. And last October, the 1st EADC Symposium was held in Xi'an, China.

The main theme of this symposium — Sediment Management and Dams — follows up on the theme of the 1st EADC Symposium. The "sedimentation problem" is not a problem just for dams but also for all of the river basin. Therefore, a wide-ranging discussion is needed; thus, not only the three EADC members but also those academic societies in Japan closely related to the issue, such as the Japan Society of Hydrology and Water Resources, the Ecology and Civil Engineering Society and the Japan Society of Dam Engineers, are taking part in the event as co-sponsors.

Furthermore, in line with the EADC's establishment, invitations were sent to members of the Asia-Pacific Group of ICOLD and we received two papers from India, in addition to having a representative from Thailand join us here at this symposium.

In terms of the number of papers, there were 16 submitted from Japan, 10 from China, 9 from Korea and 2 from India, for a total of 37 papers. Of these — needless to say all of high quality — although all 37 is to be compiled in the proceedings, 17 of these will be presented at the symposium venue. It is my wish that these important papers will be used for the future development of dam operations.

In Japan, the end of October is the most amenable season in terms of weather. I hope our overseas visitors will take this opportunity to gain an understanding of our society and culture in an enjoyable manner. I would also like to call upon the Japanese participants to exchange opinions with as many of the overseas participants as possible, in order to expand their base of knowledge and to deepen their friendship with our visitors.

Finally, I would like to close my remarks by noting that I would be very glad to see the symposium succeed and the results thereof will contribute to the world's sustainable dam operations.

Takashi Toyoda, Dr. Eng.
Chairman, Organizing Committee for International Symposium on Sediment Management and Dams

GUEST SPEECH
&
SPECIAL LECTURE

The RESCON Approach to Reservoir Sedimentation Management

Mr. Alessandro Palmieri
Senior Specialist of Dams, The World Bank

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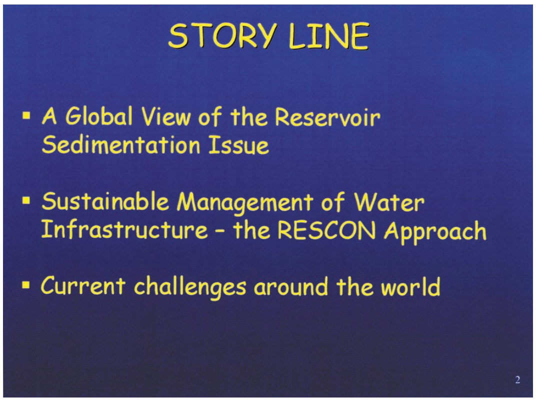

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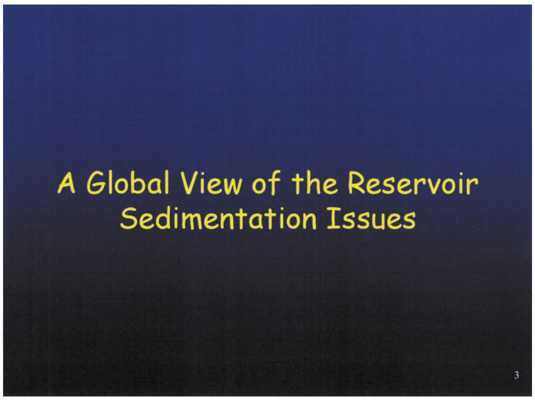

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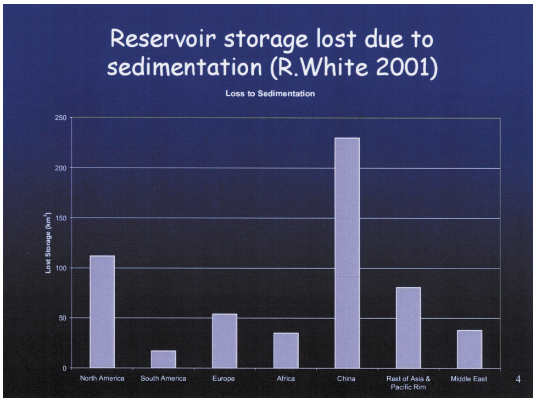

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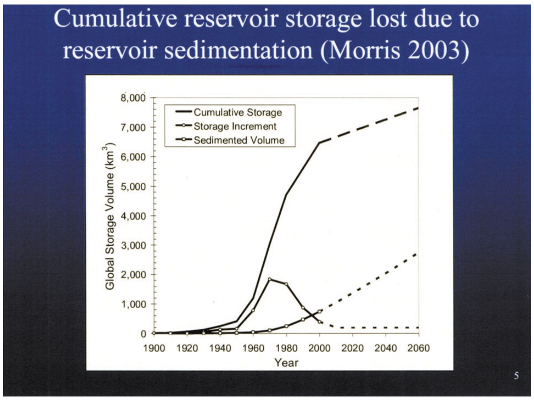

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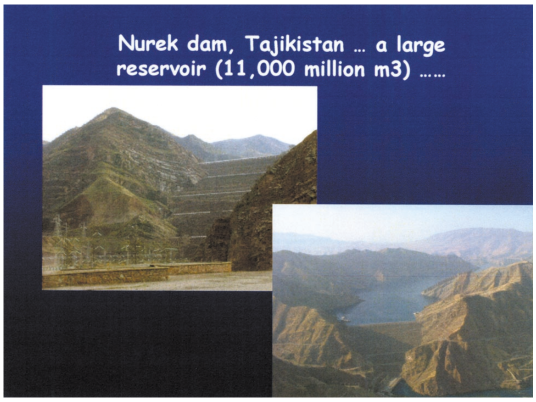

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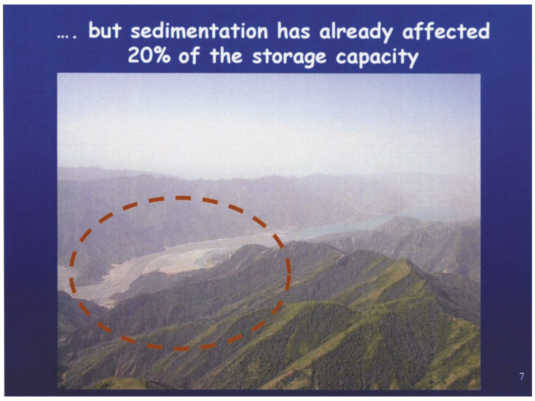

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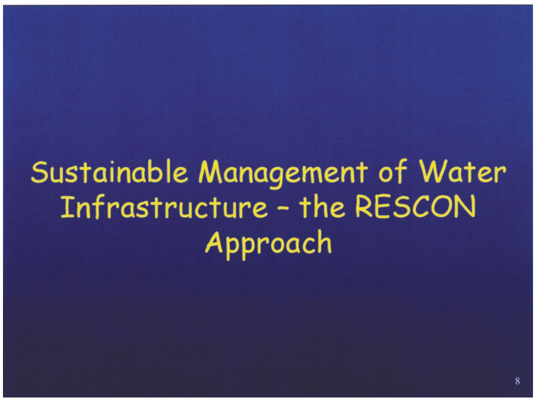

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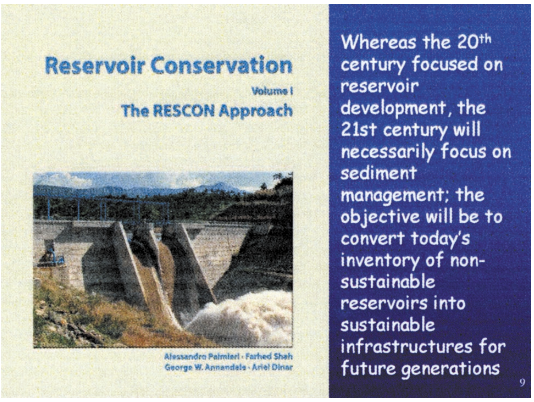

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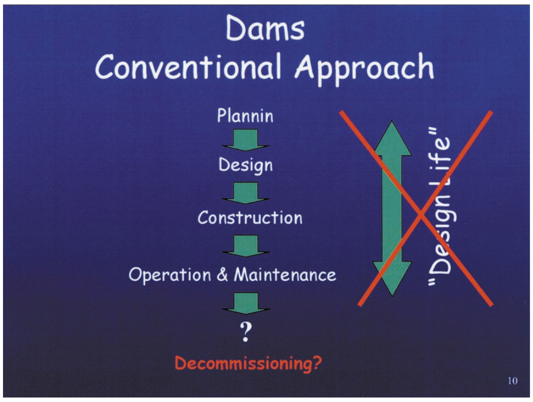

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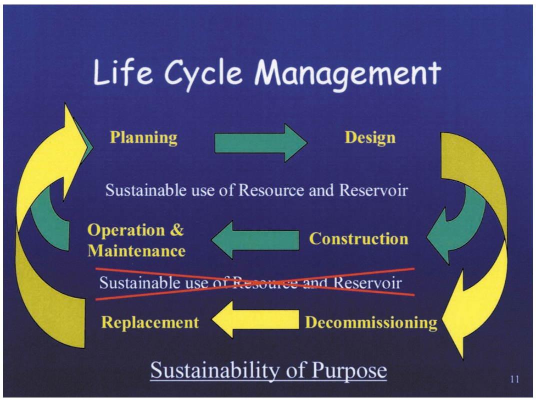

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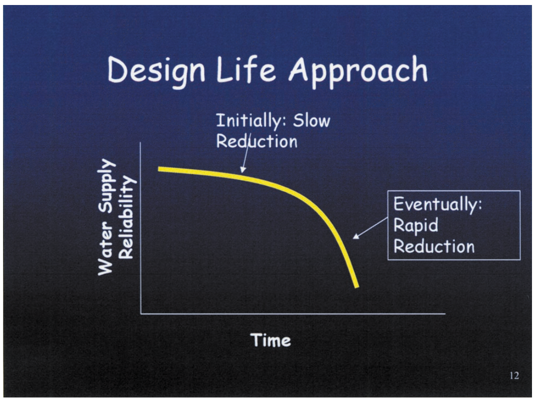

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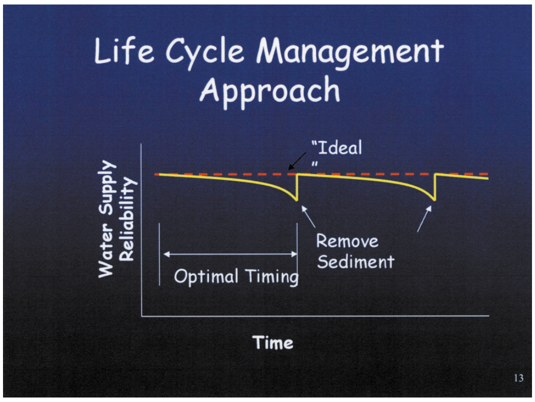

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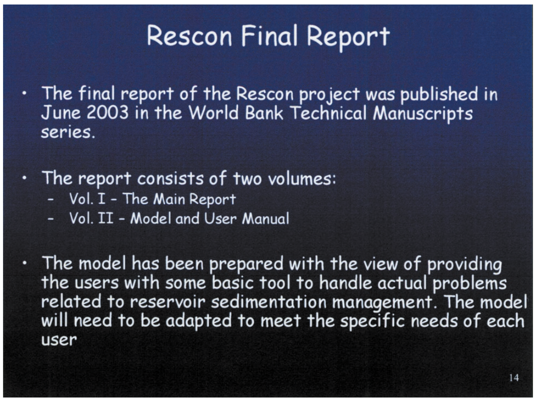

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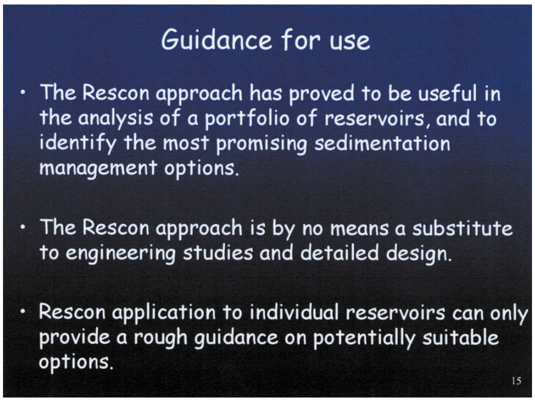

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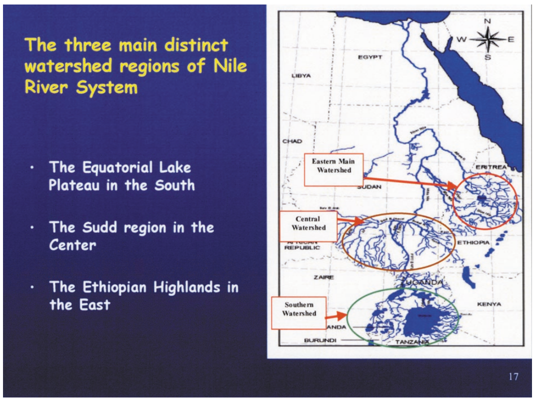

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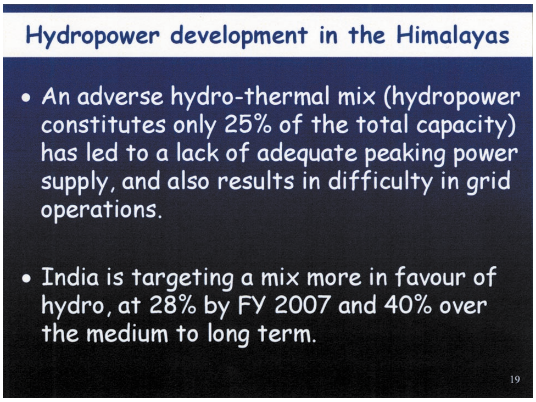

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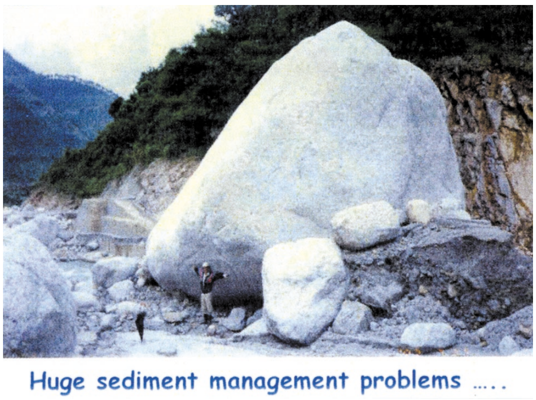

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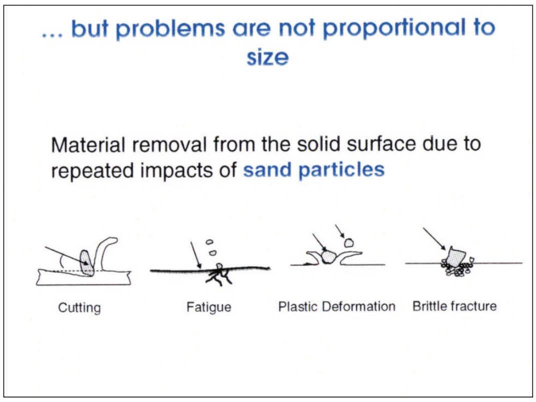

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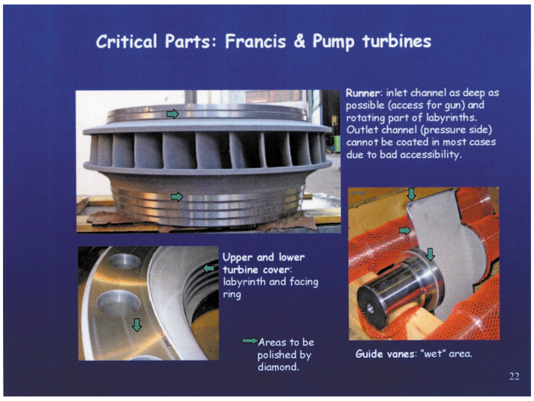

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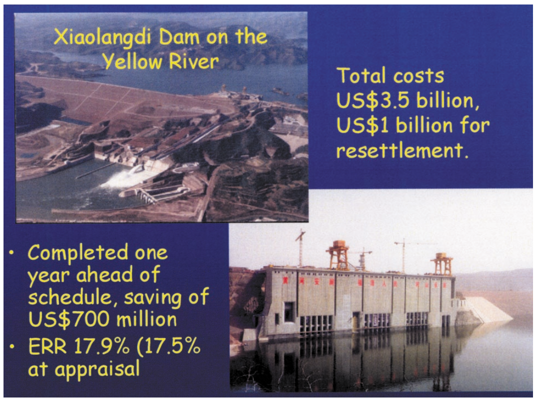

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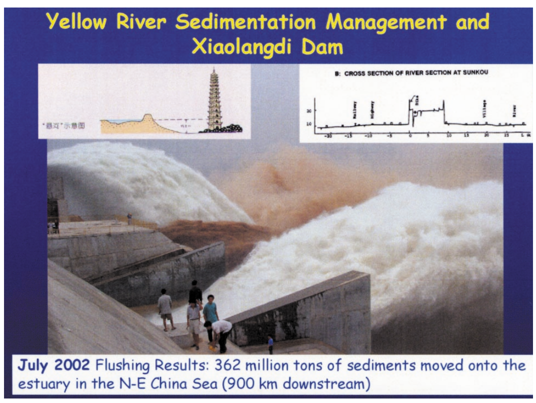

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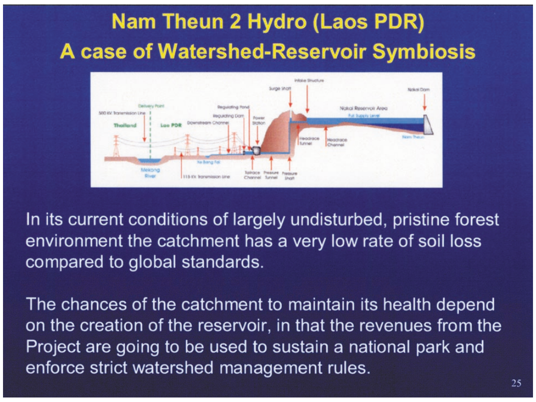

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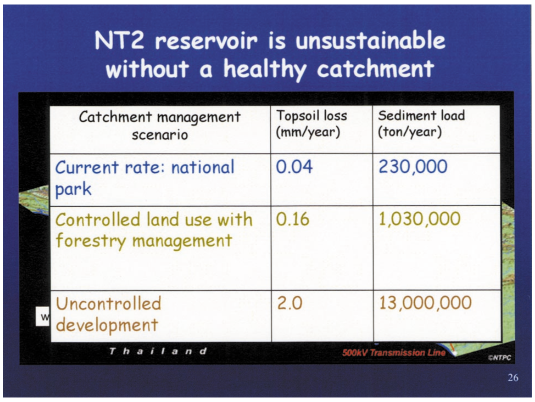

Catchment management scenario	Topsoil loss (mm/year)	Sediment load (ton/year)
Current rate: national park	0.04	230,000
Controlled land use with forestry management	0.16	1,030,000
Uncontrolled development	2.0	13,000,000

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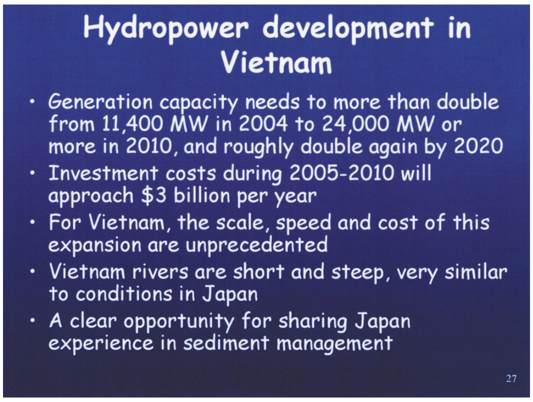

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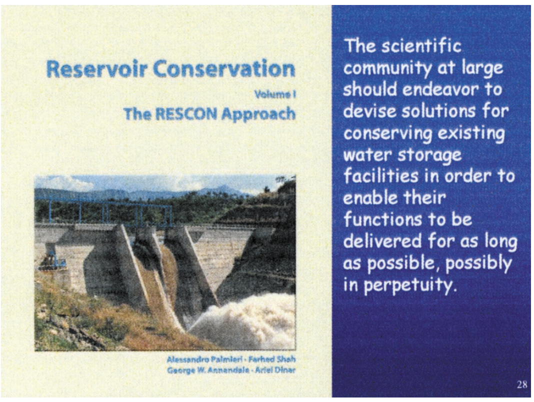

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Sedimentation Issues in the Nile River

Prof. Abdalla A. Ahmed
UNESCO

International Symposium on Sediment Management and Dams in Japan

Sedimentation in the Nile River

Prof. Dr. Abdalla Abdelsalam Ahmed

24 – 29 Oct. 2005 Tsurumi-ku – Yokohama, Japan

1

- Water is essential for mankind survival and the development of civilizations.
- The population of the world continues to increase by 100 million people each year, with a corresponding demand of water.
- Environmental pollution and water contamination make the situation very critical.
- Sediment is considered one of the main water pollutant factors.

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Sedimentation impacts

Sedimentations has its impacts on the utilization of the Nile Basin water resources, ranging from watershed degradation, sediment deposition in reservoirs to management difficulties of irrigation canals networks.

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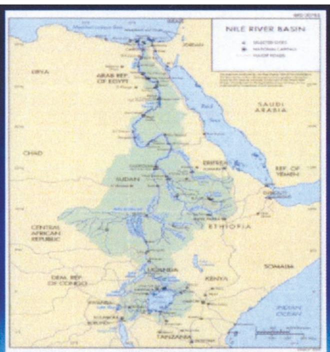

General Layout of the Nile Basin

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Nile River receives its flow from three main distinct watersheds:-

I. Equatorial lakes plateau in the South.

II. Sudd region in the Center.

III. Ethiopian Highlands in the East.

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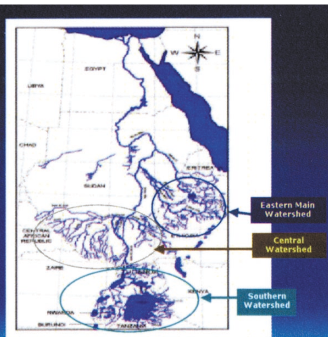

Main Watersheds of Nile River Basin

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The share and distribution of the 10 riparian countries to the Nile River

Country	Total area of the country (km2)	Area of country within the basin (km2)	As % of total area of basin	Aver. annual rainfall in the basin area (mm)		
				Min.	Max.	Mean
Burundi	27 834	13 260	0.4	895	1 570	1 110
Rwanda	26 340	19 876	0.6	840	1 935	1 105
Tanzania	945 090	84 200	2.7	625	1 630	1 015
Kenya	580 370	46 229	1.5	505	1 790	1 260
DR Congo	2 344 860	22 143	0.7	875	1 915	1 245
Uganda	235 880	231 366	7.4	395	2 060	1 140
Ethiopia	1 100 010	365 117	11.7	205	2 010	1 125
Eritrea	121 890	24 921	0.8	240	665	520
Sudan	2 505 810	1 978 506	63.6	0	1 610	500
Egypt	1 001 450	326 751	10.5	0	120	15
Nile basin	8889 534	3 112 369	100.0	0	2 060	615

7

Nile River Flows

- From the confluence of the Atbara River north to the Mediterranean, the Nile obtains no effective inflow.
- The total estimated annual inflow entering Lake Victoria from streams flow and rainfall is 118 billion m3 while the evaporation is estimated to be 94.5 billion m3, leaving only 23.5 billion m3 to flow down the Nile River
- In the Sudd area inside Sudan, the loss is estimated as 33.9 billion m3, leaving 15 billion to flow into the White Nile.
- In the High Aswan Dam reservoir, the losses are estimated to be 10.5 billion m3, while the losses within Sudan downstream Malakal are about 7.0 billion m^3.

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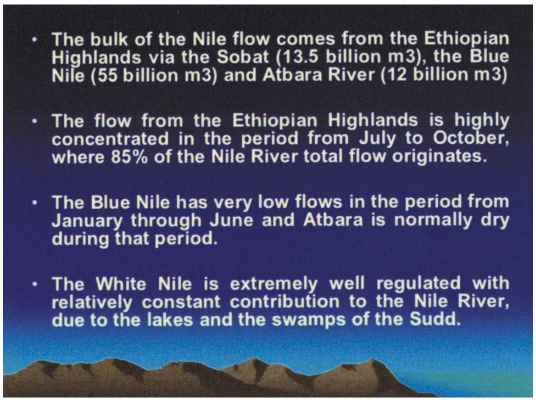

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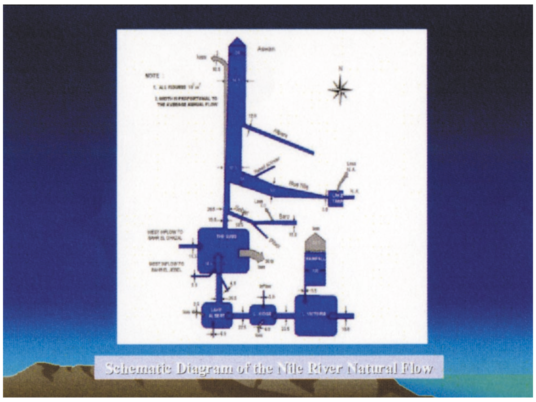

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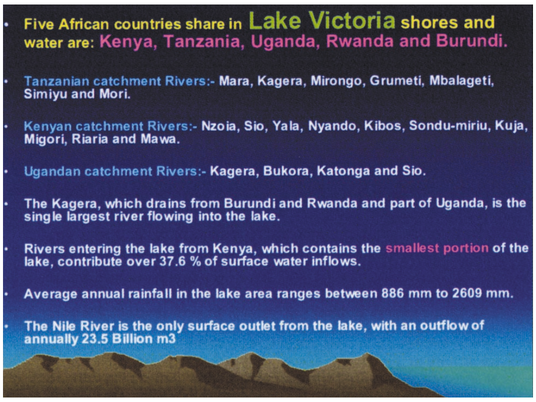

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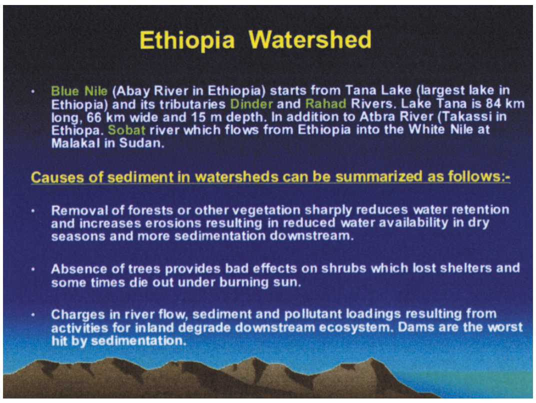

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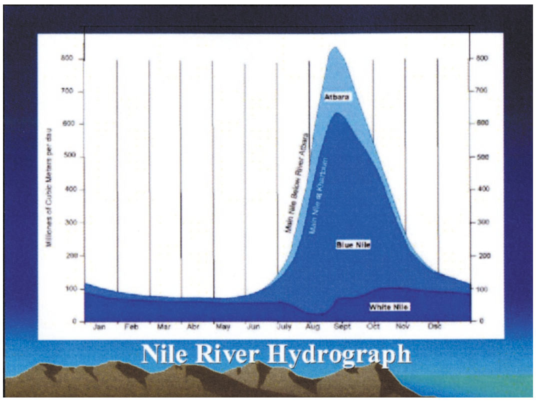

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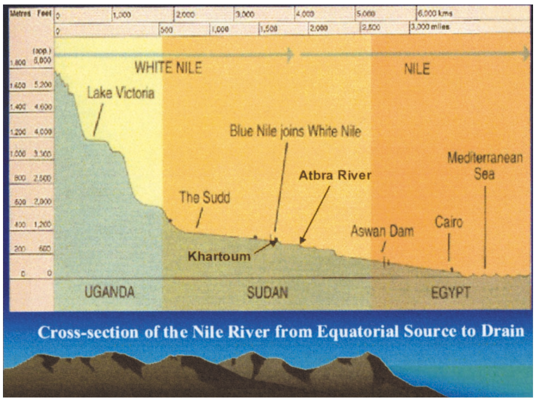

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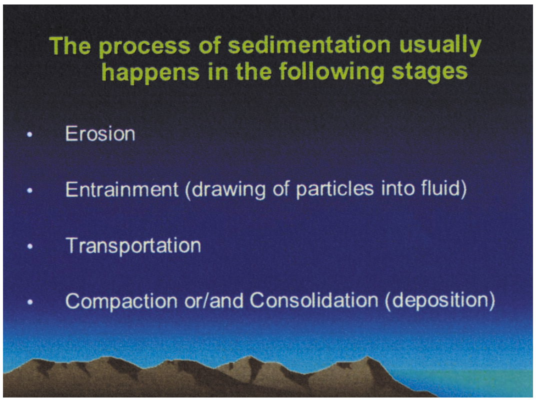

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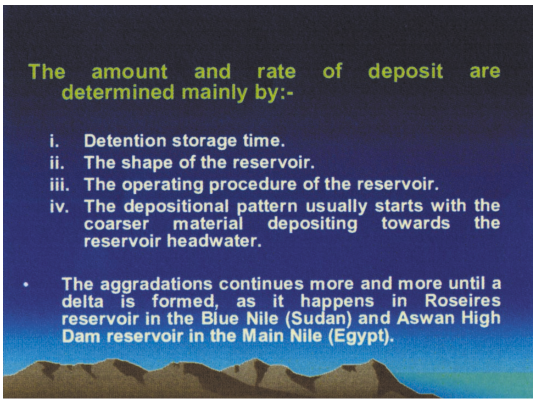

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Nile River Sediment

- Most of the sediment in the Nile flows from the Ethiopian Highlands through the Blue Nile and Atbara River.
- The White Nile and its tributaries lose most of its sediment load by spilling and deposition over flood plains, lakes and marshlands inside Sudan.
- Nearly all of the sediment (~ 90%) comes from the Blue Nile during the flood season (July- Oct.)
- **The sediment load of the Blue Nile at El Diem is 140 million tons per year.**
- **The sediment load at Aswan High Dam was 160 million tons taking into consideration the amount of sediment transported by Atbara River.**

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- **This is compared to 150 Million tons for Mississippi River, 250 Million tons the Colorado River and 2000 Million tons for Yellow River in China.**
- **there is no reliable means of bed load information in the Nile River. However, the bed load is believed to be negligible.**
- **The coarser sand usually deposits in the upper portion of the Blue Nile near the Ethiopian/Sudanese boarder.**
- **The lighter sediment is carried by the flow to the downstream.**
- **The suspended sediment load distribution is 30% clay (<0.002 mm), 40% silt (0.002 – 0.02 mm) and 30% fine sand (0.02 – 0.2 mm). Therefore, it may be considered in general wash load.**

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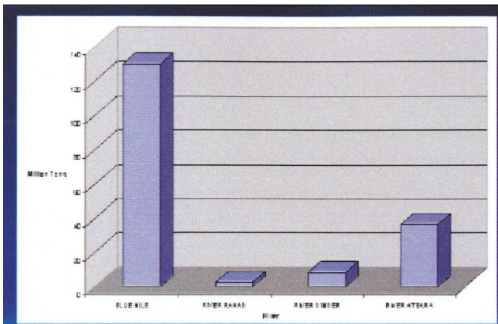

Total Annual Sediment Load (million tons) in The Nile River

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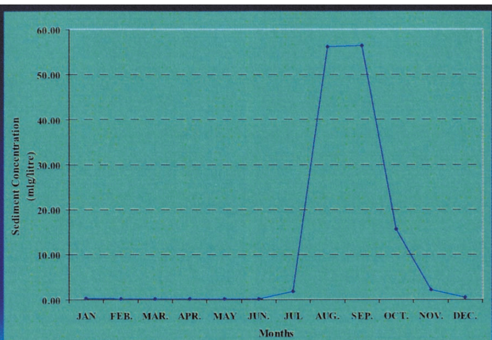

Suspended Sediment Concentration in AHD Reservoir

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Runoff and Sedimentation

- The Blue Nile runoff coefficient is higher than Atbara River one.
- This can be attributed to several reasons:
  - Atbara River catchment area is northern of the Blue Nile one; therefore, it is drier with less green cover.
  - Higher sediment concentration in Atbara River compared to the Blue Nile.
  - The Blue Nile contributes to the flow of the Nile system with 65% while Atbara River is contributing with only 16%.

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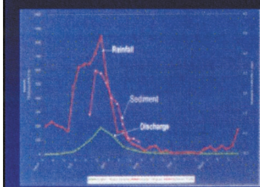
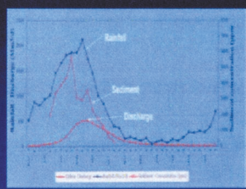


Comparison of rainfall, Discharge and Sediment Yield in the River Atbara



Comparison of Rainfall, Discharge and Sediment Yield in the Blue Nile

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- Here in Sudan the problem has been reflected downstream in terms of sediment deposition in the reservoirs and the irrigation canalization networks, causing:

- ❑ flood risks
- ❑ crops damage
- ❑ pumps intakes blockage
- ❑ low production
- ❑ hydropower generation difficulties

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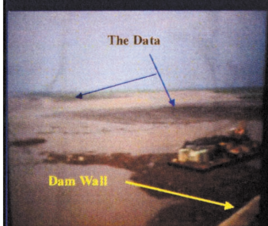
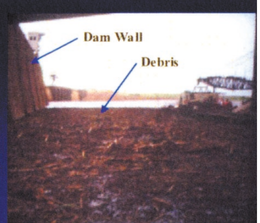


Delta Formation Upstream, Roseries Dam



Debris in the Reservoir Upstream, Roseries Dam

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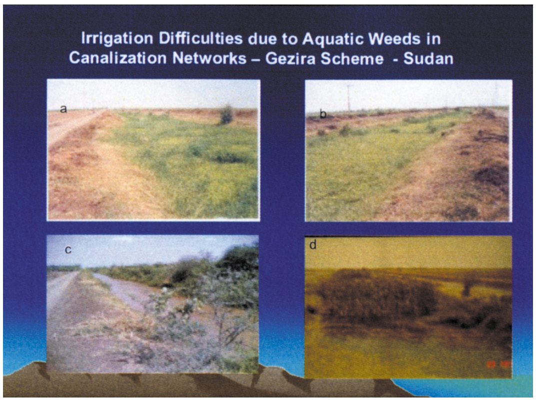

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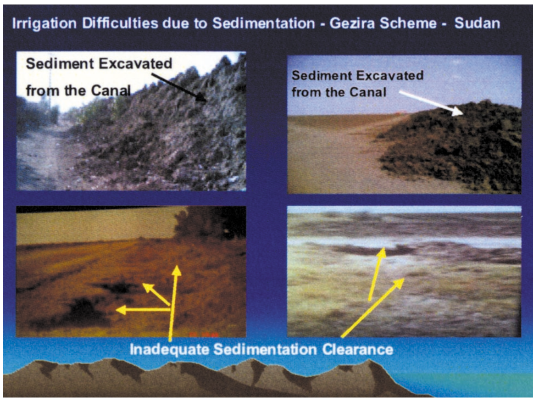

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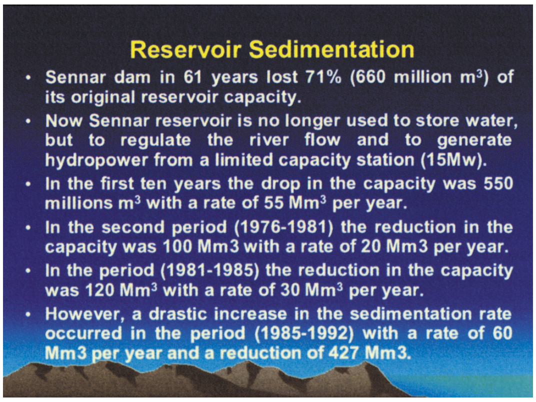

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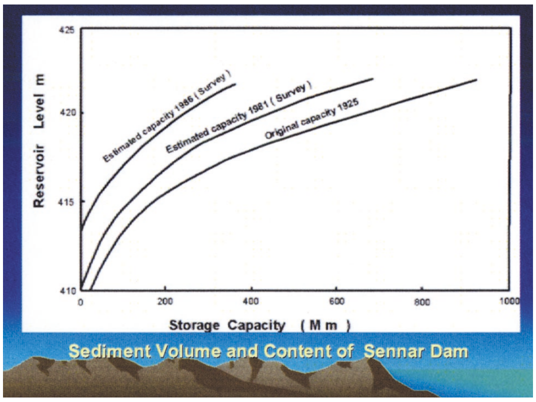

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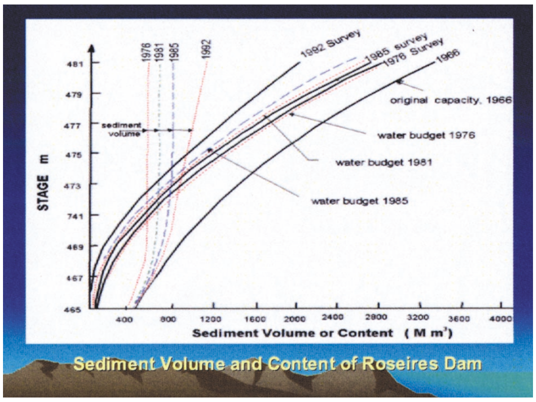

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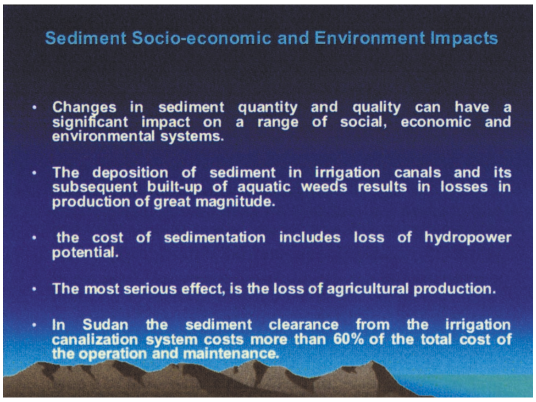

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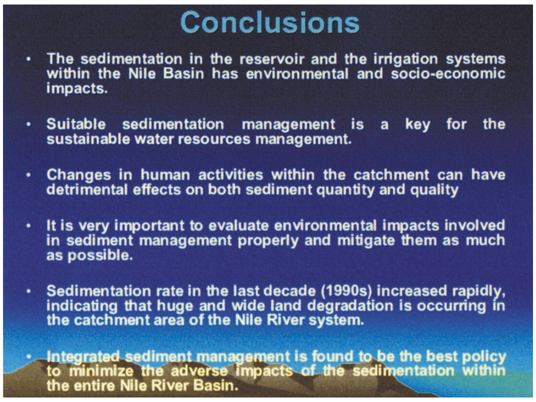

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PAPERS FOR ORAL PRESENTATION

Sediment Flushing Efficiency and Selection of Environmentally Compatible Reservoir Sediment Management Measures

Tetsuya Sumi
Associate Professor, Department of Civil Engineering
Faculty of Civil Engineering, Kyoto University, Japan

Abstract

The Japanese rivers are characterized by high sediment yield in comparison with other countries due to the topographical, geological and hydrological conditions. This has consequently caused sedimentation problems to many reservoirs constructed for water resource development or flood control purposes. Among them, a significant amount of sediment storage capacity, which was usually designed to accommodate in 100-years, has already been lost because of sedimentation proceeding faster than expected. Since 1980s, when the need for a reservoir safety check on sedimentation was recognized, Japanese dams above a certain size were obliged to regularly conduct sediment investigation. A lot of valuable data collected by these investigations have provided so much knowledge on reservoir sedimentation management.

The necessity for the reservoir sediment management in Japan can be summarized in the following three points: 1) to prevent the siltation of intake facilities and aggradations of upstream river bed in order to secure the safety of dam and river channel, 2) to maintain the storage function of reservoirs, and realize sustainable water resources management for the next generation, and 3) to release sediment from dams with an aim to conduct comprehensive sediment management in a sediment routing system. Which point gains importance depends on each dam and each river; however, it is necessary for every dam to recheck the need for sediment management.

Sediment management approaches are largely classified into the following techniques: 1) to reduce sediment transported into reservoirs, 2) to bypass inflowing sediment and 3) to remove sediment accumulated in reservoirs. In Japan, in addition to conventional techniques such as excavation or dredging, sediment flushing and sediment bypass techniques are adopted at some dams: e.g. at Unazuki dam and Dashidaira dam on the Kurobe river, and at Miwa dam on the Tenryu river and Asahi dam on the Shingu river, respectively. These dams practically using such techniques are focused on as advanced cases aiming for long life of dams.

The problems to promote such reservoir sedimentation management in future are 1)Priority evaluation of reservoirs where sediment management should be introduced, 2)Appropriate selection of reservoir sediment management strategies and 3)Development of efficient and environmental compliance sediment management technique. Especially, when the sediment management measures are selected, it is necessary to consider those environmental effects in the river in the advanced country including Japan. In that case, development of the technique that minimizes the minus influences such as the water quality change caused by the sediment discharge and maximizes the plus influences such as the recovery of the sediment routing system is demanded.

In this paper, the promotion strategy of future reservoir sediment management including such an environmental issue is discussed.

Keywords: *Reservoir sedimentation, reservoir sedimentation management, sustainable water resources management, comprehensive sediment management in the sediment routing system, sediment flushing, sediment bypass, environmental assessment, flushing efficiency*

1. Introduction

The sediment yields of the Japanese rivers are high in comparison with other countries due to the topographical, geological and hydrological conditions. This has consequently caused sedimentation problems to many reservoirs constructed for water resource development or flood control purposes. Under such circumstances, studies on estimation of sediment volume and countermeasures for sedimentation have been conducted since long time ago.

Currently, the reservoir sedimentation management in Japan is embarking on new stages from two points of view. One is, in contrast to the emergent and local conventional countermeasures such as dredging and excavation, the active promotion of introduction of sediment flushing using sediment flushing outlets and sediment bypass systems, which aim at radically reducing the sediment inflowing and deposition. Unazuki dam and Dashidaira dam on the Kurobe River, and Miwa dam on the Tenryu River and Asahi dam on the Shingu River are, respectively, advanced examples of using sediment flushing and sediment bypass techniques, which are placed as permanent measures for sedimentation at dams. The other is, considering a sediment movement zone from mountains through coastal areas, the initiation of a comprehensive approach to recover a sound sediment circulation in the sediment transport system.

However, these advanced techniques for sediment management aiming for long life of dams have only been applied to a limited number all over the world, and therefore continuous study is required. It is also important to solve the social issues, such as consensus building on the need for sediment management throughout the basin people, establishment both of legal system and cost allocation system.

In addition, when the sediment management measures are selected, it is necessary to consider those environmental influences in the river in the advanced country including Japan. In that case, development of the technique that minimizes the minus influences such as the water quality change caused by the sediment discharge and maximizes the plus influences such as the recovery of the sediment routing system is demanded.

In this paper, the promotion strategy of future reservoir sediment management including such an environmental issue is discussed.

2. Necessity of Reservoir Sedimentation Management in Japan

(1) Present State of Sedimentation Problems

Modern development of dams in Japan goes back to approximately 100 years ago. The original targets were mainly the utilization of water for water supply and agriculture purposes. With subsequent economical development, however, several other targets were added to dam development, such as hydropower generation, industrial water use and flood control for mitigating flood damage in the developed cities on the downstream flood plain. At present, the multi-purpose dams make up the majority of the Japanese dams. Approximately 2,730 dams over 15 meters in height have been constructed so far in Japan, but the total reservoir storage capacity is only 23 billion m^3.

On the other hand, from a topographical point of view, many rivers in Japan are very steep which run down over a short distance from mountains of 2,000 to 3,000 meters above sea level to lowland areas. And, from a geological point of view, there are two large faults, the Median Tectonic Line and the Itoigawa - Shizuoka Tectonic Line, where weathering is proceeding in particular (See Figure 1). Weathering is also seen

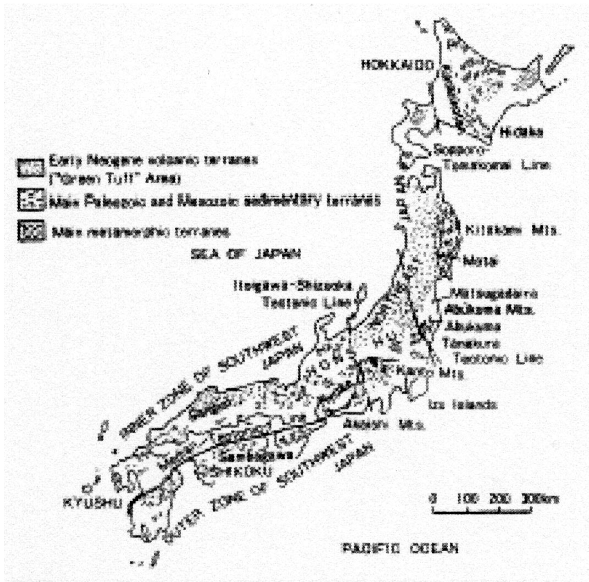

Figure 1 Geology in Japan

in many other regions. The annual average precipitation is approximately 1,700 mm, and, sometimes, intensive rainfall such as 100 mm in an hour or 200 to 500 mm in a day can be recorded. These topographical, geological and hydrological conditions have great impacts on sediment yield in river basins, and consequently reservoir sedimentation has been accelerated especially in Chubu and Hokuriku region which is located in the center of the main island.

Reservoir sedimentation problems in Japan originated with siltation at power plant intakes of the small-scale hydro projects on the mainstream, and then scoring gates used to be set up as a countermeasure. Later, with sedimentation proceeding at middle-scale dams, the increased flood risk caused by sedimentation at the upstream channel of reservoirs became an object of public concern, and the importance of sedimentation management was recognized on a nationwide scale. In the case of the intake facilities of the run-of-river hydropower projects, necessary storage capacity can be secured without difficulty. However, in case of dams for power generation or those for water utilization and flood control, maintaining storage capacity becomes a major issue.

In Japan, sedimentation was taken into account by design sedimentation depth to calculate the silt pressure on dam body in the early years. Later, the idea of sediment storage capacity was specified, for the first time, in the Manual for River Works in Japan made by the Ministry of Construction in 1957. According to the specifications, multi-purpose dams were so planned and constructed as to secure commonly 100 years' design sediment storage capacity in addition to active storage capacity. In case of estimating design sediment storage capacity, various proposed equations (such as on the basis of topography, geology or reservoir capacity) and the actual sedimentation records of neighboring dams or erosion control dams have been referred. However, in some cases, because the amount of inflowing sediment was assumed to be significantly large, sediment storage capacity of less than 100 years (e.g. 30 or 50 years, etc.) was compelled to be used. And in other cases, where 100 years' design sediment storage capacity was able to be secured, the actual sediment yield largely surpassed the originally estimated sediment yield. Consequently, more sediment has already accumulated than the design sediment yield and active storage capacity is decreasing year by year.

(2) Analysis of Sedimentation Data

In Japan, following the widespread recognition of sedimentation problems, all dams having a storage capacity over 1 million m^3 were obliged to report sediment condition to the authority every year since 1980s. As of 2003, from 922 dams accounting for approximately 1/3 of all dams in Japan, annual changes in sedimentation volume and the shape of accumulated sediment were reported. It is probably only Japan that established such a nationwide survey system, and such accumulated data is regarded as considerably valuable records on a global basis.

Figure 2 is the "Sediment yield potential map of Japan" that is made by GIS (Geographical Information System) by using the reservoir sedimentation records and existing geographical features and geological data. The topographic data used here is the "Relief (altitude difference of the highest point and the lowest point in the mesh)" and the "Average altitude" calculated by the "digital national land information" of secondary mesh (about 100km^2) and, in addition, "Relief degree" that considers the distribution of the "Relief".

Regarding geological data, the geologic division decided by hardness of the rock and crack development frequency in consideration of the kind and formation ages of the rock of the basin and the difference of the resistance to the erosion and the collapse etc. Figure is presumption of the specific sediment yield volume obtained thus. Quite a lot of sediment yield volumes were recorded up to several hundreds to thousands m^3/km^2/year in Japan and it is understood that a high rate area is coincided with the Median Tectonic Line and Itoigawa-Shizuoka Tectonic Line by comparing Figure 1.

Figure 3 shows the relationship between reservoir sedimentation rate and years after dam completion. Here, the sedimentation rate is calculated using sedimentation volume to gross storage capacity. Concerning the dams constructed before World War II (ended in 1945) and used for more than 50 years, sedimentation proceeded in the range from 60 to beyond 80 % in some hydroelectric reservoirs. Likewise, for the dams constructed approximately between 1950 and 1960, or from the postwar years of recovery through the high economic growth period, and used for more than 30 years, sedimentation rates beyond 40 % were found in many cases. The influence of sedimentation in those hydroelectric reservoirs depends on the type of power generation. Following this period, meanwhile, large numbers of multi-purpose dams gradually came to be constructed. This type of dams does not have high

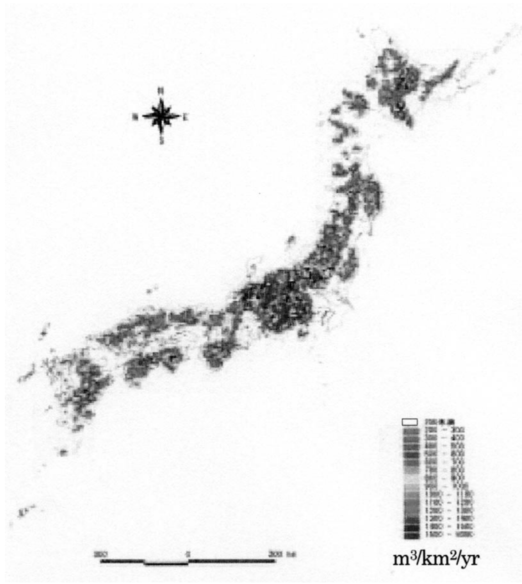

Figure 2 Sediment yield potential map of Japan

sedimentation rates compared to the hydroelectric type, though, the rates of 20 to beyond 40 % were found in some dams. Since maintaining storage capacity is directly linked to maintaining the function of dams such as flood control, the influence of sedimentation in the multi-purpose reservoir becomes large.

Figure 4 shows the relationship between the annual storage capacity loss and the gross storage capacity. The annual storage capacity loss generally decreases with increase in the gross storage capacity and the reservoir life is extended. Figure 5 shows the relationship between the annual storage capacity losses and specific storage capacity (mm), which is defined as ratio of gross storage capacity to catchment area. Many multipurpose dams have specific storage capacities of 50 to 1000 mm and annual storage capacity losses of approximately 1.0 to 0.1 %. In other words, it is noted that the reservoir lives to the gross storage capacities are approximately 100 to 1000 years. On the other hand, in some cases of the hydroelectric dams constructed on the main streams, the catchment areas to the storage capacities are usually very large and the annual storage capacity losses are extremely high.

(3) The Need for Reservoir Sedimentation Management

The need for the reservoir sediment management in Japan can be summarized into the following three points:

1) To prevent the siltation of intake facilities and aggradations of upstream river bed, accompanied by the sedimentation process in reservoirs, in order to secure the safety of dam and river channel.
2) To maintain storage function of reservoirs, and realize sustainable water resources management for the next generation.
3) From a perspective on comprehensive sediment management in a sediment routing system, to release sediment from dams.

The point in 1), as stated above, became major con-

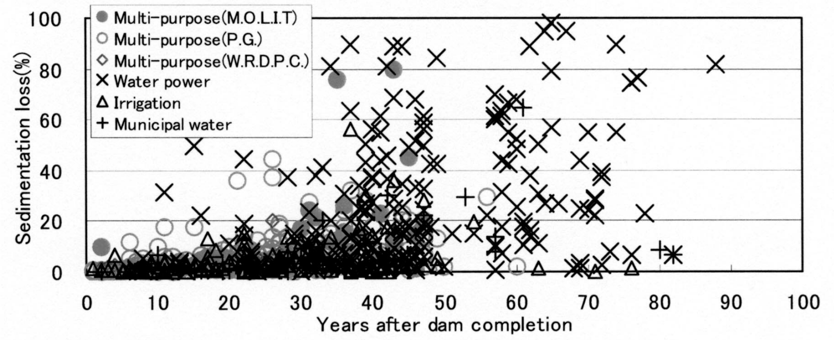

Figure 3 Relationship between reservoir sedimentation rate and years after dam completion

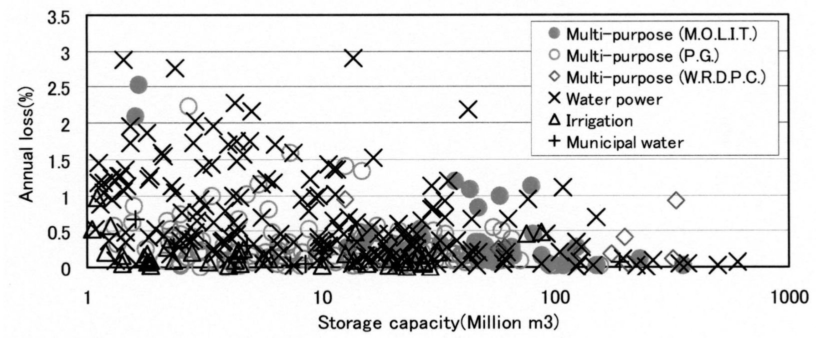

Figure 4 Relationship between annual storage capacity loss and gross storage capacity

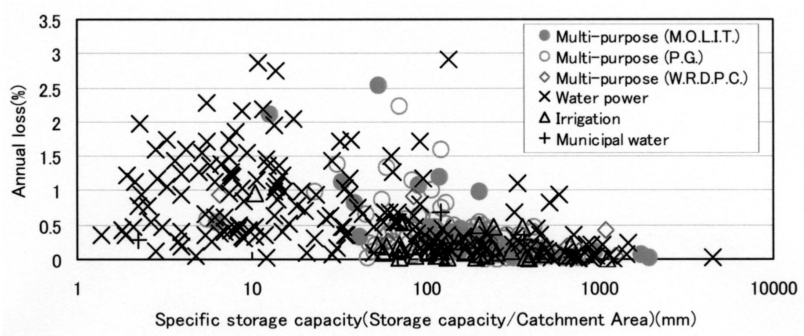

Figure 5 Relationship between annual storage capacity loss and specific storage capacity

cerns in the middle-scale hydroelectric dams constructed in early years and the following measures were then taken: to install sediment scouring gates and locally discharge sediment accumulated in front of the intake facilities, or to use spillway to accelerate traction and discharge of sediment deposited at the end of reservoir.

The point in 2) is an important issue in the future. As shown in Figure 3, the reservoirs in Japan are now facing a critical question of sedimentation. To maintain the existing dams and their facilities over the long term becomes an essential policy issue because of the following reasons: sedimentation is proceeding more than expected in many dams; the share of the dams having a design life of more than 50 years, such as multi-purpose dams where maintaining storage capacity is absolutely necessary, will rapidly increase in the future; and due to recent social changes in environment-conscious trend and an era of low-growth economy, it seems to be difficult to promote new development at the same pace as before.

Here, an average annual capacity loss rate that is obtained by the annual sedimentation survey described in Figure 3 is 0.24 %/year and it is very high up to 0.42 %/year in Chubu region along the Tectonic Lines where a large amount of sediment is produced in the catchment.

The point in 3) represents a new policy in Japan. The amount of sediment supplied from rivers to coasts was radically reduced with construction of erosion control dams or storage dams in mountain areas and acceleration of the aggregate excavation from riverbed after World War II. As a result, various problems rose up including riverbed degradation at downstream channel, oversimplification of river channel, and retreat of shoreline due to the decrease in sediment supply to the coast.

In Japan, Sabo (Erosion and Sediment Control) Plan has been developed from the viewpoint of prevention of sediment disasters such as debris flow. According to this plan, a reference point was set at the exit of the area where sediment was produced in the upstream basin of a river, and the construction of erosion control dam was carried out to control the dischargeable amount of sediment to the downstream region; however, the amount of sediment transported to the downstream region including reservoir area was beyond the scope of the plan.

On the other hand, under the comprehensive sediment management in which the water system is considered consistent, reference points are set at each point, including storage dam and alluvial fan area, in addition to the area of sediment production. Sediment budget at each point is figured out and then a proper amount of sediment to be transported in future is determined on both normal time basis and flood time basis. The important thing here is to focus on the quality of sediment (grain size) as well as the amount, and to recognize the need for the linkage between water management and sediment management considering the indispensability of water for sediment transport.

Following a recommendation by River Council of Japan in 1997, this comprehensive sediment management is now being advanced earnestly. To be more precise, the erosion control dams in the upstream region are planned to be converted to slit dams with notches, which are so designed as to pass, not to trap, as much fine sediment carrying less risk of sediment disaster as possible. For storage dams, sediment bypass or sediment flushing outlets are also progressively added in order to reduce sedimentation and accelerate sediment discharge to the downstream and, at the same time, an attempt to return the excavated and dredged sediment to the downstream river has been undertaken. The influence of storage dams on the sediment routing system is extremely huge, and therefore it is highly meaningful to reduce sediment trap there by means of appropriate sediment management.

As stated above, reservoir sediment management is an unavoidable issue in Japan. The level of importance depends on each dam and water system, and the establishment of techniques to trap less transported sediment and discharge as much as possible to the downstream is required. The next section describes the present situation and future issues on reservoir sedimentation management in Japan.

3. Reservoir Sedimentation Management in Japan

Sediment management in reservoirs is largely classified into the three approaches: 1) to reduce sediment inflow to reservoirs. 2) to route sediment inflow so as not to accumulate in reservoirs, and 3) to remove sediment accumulated in reservoirs. Figure 6 shows how sediment management is undertaken and classified in Japan. In Figure 7, dams in the Japanese whole country were plotted by the parameter of the turnover rate of water (CAP/MAR=Total capacity/Mean annual runoff) and sediment (CAP/MAS=Total capacity/Mean annual inflow sediment). It is thought that the selected sediment management measures are changed by these two parameters. This is described later.

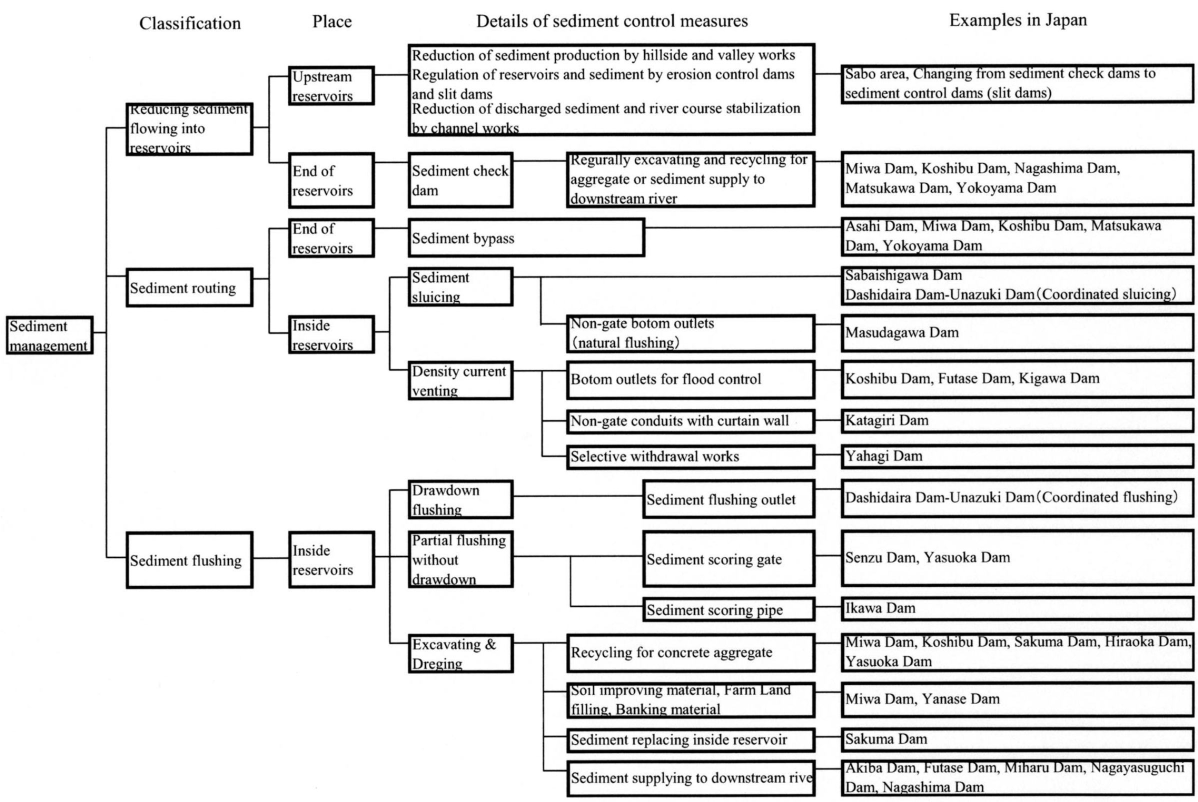

Figure 6 Classification of Reservoir Sedimentation management

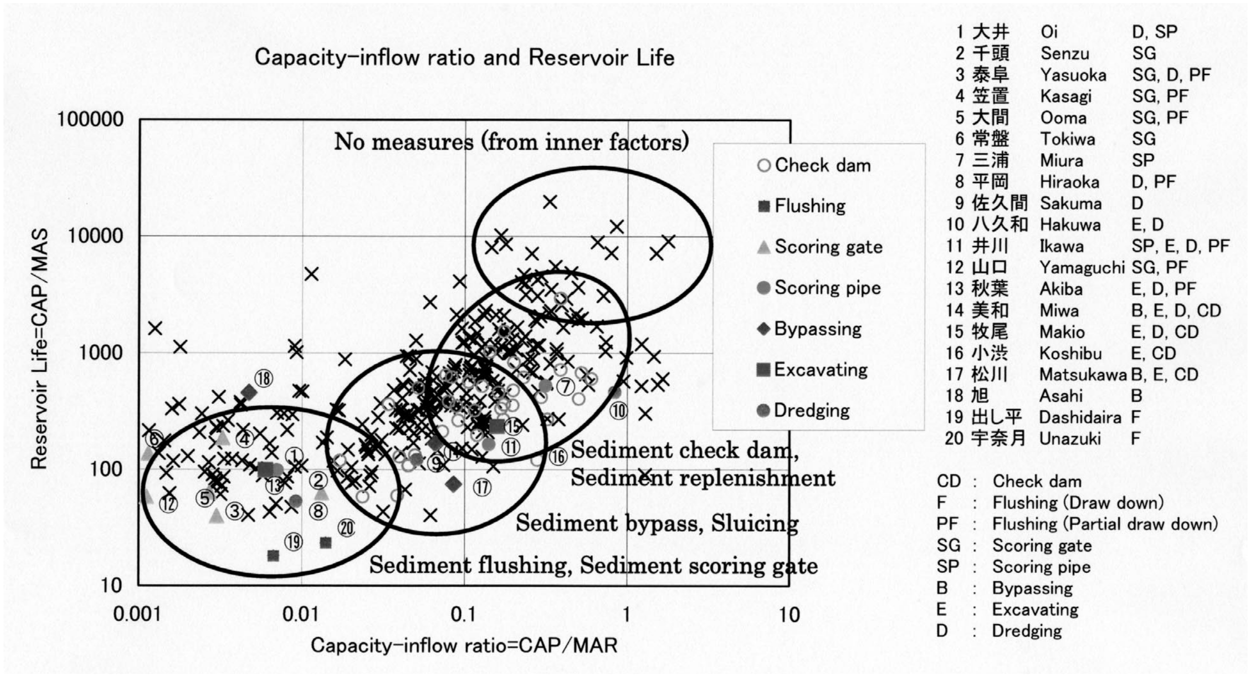

Figure 7 Representative sediment control examples in Japan
(Relationship between capacity-inflow ratio and reservoir life)

(1) Reduction of Sediment Inflow into Reservoirs

There are two techniques to reduce the amount of transported sediment: 1) countermeasure to control sediment discharge which covers entire basin including the construction of erosion control dams; and 2) countermeasure to forcibly trap sediment by constructing check dams at the end of reservoirs. Although the catchment areas of dams have high forest cover rates, a remarkable amount of sediment is produced in the watershed where landslides frequently occur due to the topographical and geological conditions. Some other factors are also contributing to an increase in sediment yield: Management system is complicated because the boundaries between national forest land and private property are intricate; natural hardwoods have been intentionally converted to softwoods of commercial value; and forest road construction has been implemented for forest management work. Especially some mountain streams, where landslides frequently occur and a large amount of sediment is produced, are designated as the areas for erosion and sediment control, and countermeasures are taken to control sediment discharge, such as the construction of erosion control dams. When sediment yield is also expected from side slopes surrounding dam reservoirs, a project to buy and preserve a certain plot of forest as a greenbelt has been implemented by dam administrator itself.

On the other hand, an attempt to trap sediment using check dams is found effective for the reservoirs where bed load of relatively coarse grain size accounts for a large percentage of sediment inflow, so recently many dams have proceeded in constructing them. In this technique, a low dam is so constructed at the end of reservoir as to deposit transported sediment, and then the deposited sediment is regularly removed. The accumulated sediment can be excavated on land except for flood time, and the removed sediment is utilized effectively as concrete aggregate. As of 2000, the check dams have been constructed at 57 out of the dams under jurisdiction of Ministry of Land, Infrastructure and Transport.

Figure 8 shows the longitudinal profile of Koshibu reservoir. The upstream check dam (overflow section: 65 m wide x 10 m high) which was constructed in 1978, is shown in Photo 1, and a private plant for taking aggregate in Photo 2.

Recently, sediment replenishment tests have been carried out in some dams. Trapped sediments in the sediment check dam upstream of the reservoir are excavated and transported to the downstream of the dam. These sediments are put on the downstream river channel temporarily and washed out by the natural flood flows. Nagashima dam, Miharu dam, Akiba dam, Futase dam, Shimokubo dam, Urayama dam, Hachisu dam, and a Nunome dam etc. are typical cases. Moreover, there is an example of combining with the environmental flushing flow such as in the Managawa dam and the integrated improvement of the river environment is expected by the flood disturbance and the sediment supply.

(2) Routing of Sediment Inflow into Reservoirs

Another possible approach to sediment management, next to the reduction of sediment inflow itself, is to route sediment inflow so as not to allow it to accumulate in reservoirs. In Japan, the following techniques are adopted: 1) sediment bypass by directly diverting sediment transport flow, and 2) density current venting by using a nature of high-concentration sediment transport flow.

In Japan, it is sediment bypass tunnels that have

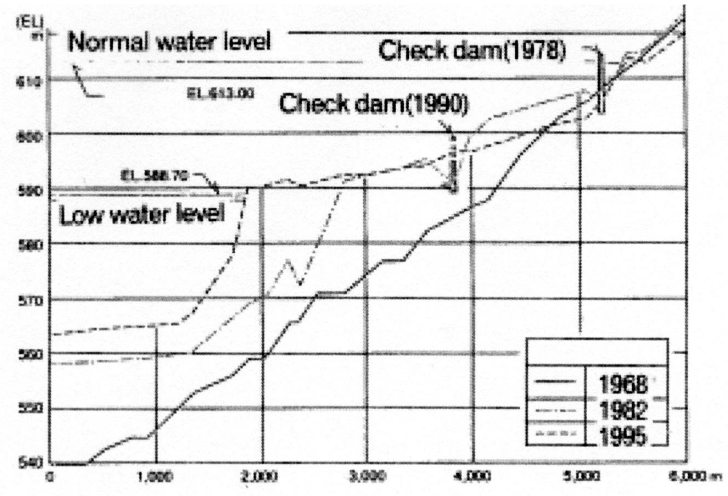

Figure 8 Longitudinal profile of Koshibu reservoir

Photo 1 Upstream sediment check dam of Koshibu dam (Constructed in1978)

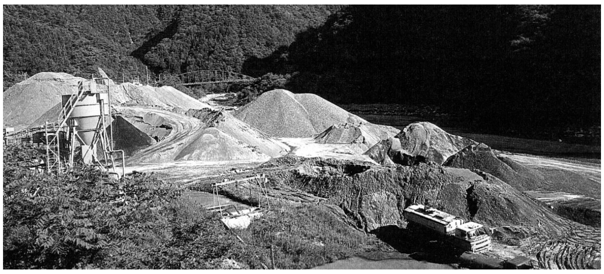

Photo 2 Private plant for taking aggregate

been studied most exhaustively. Although this technique involves high cost caused by tunnel construction, it is also applicable to existing dams; it does not involve drawdown of reservoir level and therefore no storage capacity loss; and it has relatively small impact on environment because sediment is discharged not so rapidly as sediment flushing, which is described later. Nunobiki dam is an initial example of the bypass tunnel in Japan. The reservoir to which longevity is estimated to be only 25 years without bypass is prolonged to over 1000 years. Recently, the effect is clarified since the bypass completion of the Asahi dam, and the planning and construction of the bypass have been advanced also in the Miwa, Matsukawa and Koshibu dam.

The subjects of designing sediment bypass tunnels are to secure the safety of sediment transport flow inside tunnels and to take countermeasures for abrasion damages on the channel bed surface. Among factors that significantly relate to these problems are grain size, tunnel's cross-sectional area, channel slope, and design velocity.

Table-1 shows some examples of existing sediment bypass tunnels and the ones under construction and study. It should be understood that design condition becomes increasingly hard if higher velocity and larger grain size will be expected.

Density current venting, on the other hand, is a technique to use a nature of high-concentration sediment transport flow, which runs through relatively deep reservoir with original channel bed of steep slope as a density current with less diffusion, and to discharge it effectively through outlets in timing of reaching dam. In both techniques, the main target is fine-grained sediment such as suspended sediment and wash load. In the multiple-purpose dams in Japan that usually have high-pressure bottom outlets for flood control, the effective operation of these facilities during flood season can increase a chance to actively discharge fine-grained sediment.

In addition, as countermeasures for long-term turbid water discharge problem, selective withdrawal works are installed at many dams. And, in Katagiri dam at Tenryu River, curtain wall is installed in front of a non-gate outlet conduit to discharge water from the bottom layer of reservoir. Discharge of fine-grained sediment using these facilities can also be classified in density current venting.

On the other hand, it is possible to reduce the impounded water volume of the reservoir by greatly draw downing and emptying during a certain period of the flood season with a lot of sediment inflow to promote the sediment passing thorough the reservoir. These operations are generally called sediment sluicing and there are a lot of adoption cases in China and Taiwan. The Sabaishigawa dam where reservoir water level is regularly drawn down in the snow-melting season corresponds to this case in Japan.

Table-1 Sediment Bypass Tunnels in Japan and Switzerland (Sumi 2004)

Name of Dam	Country	Tunnel Completion	Tunnel Shape	Tunnel Cross Section (B×H(m))	Tunnel Length (m)	General Slope (%)	Design Discharge (m^3/s)	Design Velocity (m/s)	Operation Frequency
Nunobiki	Japan	1908	Hood	2.9×2.9	258	1.3	39		
Asahi	Japan	1998	Hood	3.8×3.8	2,350	2.9	140	11.4	13 times/yr
Miwa	Japan	Under construction	Horseshoe	2r = 7.8	4,300	1	300	10.8	-
Matsukawa	Japan	Planning	Hood	5.2×5.2	1,417	4	200	15	-
Egshi	Switzerland	1976	Circular	r = 2.8	360	2.6	74	9	10days/yr
Palagnedra	Switzerland	1974	Horseshoe	2r = 6.2	1,800	2	110	9	2～5days/yr
Pfaffensprung	Switzerland	1922	Horseshoe	A = 21.0m^2	280	3	220	10～15	～ 200days/yr
Rempen	Switzerland	1983	Horseshoe	3.5×3.3	450	4	80	～14	1～5days/yr
Runcahez	Switzerland	1961	Horseshoe	3.8×4.5	572	1.4	110	9	4days/yr

In addition, the request to the flood control has risen relatively in the river basin. Then, the number of dams only for the flood control that has the gateless conduit which can also sluice inflow sediment during flood events increases. Figure 9 shows an example of the Masudagawa dam. The reservoir is emptied through year and it is paid attention as a dam without reservoir sedimentation.

If such an idea is advanced further, the idea of "Separated dam project for the flood control and the water use" where the dam only for the flood control is constructed in the main stream without sediment management and the water use capacity is secured in the tributary stream separately is worth the examination, too. This is an examination problem in relation to the fact of rearranging capacity mutually so that it may become easy to manage reservoir sediment not only in the new construction project but also in the reorganizing project whole in the river basin in the future.

(3) Removal of Sediment Accumulated in Reservoirs

This approach is regarded as a last resort in case sediment is accumulated in reservoirs in spite of various efforts being done: 1) mechanically excavating sediment accumulated in the upstream region of reservoirs, 2) dredging sediment accumulated at the middle and downstream regions, and 3) flushing out sediment with tractive force. As for excavation and dredging techniques, it is important that the removed sediment should be treated properly and reused.

On the other hand, sediment flushing is a technique to restore tractive force in a reservoir beyond its critical force by means of drawdown of reservoir level, and flush the deposits though bottom outlets in the dam body with inflow water, mainly in an open channel flow condition, to the downstream of dam. When the amount of sediment inflow is significantly large, man-powered techniques such as excavation and dredging are hard to be adopted because of problems involving transportation and dump site. In such a case, however,

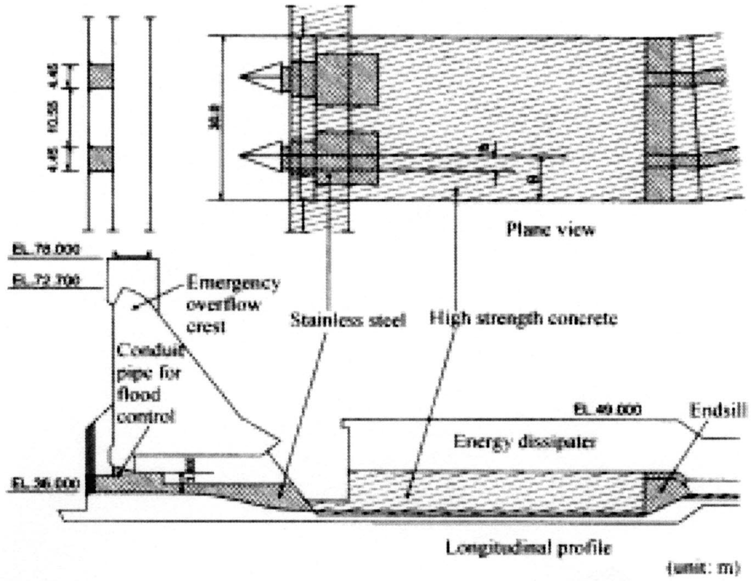

Figure 9 Masudagawa dam – Natural flushing Non-gate dam (Kashiwai 2000)

sediment flushing can be a permanent measure if conditions are met. Traditionally in Japan, sediment flushing facilities such as flushing sluices and outlets were installed at small-scale hydroelectric dams or weirs for the purpose of discharging sediment deposited in the vicinity of intake. In contrast, at Dashidaira-Unazuki dams in the Kurobe river, where a large amount of sediment is discharged, sediment flushing is implemented in coordination of upstream and downstream dams (coordinated sediment flushing).

Sediment flushing is performed at many dams all over the world, as shown in Table-2. The necessary conditions for dams' adopting sediment flushing are; they are equipped with bottom outlets (sediment flushing outlets) through which reservoir level is drawn down and flowing water can be discharged in an open channel during sediment flushing; sufficient amount of water is secured for a series of operations of reservoir level drawdown, open channel discharge and reservoir refill. Sediment flushing is considered as an extremely effective technique for discharging sediment in terms of harnessing tractive force in natural river channel. However, when this technique is introduced, an extensive study is required in the planning stages, considering such conditions as inflow, sediment inflow, storage capacity, grain size distribution and reservoir operation. At the same time, it is also required to consider measures concerning environmental problems under sediment flushing process.

Moreover, HSRS (Hydro-suction Sediment Removal System) where can intake and discharge sediment using only the water level differences without the mechanical force is developed in some types in recent years. There are stationary and movable types in the system. In case of the stationary type, establishment of the measures to move sediment to the system neighborhood in the reservoir and, in case of the movable type, securing enough operation time corresponding to the target sediment volume to be discharged during a year and a safe work environment are problems to be solved.

4. Promotion strategy of reservoir sedimentation management

The problems to promote such reservoir sedimentation management in future are 1) Priority evaluation of reservoirs where sediment management should be introduced, 2) Appropriate selection of reservoir sediment management strategies and 3) Development of efficient and environmental compliance sediment management technique.

(1) Priority evaluation of reservoirs where sediment management should be introduced

The World Bank is advancing RESCON project (Reservoir Conservation Project) (Palmieri 2003). It will be thought that reservoir sedimentation problem becomes a key issue while putting the target on the redevelopment project in the future though the World Bank has financed the developing country only to the new construction projects. For instance, when there are several dams in the same water system, estimated each reservoir life when measures are not taken is concretely presented. And, if it is understood that remain-

Table-2 Sediment flushing dams in the World

Name of Dam	Country	Dam completed	Dam Height (m)	Initial Storage Capacity (CAP) (million m^3)	Mean Annual Sediment Inflow (MAS) (million m^3)$^{1)}$	1／(Mean Annual Runoff) (=CAP/MAR)	Reservoir Life (=CAP/MAS)	Average Flushing Discharge (m3/s)	Flushing Duration (hrs)	Flushing Frequency (1/ yr)
Dashidaira	Japan	1985	76.7	9.01	0.62	0.00674	14.5	200	12	1
Unazuki	Japan	2001	97	24.7	0.96	0.014	25.7	300	12	1
Gebidem	Switzerland	1968	113	9	0.5	0.021	18.0	15	70	1
Verbois	Switzerland	1943	32	15	0.33	0.00144	45.5	600	30	3
Barenburg	Switzerland	1960	64	1.7	0.02	0.000473	85.0	90	20	5
Innerferrera	Switzerland	1961	28	0.23	0.008	0.00018	28.8	80	12	5
Genissiat	France	1948	104	53	0.73	0.00467	72.6	600	36	3
Baira	India	1981	51	9.6	0.3	0.00489	32.0	90	40	1
Gmund	Austria	1945	37	0.93	0.07	0.00465	13.3	6	168	N.A.
Hengshan$^{2)}$	China	1966	65	13.3	1.18	0.842	11.3	2	672	2～3
Santo Domingo	Venezuela	1974	47	3	0.08	0.00667	37.5	5	72	N.A.
Jen-shan-pei^{2}	Taiwan	1938	30	7	0.23	N.A.	30.4	12.2	1272	1
Guanting	China	1953	43	2270	60	1.5	37.8	80	120	N.A.
Guernsey	USA	1927	28.6	91	1.7	0.0433	53.5	125	120	N.A.
Heisonglin	China	1959	30	8.6	0.7	0.6	12.3	0.8	72	N.A.
Ichari	India	1975	36.8	11.6	5.7	0.00218	2.0	2.16	24	N.A.
Ouchi-Kurgan^{2}	Former USSR	1961	35	56	13	0.00376	4.3	1000	2400	N.A.
Sanmenxia$^{2)}$	China	1960	45	9640	1600	0.224	6.0	2000	2900	N.A.
Sefid-Rud$^{2)}$	Iran	1962	82	1760	50	0.352	35.2	100	2900	N.A.
Shuicaozi	China	1958	28	9.6	0.63	0.0186	15.2	50	36	N.A.

1) Average after dam completion,　2) Sluicing dams

der of the life is very short, the feasible sediment management measures considering economy etc. is concretely proposed.

It will be thought that such an approach is important also in Japan. Especially, when several dams exist in the water system, it is realistically difficult to introduce the sediment management all together from the limit of the budget under the present situation. Then, it is necessary to evaluate priority according to some indices. Then, I want to propose the following three points as an index that evaluates the priority of the sediment management.

The first point is an inner factor concerning the sustainability of the dam, and has two indices. In one, the excess extent (sedimentation speed magnification) at the actual sedimentation speed to the planning speed for a hundred years, and another one is the reservoir life (CAP/MAS) of ahead. Among these, as for the sedimentation speed magnification, about 1/4 in the entire dam where sedimentation volumes are annually investigated is twice or more, and the priority of such a dam is high. On the other hand, if reservoir lives of multipurpose dams in Japan are evaluated, 1000 years or more is 34%, and 500-1000 years is 25%, and 100-500 years is 34%, and 100 years or less is 7%, and 400-500 years is the average. From this, for dams which have below the half of the average reservoir life such as CAP/MAS<200, it is necessary to materialize some sediment management strategies immediately in the future. In this case, if the reservoir life of 1000 years or more which is realized at the Nunobiki dam is made as a target, selection of the applicable measures and evaluation after the project execution become clearer.

The second point is an external factor concerning the continuity of sediment routing system such as the impact degree to the downstream river and an actual environmental deterioration degree there. As for the impact to the downstream river, height of a dam, a turnover rate of the reservoir (1/(CAP/MAR)) and the river extension until the major river is joined become indices. On the other hand, armoring of the river bed, the immobilization of the sand bar, degradation and immobilization of the channel, decreasing the gravel bed surface area and increasing the grove area in the actual downstream river channels become indices as an environmental deterioration degree of the downstream river.

The third point is a technical difficulty viewpoint when the sediment management of each dam is introduced. Height of dam, the turnover rate of the reservoir, the reservoir extension, and the average annual sedimentation volume generally become indices though technical difficulty is different depending on the sediment management measures.

Priority can be evaluated comprehensively by combining the above-mentioned three points and putting the appropriate weight for each index. As the trial, the priority was evaluated for the seven existing dams of the Yodogawa river system and the dam where the sediment management strategy should be studied previously became clear.

(2) Appropriate selection of reservoir sediment management strategy

If the dam where priority is high is selected, it is necessary to select a concrete sediment management strategy. In Figure 7, existing sediment management practice dams were specified referring to the parameter of the turnover rate of water and sediment. It is understood that measures actually selected have changed in order of the sediment flushing, the sediment bypass, sediment check dam and excavating, and dredging as CAP/MAR increases (decrease in the turnover rate) roughly. This is because of greatly depending on the volume of water to be able to use the sediment management measure that can be selected for the sediment transport. Here, the quality of sediment (size etc.) and the river environment conditions which may restrict the sediment discharge are not considered.

Here, it is important to clarify the range that the sediment flushing and the sediment bypass that uses the tractive force for the sediment discharge can be applied for selecting the sediment management strategy. Especially, it is a trade-off in the sediment flushing between maximizing sediment discharge and minimizing environmental impact on the downstream river. In a Kurobe river, Dashidaira dam in Japan that is a typical sediment flushing case, the sediment was discharged for the first time in 1991. The sediment flushing of 13 times in total was carried out by June, 2005, and the sediment of six million m3 or more that equaled 2/3 or more of the total reservoir capacities in total was discharged as in Figure 10.

In addition, after completion of Unazuki dam downstream of Dashidaira dam in 2001, a coordinated sediment flushing of these dams have started and the sediment flushing of five times in total and sediment sluicing of four times are executed up to July, 2005. The sediment flushing is executed at the first major flood event every year and the sediment sluicing is done at the successive bigger ones by the similar sediment flushing operation preventing additional sediment deposit in the reservoir.

A present sediment flushing operation in the Kurobe river assumes 'Execute it to maintain a constant bed form without storing sediment in the reservoir as much as possible at the natural flood events between June and August' to be a principle. As a result, the phenomenon of the water quality deterioration because of the sediment flushing at first in 1991 is not seen, and it contributes to the maintenance of the capacity of the reservoir greatly. The rule that executes the sediment flushing at constant frequency so that the interval should not become long according to this natural flood agrees with the finding in Switzerland and France that has longtime results for the sediment flushing, and becomes a good reference to promote the reservoir sedimentation management in the future very much.

Next, it is a very much interest how much efficiencies were obtained by these sediment flushing operations. There are not a lot of dams surveyed in detail

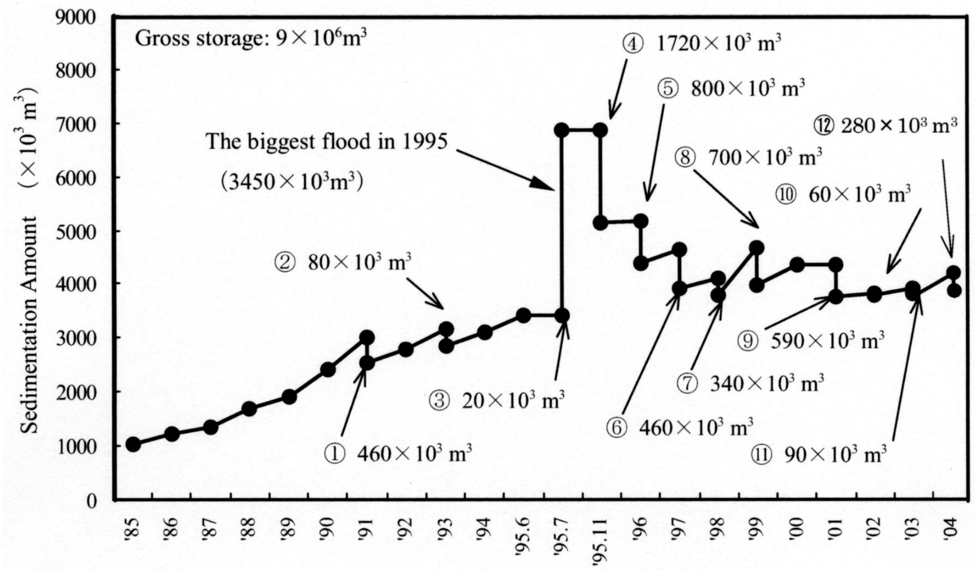

Figure 10 Sedimentation volume change in Dashidaira dam

about usual results among sediment flushing dams listed in Table 2. Then, the relation between the water consumption and the amount of the sediment flushing was shown in Figure 11 about four dams, Dashidaira dam in Japan, Gebidem and Verbois dams in Switzerland and the Baira dam in India, where each flushing result was recorded. Here, the water consumption for the sediment flushing is only calculated during the fully draw down period though the sediment discharge actually starts from fine materials during the reservoir drawdown period and this water volume should be included in the water consumption.

Figure shows sediment flushing efficiency (F_e $=S/W$) calculated by the sediment volume and the water consumption. Among these, the sediment flushing efficiency in Gebidem dam is comparatively high since the sediment flushing is executed with a low flow discharge for a long time. Moreover, in Baira dam flushing efficiency is also comparatively high though there are some fluctuations. On the other hand, since Verbois dam is located at the mainstream of the Rhone River and the sediment flushing is executed with a large amount of water from Lac Leman, the sediment flushing efficiency is not large. In Dashidaira dam, the sediment flushing efficiency is not so high but close to Verbois dam except the sediment flushing after the Big flood in 1997 though there are great fluctuations in the amount of the sediment flushing and the water consumption.

In these four dams, the sediment flushing is strictly

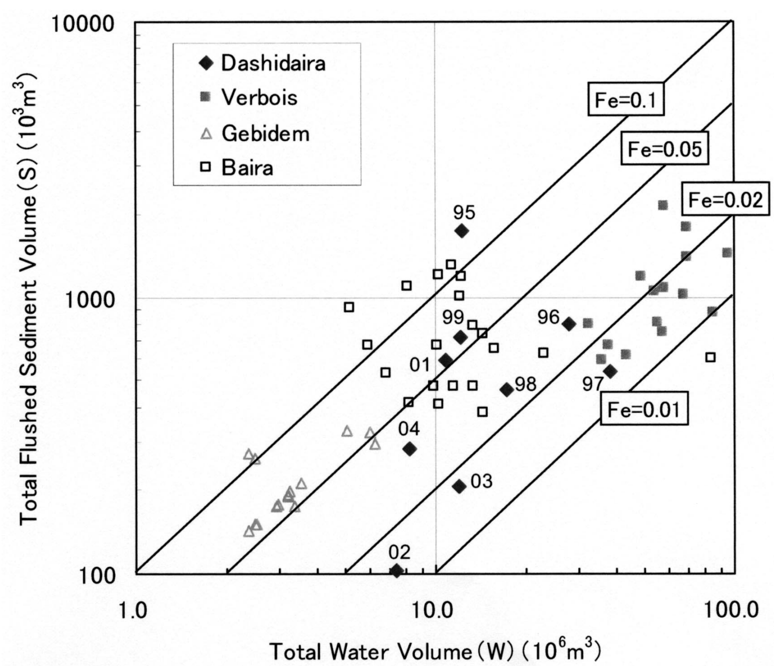

Figure 11 Total water volume and flushed sediment volume in sediment flushing dams

managed in Dashidaira dam and Verbois dam so as not to cause a remarkable water quality changing to the downstream river by maintaining considerably a lot of water compare to the amount of sediment. As a result, in consideration of the downstream river environment, an enough volume of water is required to be used so that the sediment flushing efficiency may come to be suppressed. In the sediment flushing of the Kurobe River, the sediment flushing is executed by securing enough river discharge just after the natural floods and also the additional discharge is recently examined to wash the fine sediment silted in the downstream river channel after the sediment flushing, and thus the sediment flushing is using more volume of water.

Next, the sediment flushing efficiency of other dams is shown in Figure 12. The sediment flushing efficiency is about roughly F_e=0.01-0.15, and it is thought that it is necessary to suppress it to about F_e =0.05 or less when consideration to the river environment is especially necessary. According to the research on the feasibility evaluation of the sediment flushing, a possible range of the sediment flushing can be obtained by the following equation by using the parameter shown in Figure 7 (Sumi 2000). Here, the sediment flushing efficiency and the proportion of the water consumption by the sediment flushing to the mean annual runoff volume (MAR) are defined F_e and β respectively.

$$\frac{CAP}{MAS} > \frac{\dfrac{CAP}{MAR}}{F_e\left(\beta - \dfrac{CAP}{MAR}\right)} \qquad (1)$$

In Figure 13(a) and (b), possible range of the sediment flushing in the case where F_e changes to 0.01, 0.02 and 0.05 with the fixed β=0.1, and in the case where β changes to 0.05, 0.1, 0.2 with the fixed F_e =0.2 are shown respectively. Possible ranges are shown in the left side of each line. According to these, the change in F_e mainly influences within the small range of CAP/MAS and even a small turnover rate of the reservoir, e.g. large CAP/MAR, becomes a possible rising of F_e under β constant. If the river environment conservation is considered, possible range of the sediment flushing becomes narrower because it should estimate F_e low. On the other hand, if β can be enlarged, the sediment flushing possibility will extend under the same F_e since the water volume ratio that can be used for the sediment flushing increases. However, β and the original storage purposes of the reservoir is in the relation of the trade-off and it is not possible to adopt a too big value.

It is also necessary to advance the examination of coverage in the sediment bypass as well as the sediment flushing. In this case, it is necessary to estimate the sediment sluicing efficiency that can be achieved in the sediment bypass as well as the sediment flushing efficiency and to study how much water volume can be used for bypassing and how the operation rule can be defined become problems. Though it is necessary to consider the downstream river environment for the sediment bypass, it is thought that it doesn't become a big problem since only it is to pass what flows into the reservoir from upstream basically. The operation results of the sediment bypass are limited, and anyway, it will be necessary to accumulate these data in the future and to advance the examination. Moreover, the verification work is also necessary though sediment sluicing is roughly assumed that its coverage is similar to the sediment bypass.

(3) Development of efficient and environmental compliance sediment management technique

Finally, it is necessary to develop wide sediment management measures other than the sediment flushing and the sediment bypass. Especially, the appropriate sediment management measure to the area where the coverage of sediment bypass and sediment sluicing is exceeded in Figure 7. In this area, the necessity intended for a large amount of sediment can be a little, and the sediment check dam and it's regularly excava-

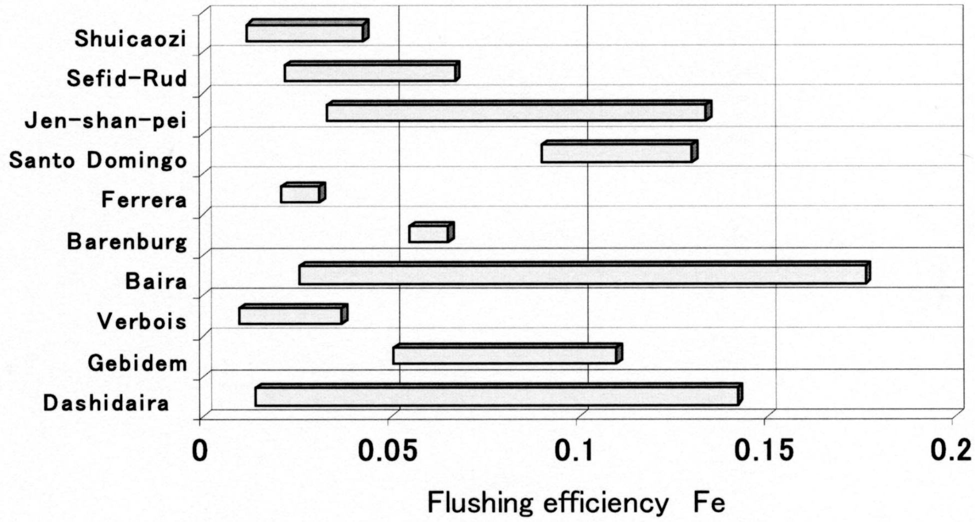

Figure 12 Sediment flushing dams and flushing efficiency

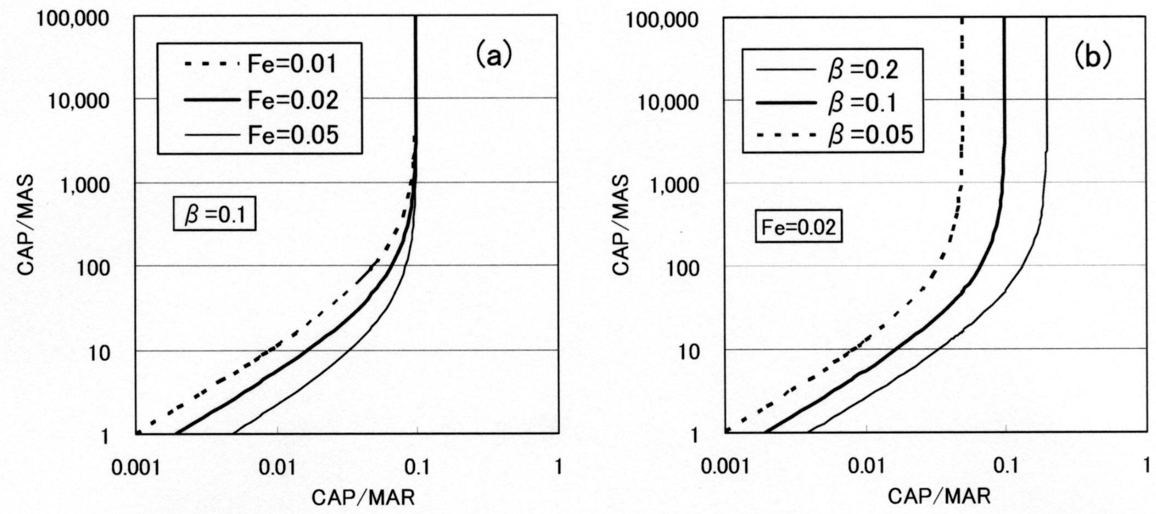

Figure 13 Possible range of the sediment flushing
((a) Proportion of the water consumption to the mean annual runoff
volume β is fixed; (b) Sediment flushing efficiency F_e is fixed)

tion can be applicable. However, its effect doesn't continue because of the insufficient maintenance of the check dam in the past and a fundamental solution is hoped for. The sediment replenishment measure is the one of them and the technological development expected in this area is summarized into "Take it", "Transport", and "Discharge" technologies.

Regarding "Take it" technology, more development of the Hydro Suction Sediment Removal System (HSRS) is expected very much. A current problem is thought that the establishment of the way of positively adopting it by promoting such a new technology to the practical level.

Regarding "Transport" technology, it is necessary to examine other transportation methods by the situation as though it is the most natural to use the stream of the river. In that case, the belt conveyer transportation, capsule transport, and hydraulic transportation of slurry, etc. can be applied besides usual trucking if the condition is suitable.

Regarding "Discharge" technology, it is necessary to develop the technology that discharges sediment safely to the river from the view points of a local piling up problem of the coarse sediment in the downstream river channel and the water quality problem of the generation of a high turbid water by the fine sediment and so as not to become the trouble of the sediment management.

5. Conclusions

It has passed for a while being recognized the necessity of the integrated integrated sediment management of the sediment routing system. It is securing of the continuity of the sediment mobility. The reservoir sediment management especially occupies the important position among them. The dam is a property of a society, and it is necessary to aim at sustainable use by appropriate reservoir sediment management without doing to disposable. It is an extremely important point for the idea of "Intergenerational equity" that doesn't turn the load of sediment measures to future generations not to postpone the reservoir sedimentation management. The reservoir sedimentation problem is given to one of the reasons why the new dam construction is criticized. It is likely to come at time when the dam project of the type of the 21st century and the ideal way of the dam management are presented.

Reservoir sedimentation management in Japan is entering a new era. Although there still exist technical problems to be solved, we believe that the importance of pursuing sediment management will increasingly grow. Assessing issues, depending on each case, of dam security, sustainable management of water resources and sediment management in a sediment routing system, we have to draw up an effective sediment management plan with a limited budget and take specific action. Needless to say, of course, our best endeavors should be exerted to minimize negative environmental impacts involved in sediment management.

Finally, I'd like to emphasize the technologies for the reservoir sedimentation management is very much paid attention not only in Japan but also in the world. Reservoir sedimentation problem has been a challenging issue for many countries all over the world, however only a limited number of countries have been actively playing roles in reservoir sedimentation management. Under the present situation, for those countries which have progressively developed various techniques and possess example cases, key questions are not to put off dealing with this problem but to make continuous effort of further developing techniques and putting them into practice, and to widely share the resulting information and knowledge with every country in need of them.

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Sedimentation and Control Method of Lubuge Reservoir

Mei Zhihong
China Hydropower Kunming Investigation Design and Research Institute, Kunming, China

Abstract

This paper is mainly concerned with sediment deposits and reservoir operation of the Lubuge Reservoir. Firstly, field data since water storage was analyzed and sediment deposit properties of the reservoir were studied. Through the comparison with the status of sediment deposition of the reservoir, operation method of sediment control in the original design was assessed. By using 1D numerical model of non-equilibrium sediment transport, sediment deposit process in the reservoir was predicted. And water level of flood season to optimize the sediment operation of the reservoir was proposed. This result can give a scientific reference to compromise better electric power generation and control of reservoir deposits. Finally, considering the precondition of the rate of water lowering limited by stability requirement of landslide mass in the reservoir, the deposits flushing mode and its effect by lowering water lever of the dam was studied.

Key words: *reservoir, sedimentation, sediment control method*

1. INTRODUCTION

Lubuge Power Station is a cascade of Huangnihe river, which is a branch river of Zhujiang river. The task of exploitation is to generate electricity, and the installed capacity of the electric power station is 600MW. The rock dam is 103.8m high, and the elevation is 1138 m. The water tunnel is 9378 m long, and the discharge is 214 m^3/s, using the hydraulic current of 372.5m high.

The dam site of Lubuge reservoir controls the drainage area about 7300 km^2, occupying 88.3 percent of the drainage area of Huangnihe river. The reservoir storage water level is 1130 m, and at this level the length of reservoir is 19.4km, with the storage 1.11×10^8 m^3. The dead water level is 1105 m, with the 16km and the storage 0.37×10^8 m^3. The reservoir is a typical mountain stream reservoir, as the hillsides of the two banks are steep, with no large tributaries and very narrow valley.

2. DESIGNED OPERATION METHOD OF SEDIMENT CONTROL

2.1 Principle of sediment control

The capacity of Lubuge Reservoir is small, and the incoming sediment load is great. The ratio of total storage capacity and incoming sediment load is 35, while this value is 11 between dead storage and the incoming sediment load. In order to control the sedimentation and keep the regulating storage for long time, based on the characteristics that the incoming sediment load focuses on flood season with much discarding water when generating electricity, it is demanded that the water level be reduced for sand flushing during this period. The designed water level for sand flushing in flood season is 1105m (the same as the dead water level), causing most of the sedimentation in dead storage and decreasing the sedimentation in regulating storage. Meanwhile, the reservoir is emptied for sand flushing every other 3 or 5 years, resuming some dead storage. Then, using the dead storage to finish solid flow regulation, make the elevation of the sedimentation before dam lower and decrease the sediment concentration across the generator.

2.2 Usage of flood discharge and sand flushing establishments

The constructions of flood discharge and sand flushing is composed of spillway, spillway tunnel of left bank, spillway tunnel of right bank and flushing tunnel.

The flushing tunnel is below the intake of the power station, and the most discharge is 257m^3/s at the level of 1105m. The gate is small but flexible for opening and closing, so it can be usually opened to regulate water level little. And using the discarding water to flush in time, can diminish the sediment concentration across the generators.

The combined discharge of spillway tunnels at two banks and flushing tunnel achieves 2753m^3/s, equal to 10% of the peak flow.

The spillway tunnels can control the water level, and also empty the reservoir to flush. Especially, the floor elevation of the right tunnel is 1060m, only 10m higher than original river bed, so the right tunnel is the main passage for sand flushing.

3 RUNNING OF LUBUGE RESERVOIR IN PRACTICE

3.1 INFLOW DISCHARGE AND SEDIMENT

Chajiang hydrologic station is located at the backwater area of Lubuge Reservoir, which controls the drainage area about 6950km^2. Through the field data from 1988 to 2003, mean annual discharge is 144m^3/s, mean annual sediment transport rate is 123kg/s, mean annual sediment load is 387×10^4t, mean annual sediment concentration is 0.854 kg/m^3.

The field data of Chajiang hydrologic station has following characters:

(1) The inflow sediment is highly concentrated, and the sediment peak is well corresponding to the flood peak:

In flood season, the inflow discharge is 79.3 percent of that in the total year, and sediment discharge is 95.9 percent. The maximum monthly-mean values of sediment discharge and sediment concentration is in June, while the minimum is in October.

The field data of maximum daily averaged sediment transport rate is 23840kg/s on July 15, 1997. The daily averaged discharge of the same day is 2020m^3/s, which is the second among the observed discharges.

(2) The year-to-year sediment load variation is large.

The largest yearly sediment load is 960×10^4t in 1994, while the smallest yearly sediment load is 132×10^4t in 1989, and the ratio of these two values is 7.27.

Obviously, in the designed principle for controlling sediment, the feature is fully considered that the inflow sediment load is highly concentrated in flood season. In flood season, making the water level lower for sand flushing can control the quantity and location of the deposition in the reservoir in effect.

Compared with the designed sediment load, the inflow sediment load increases obviously. From 1988 to 2003, the mean annual sediment load is 387×10^4t, increasing 12.5 percent more than the designed value 344×10^4t.

3.2 RUNNING WATER LEVEL

In practice, if the water level in flood season works near the dead water level 1105m according to the design strictly, the following problems exist:

(1) If the water level in flood season maintains 1105m, no regulating storage will exist. When the inflow discharge is less than the discharge for generating electricity, the electricity power output can not satisfy the demand of the electric network.

(2) Gate is opened and closed frequently, the accident probability will increase. As the entering flood rises and falls, the gate opening of spillway tunnels should be adjust at any moment to maintain the water level, risking to be emptied.

(3) The weir crest elevation of spillway is 1112.6m. When the water level is running lower than 1112.6m, the floating debris can not be released in time, so that the trash boom will be blocked by any possibility, threatening the safety of the diversion tunnel.

So, in fact, it is difficult for the power station to perform according to the design to control sediment. But according to the design principle, the water level for sand flushing can be still decreased, only not keeping as the dead water level 1105m all the time, for the reasons above.

Table 1 counts the water levels in flood season since the power station operated. From June to September, the running water level is between 1105m and 1115m, and the average value is 1110.13m.

3.3 SEDIMENT DEPOSITION

(1) Sediment deposition morphological character is delta (see Fig. 1). The deposition distribution is closely

Table 1 The running water level in flood season of Lubuge power station

(unit: m)

Year	The highest daily water level	The lowest daily water level	Averaged from June to September	Averaged from June to October
1989	1112.56	1104.39	1107.25	1107.75
1990	1123.67	1105.58	1110.82	1112.63
1991	1129.94	1105.42	1112.53	1115.08
1992	1129.56	1104.93	1110.09	1113.63
1993	1125.11	1105.52	1114.48	1116.13
1994	1129.01	1106.33	1110.41	1113.14
1995	1126.64	1105.51	1109.39	1111.45
1996	1129.78	1105.85	1110.26	1113.38
1997	1129.65	1090.39	1108.55	1110.89
1998	1127.62	1106.13	1109.30	1111.99
1999	1128.84	1105.63	1110.58	1113.13
2000	1126.85	1106.88	1109.97	1112.09
2001	1128.78	1107.80	1110.85	1112.33
2002	1128.94	1105.70	1108.37	1111.04
2003	1125.11	1105.97	1109.20	1111.82
Averaged	1126.80	1104.80	1110.13	1112.43

related to the topography of the reservoir. The reservoir is in dumbbell shape, that is, the upstream area and the backwater area is wide while the middle is the narrow valley. So, in the upstream area, the amount of the sediment deposition is much more than that in the middle of the reservoir. Along with the time, the elevation of sedimentation before dam gradually increased, with the character of cumulative deposition. In the backwater area, the sediment deposition changes as the running water level in flood season. For example, the averaged water level 1116.13m in 1993 is the highest averaged water level since the reservoir operated, and the deposition surface in the backwater area of the reservoir also achieved the highest.

(2) The changes of total storage and dead storage gradually decrease. The loss rate of the dead storage is the fastest, while the regulating storage keeps stable on the whole (see Fig. 2).

The total storage is 11100×10^{4}m^3 at the beginning, and then reduces to 7714×10^{4}m^3 in 2003. The total loss is 30.5 percent, and the averaged annual loss is 226×10^{4}m^3. The dead storage is 3700×10^{4}m^3 at the beginning, and then reduces to 1090×10^{4}m^3 in 2003. The total loss is 70.5% percent, and the averaged annual loss is 174×10^{4}m^3. The regulating storage is 7400×10^{4}m^3 at the beginning, and then reduces to 6624×10^{4}m^3 in 2003. The total loss is 10.5% percent, and the averaged annual loss is 52×10^{4}m^3. From 1993 to 2003, the regulating storage changed little, with the variation range between 6471×10^{4}m^3 and 6696×10^{4}m^3.

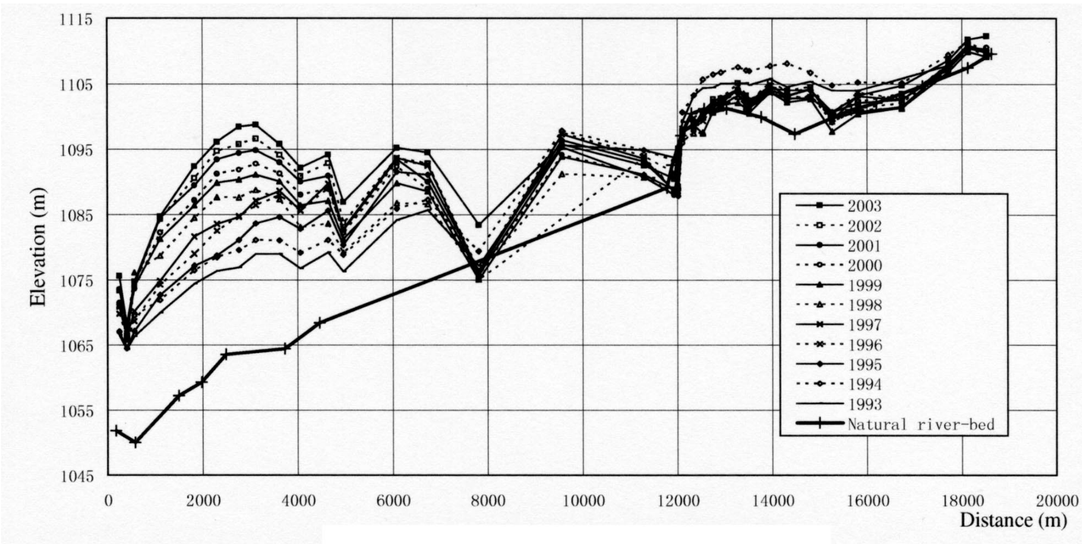

Fig. 1 Sediment deposition profile of Lubuge reservoir

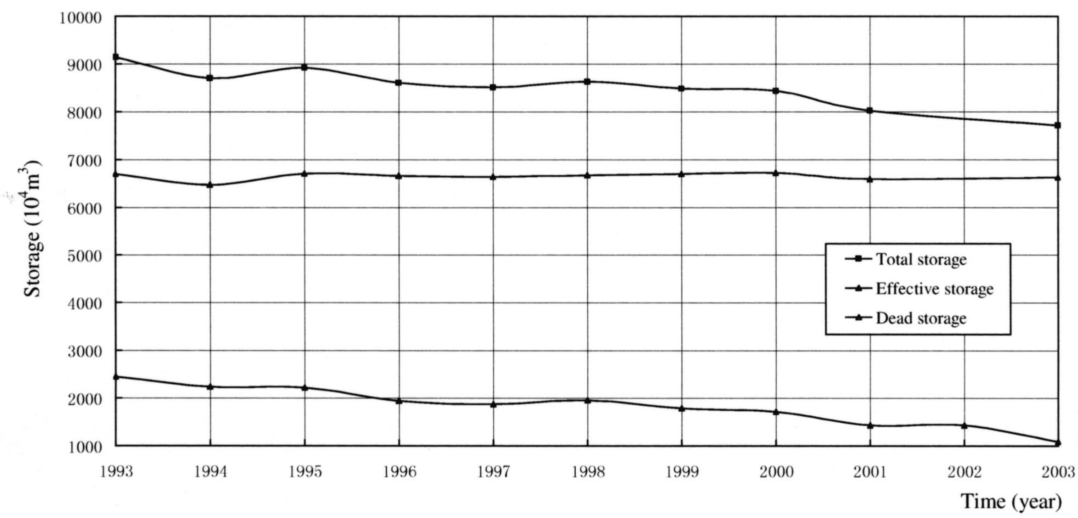

Fig. 2 Different characteristic storage of Lubuge reservoir power station

3.4 SAND FLUSHING IN 1997

From July 13 to July 17 in 1997, Lubuge Reservoir cut off the electricity to flush for the first time, and lasted 112 hours (see Table. 2).

In the process of sand flushing this time, when decreasing water level to flush (from 12:00 to 21:00 on July 14) or remaining a lower water level for longer time (from 21:00 on July 14 to 7:00 on July 15), in the discharge condition less than 1100m^3/s, the effect of sand flushing is better. While when the inflow discharge is large (after 7:00 on July 15, inflow flood increased, and peak value reached the flood return period of 20 years at 21:00), the water level for sand flushing decreased and then was compelled to rise, which will effect the flushing. Even decreasing the water level again (from 21:00 on July 15 to 20:00 on July 17), but because the discharge is much larger (more than 1500m^3/s), the effect is not perfect.

The net erosion amount 407×10^{4}t due to sand flushing this time can recover the storage close to that of the deposition loss in two years.

4. FORCASTING FOR SEDIMENT DEPOSITION

The spillway weir crest elevation of Lubuge power station is 1112.6m. If the water level in flood season is a little higher than 1112.6m (such as 1113.5m), the spillway gate can be used to control the water level, which can lessen the opening frequency of spillway tunnel gate, release the floaters in time, and supply the convenience for power station to operate in peak time to increase the power benefit. But raising the running water level in flood season will increase the sediment deposition, so whether the necessary regulating storage can be maintained for long time still needs to be researched. Here the 1D non-equilibrium sediment transport mathematical model, established by Wuhan University, is used to predict the sediment deposition in the reservoir.

The assumptions for sedimentation prediction are as follows:

(1) The principle for sediment controlling remains not changed, but the water level for sand flushing in flood season is raised to 1113.5m.

(2) The inflow representative year is selected, considering the inflow sediment more than the designed value.

4.1 MODEL CALIBRATION

Analyzing the integrative data of inflow water and sediment and measuring the reservoir from 1993 to 2003, the inflow sediment loads of some years do not correspond with the sedimentations measured in cross-sections, and the data from 1995 to 2000 has rather high precision. Considering the sand flushing in 1997, the measured landform in 1995, as well as the measured cumulative sedimentation with vertical and horizontal sections in 1996 and 1997 (before sand flushing), are selected to calibrate the model, until the results are much closer to the field data. The comparison between measured and computed results before flood season in 1997 is shown in Fig. 3.

4.2 FORCASTING RESULT

The forecasting result of forty years from 2003 to 2043 indicates that:

(1) Along with the time, the sedimentation increases. The delta continuously moves to the dam, will reach the dam in 2018, and then change as the form of cone (see Fig. 4).

(2) The elevation of sedimentation before dam continuously increases, and will reach about 1090.8m in 2013, close to the intake elevation of the power station. As the time increasing, the sedimentation will necessarily influence water intake of the power station, and the sediment concentration across generators will also increase, so the reservoir need open to flush.

(3) According to the design report, the total stor-

Table 2 Sand flushing of Lubuge power station in 1997

Term	Unit	numerical value
Time interval		From July 13 to July 17 in 1997
Inflow discharge range	m^3/s	554 ~ 3221 (peak value reached the flood return period of 20 years)
Outflow discharge range	m^3/s	550 ~ 2833
Water level range	m	1079.84~1108.79
Inflow sediment concentration range	kg/m^3	0.27 ~ 19.5
Outflow sediment concentration range	kg/m^3	2.75 ~ 64
Largest range of water level	m/h	4.82 (from 9:00 to 10:00 on June 15)
Inflow sediment load	10^{4}t	367
Total sand flushing amount	10^{4}t	774
Net erosion amount	10^{4}t	407

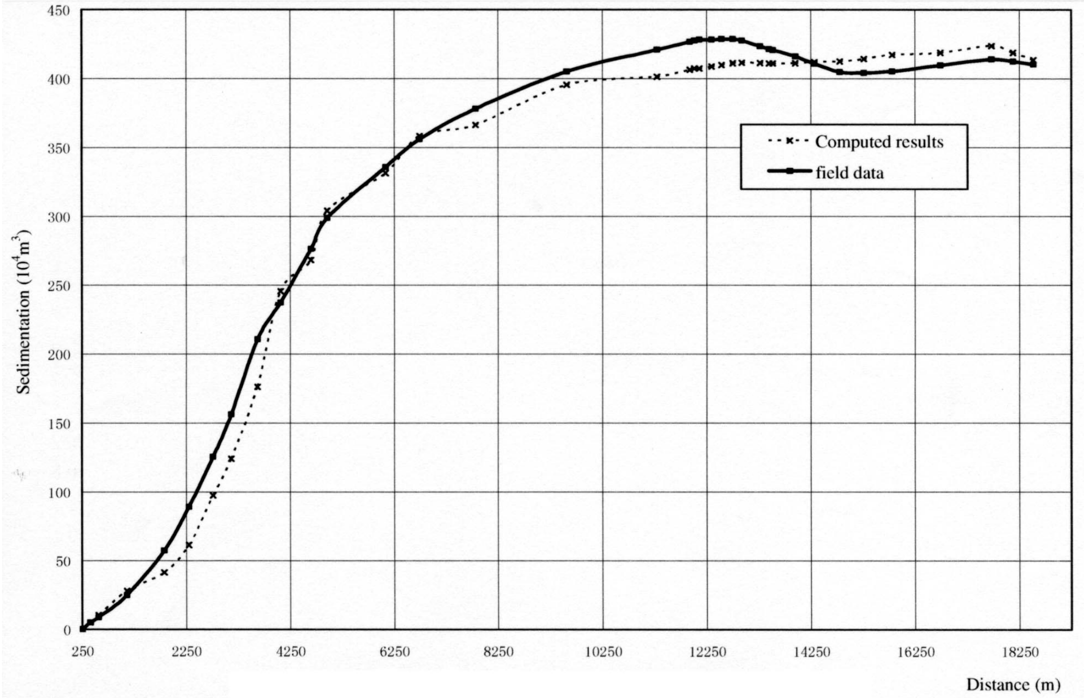

Fig. 3 Sedimentation comparison between computed results and field data in 1997

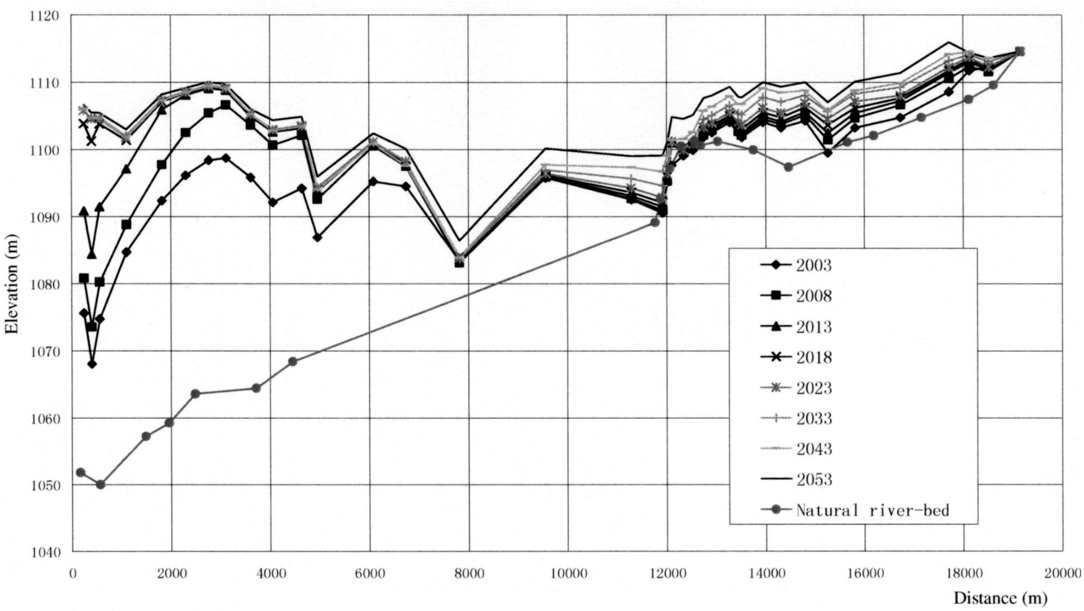

Fig. 4 Sediment deposition profile of Lubuge reservoir
(low limit level for sand flushing in flood season is 1113.5m)

age, that satisfied the need of monthly regulating, daily regulating and accident prepared, is about $3000\times10^4m^3$. As shown in Table. 4, to 2043, the regulating storage can meet the need.

5. FORECASTING OF FLUSHING EFFECT

The dead storage of Lubuge reservoir losses very quickly, seen from Table. 4, so the storage needs to be recovered by emptying the reservoir to flush every several years, according to principle for sand flushing. The model above is also used to forecast the flushing effect.

5.1 MODEL CALIBRATION

The parameters in the model are calibrated by the actual process of sand flushing in 1997. The sediment concentration comparison between the computed and measured results is shown in Fig. 5. The computed peak value agrees well with the field data. The largest output sediment concentration is $69.8kg/m^3$, 8.3 percent more than the measured $64kg/m^3$. The sediment peak appears on 15:00 July 13, which is caused by landslide, so the model can not simulate this.

Table 4 Forecasting the storage loss

Year	Total storage (10^4m^3)	Total storage loss rate (%)	Efficient storage (10^4m^3)	Efficiency storage loss rate (%)	Dead storage (10^4 m^3)	Dead storage loss rate (%)
Before building reservoir	11100	0	7400	0	3700	0
2003	7714	30.5	6624	10.5	1090	70.5
2008	6875	38.1	6302	14.8	573	84.5
2013	6352	42.8	5985	19.1	367	90.1
2018	6085	45.2	5821	21.3	264	92.9
2023	5952	46.4	5702	22.9	250	93.2
2028	5840	47.4	5599	24.3	241	93.5
2033	5734	48.3	5501	25.7	233	93.7
2038	5630	49.3	5404	27.0	226	93.9
2043	5566	49.9	5345	27.8	221	94.0

5.2 SCHEME AND EFFECT OF SAND FLUSHING

5.2.1 SCHEME DECISION

Seen from the sand flushing in 1997, the discharge for flushing is not suitable to be much great. Based on the experiences of other reservoirs, the flood return period of 2 to 5 years is commonly used for flushing. Analyzing the measured flood at Chajiang station from 1993 to 2002, the flood process in 1999 (the flood return period is 5 years) and in 2002 (the flood return period is 2 years) are chosen for flushing, and two different instances of the peak flow coming and receding are considered at the same time.

The first scheme is to decrease the water level from 1105m to 1090m. The second scheme is to decrease the water level to 1095m according to the flushing experiences in 1997. Contrast scheme is decreasing again to 1080m after remaining 1095m for one day. The two flushing processes above are the upper and lower limited schemes to actually control the water level for sand flushing.

5.2.2 SAND FLUSHING EFFECT

The computed results of different forecasting schemes are listed in Table 6.

From Table 6, it is known that: the flushing effect of basic schemes is not as well as that of contrast schemes. Flushing effect of the flood process in 2002 is better than that in 1999, because comparing the chosen flood from August 13 to 19 in 2002 with the flood from July 1 to 7 in 1999, although the peak discharge of the former is smaller, but the flood process is fatter, and the duration of big discharge is longer, so the flushing competence is better. Flushing effect is better when the peak comes than that when the peak recedes, for the scheme is the same. Although the range of water level decreasing is limited by the land-

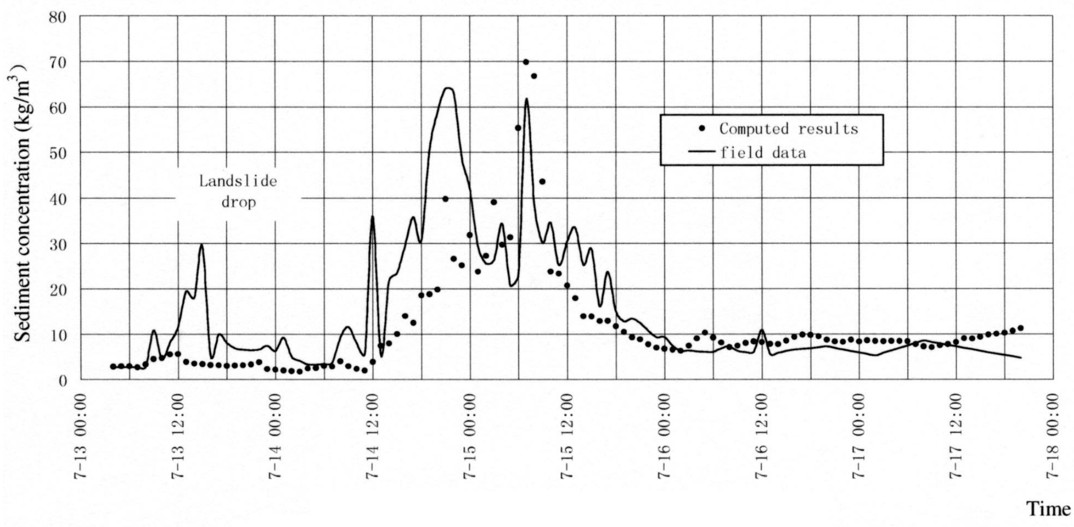

Fig. 5 Comparison between computed results and field data of sediment concentration of deposit flushing in 1997

Table 5 Sand flushing calculation schemes

Discharge process	Water level for sand flushing	
	Basic scheme (Considering water level change with landslide)	Contrast scheme (not considering water level change with landslide)
Flood peak windward in 1999	Scheme 1	Scheme A
Flood peak recession in 1999	Scheme 2	Scheme B
Flood peak windward in 2002	Scheme 3	Scheme C
Flood peak recession in 2002	Scheme 4	Scheme D

Table 6 Sand flushing effect of typical floods

Flushing conditions		Flood peak in 1999		Flood peak in 2002	
		Basic schemes	Contrast schemes	Basic schemes	Contrast schemes
Flushing amount of five days ($10^4 m^3$)	Flushing when flood peak comes	470	818	751	1153
	Flushing when flood peak recedes	278	550	634	1050

slide mass, causing the flushing effect somewhat worse, the recovering effect to the dead storage is good to some extent, seen from that the flushing amount still achieves 751×10^4 m^3 when the flood comes in 2002.

6. CONCLUSIONS

(1) The inflow sediment of Lubuge reservoir is mainly in flood season. Decreasing the water level to flush in flood season can control the sedimentation effectively. The operating experience of the power station indicates that the water level for sand flushing can be increased properly to generate electricity better and reduce the controlling difficulty.

(2) If the water level in flood season is higher than 1112.6m, the spillway gate can be used to control the water level, which will lessen the opening frequency of spillway tunnel gate, release the floaters in time, and be convenient for power station to operate in peak time to increase the power benefit. Forecasted by the mathematical model, the regulating storage will be satisfactory after Lubuge reservoir has been running at the water level 1113.5m for sand flushing in flood season for 40 years. But the loss of dead storage is very great due to sedimentation, so the storage needs to be recovered by emptying the reservoir to flush every several years.

(3) The computed results demonstrate that the discharge for emptying the reservoir to flush is selected to be close to that of the flood in fat type with return period of 2 years, the flushing effect will be much better when the peak flow comes. Because the stability of the landslide upstream dam should be considered, the range of water level decreasing is limited. Even so, the flushing effect is still good.

Chracteristics of the Measures in Controlling Erosion and Sediment from Tillage

Jae-Heung Yoon
Director, Research Management Center, Korea Institute of Water and Environment KOWACO, Daejeon, Korea

Dong-Kyun Kang
Office of Soyanggang Dam, KOWACO, Korea

Woo-Ku Kim
President, Korea Institute of Water and Environment, KOWACO, Daejeon, Korea

Abstract

Soil erosion on basin tillage has been one of the most severe problems in managing the reservoirs of multi-purpose dams since it causes turbid water and sediment. The turbid water makes it difficult to treat water at downstream areas. The sediment deconstructs the eco-systems in rivers and lakes, deteriorate the water quality of reservoirs, and diminish the life of dams. The Reservoir Soyang, which has the largest storage capacity in Korea, suffers from supplying highly turbid water to downstream areas after floods.

Its reservoir basin has lots of vegetable tillage on the slopes of the upstream areas. In the study, a variety of methods in controlling the erosion and sediment transport from the were tested for their efficiencies by experiment in a cultivated area located in the upstream basin of the Reservoir Soyang.

Among the measures, the contour plowing method is the most efficient one to reduce the sediment transport. The furrow-dam with weeds or gravels can also decrease the sediment transport by 30% to 60% even though it can not reduce the amount of soil erosion. PAM-10 and PAM-40 result in a similar effect comparing to the furrow dams. Vinyl sheet cover shows an adverse effect on the sediment control. The vinyl sheet protects the soil surface from erosion, but it increases the runoff rate in the furrow. Furrow compaction method shows about 50% reduction of sediment transport. The sediment yield roughly shows a linear relationship to total rainfall depth, and nonlinear relationship to the rainfall intensity

1. Introduction

Erosion is a geomorphologic process consisting of weathering of the rock material, entrainment of the weathered debris, and transportation and deposition of the debris (Novotny and Olem, 1994). The erosion can be caused by gravitational, molecular and chemical stresses applied on the surface of the earth. The sources of the gravitational stress include air and water movement over the surface. The erosion depends on slope, climate, geological material, and weathering processes (Leopold et al., 1964). Rainfall as water movement is the most significant cause of soil erosion. Walling and Webb (1983) reported that the rates of natural erosion and sediment yields ranged from 1 ton/km^2/year to 10,000 ton/km^2/year.

Erosion process of soil can be divided into sheet, rill, gully, and stream bank erosions. Erodibility depends on both the composition of geological materials and the consolidation state. The erodibility of unconsolidated geological materials depends on the material type, particle size, water content, composition of the materials, and vegetation. Among the vegetation is the most important factor since it provides additional resistance to shear stresses caused by falling and running water. A small amount of clay in the unconsolidated mixture significantly improves the cohesion of the soil. Cohesion is also increased by organic materials and chemical bonding (Leopold et al., 1964). The eroded sediment is transported by the water flow as suspended load or bed load. The suspended load is moved away from the river bed and affected by the turbulence, and the bed load moves along the river bed and is affected by drag force. The movement of sediment also depends on the cohesiveness of soil materials. The fine particles of eroded soil mainly form the suspended load, and the coarse particles contribute the bed load. The bed load directly makes an impact on the environmental and hydraulic systems near a source of erosion, but the suspended load is transported to the downstream far away from the source.

Soil erosion is a major cause of diffuse pollution, and sediment is also the most visible pollutant. The effects of excessive sediment loading on receiving water bodies include the destruction of aquatic ecosystems in rivers, deterioration of scenic valve, loss of reservoir storage capacity, and accumulation of bottom deposits that inhibit normal biological lives. The nutrients carried by sediment can stimulate algal growths and, consequently, accelerate the process of eutrophication (Clark et al., 1985). Fine particles are also a primary carrier of such pollutants as organic matters, metals, ammonium ions, phosphates, and many organic toxic compounds. Recently the soil erosion and sediment yields from the cultivated areas in reservoir basin become one of the most severe problems in managing the reservoir of multipurpose dams. Turbid influent contaminates the reservoir water body, and drinking water treatment plants at downstream areas are confronted with difficulties in treating turbid water.

In Korea, there are several dam reservoirs that suffer from turbid influent during a wet season running from June to September. The turbid water came into the reservoir forms a layer of turbid water of which depth ranges from 10 m to 20 m because the temperature of influent is different from that of reservoir water. Hence the turbid water layer does not appear on the surface of reservoir with a yellow color. After the wet season, the turbid water is released to the downstream of the reservoir through the hydro-power generation since the elevation of penstock intake generally corresponds to the turbid layer. The released water makes it difficult to treat the turbid water for drinking water at the treatment plants located at downstream areas. In water treatment processes, the highly turbid water causes poor coagulation and flocculation with a normal amount of chemicals, which requires more chemicals for a proper flocculation process. In addition, the turbid water makes harmful effects on the eco-system in the downstream rivers.

The Soyang dam located at the upstream of the North Han River is the largest multipurpose dam as a storage capacity in Korea. Most of the cultivated areas in the upstream basin were made by removing trees and plowing the slope. The furrows and ridges in the tillage are directed from upland to the bottom of slopes for drainage. Hence the furrows have a steep slope. If the furrows are made without slope, water stays in the furrows due to poor drainage. The water in the furrows makes potato and radish rotten. However, since the slope of tillage is generally too steep for proper drainage, contour plowing method might control the sediment. In addition the surface soil of the tillage should be replaced with new soil brought from other places every two to three years to prevent the depletion of nutrients necessary for potato and radish. Therefore bare soil of the most tillage is exposed on the surface.

Erosion control mainly consists of the prevention of soil erosion and the reduction of sediment delivery from the source areas to the receiving water body. The control methods include land management, buffer strips, channel modification, sediment traps, and other structural and nonstructural measures and practices (Novotny and Olem, 1994). The purpose of this study is to investigate the efficiencies of some sediment control measures which can be practically and easily implemented in the fields by field experiment

2. Method of Experiment

In order to identify the methods that can effectively control the erosion and sediment, a variety of methods were implemented in a field of radish, which was located at the upstream basin of the Reservoir Soyang. The experiment tillage has length of 25m, width of 25 m, and a slope of 13% as shown in Figure 1. The selected control measures of erosion control include vinyl cover, furrow compaction, Polyacrylamid-10 (PAM-10), and Polyacrylamid-40 (PAM-40), and the methods for sediment transport control are contour plowing, a clump of weeds, and a pile of gravels. The sediment yield for each control method was measured for five rainfall events, whose rainfall depth ranges from 21 mm to 175 mm. In the case of PAM, two different concentrations of the Polyacrylamid were tested for the control.

Eight furrows were chosen in tillage for the experiment. The furrows were used for seven control cases and for a non-control case. To collect the sediment yield from every testing furrow, a 20 liter plastic bucket was installed under the ground at the downstream end of every furrow. Inflow and outflow weirs were made in the bucket. The size of weirs was 10 cm high and 10 cm wide, and the bottom levels of weirs correspond to the furrow bed. A screen was installed at the outlet weir to prevent sediment from release. Hence the sediment transported from the upstream furrow easily entered into the bucket without any resistance. For each rainfall event, the weight of the sediment collected in the bucket was measured, and the bucket was made empty for the next rainfall event and was reinstalled at the original pit.

There are many factors that affect the soil loss. In order to identify the factors affecting the soil loss, it would be necessary to consider the Universal Soil Loss Equation (USLE) that is most commonly used and well known estimator of soil loss caused by upland erosion (Novotny and Olem, 1994) as described in Equation 1.

Figure 1 Control measures implemented in the experiment tillage

A = (R)(K)(LS)(C)(P) ....1)

where, A is a calculated soil loss in tonnes/ha for a given rainfall event; R is a rainfall energy factor; K is a soil erodibility factor; LS is a slope-length factor; C is a cropping management (vegetation cover) factor; P is a errosion-control practice factor. In the equation, the slope-length, vegetation, and erosion-control practice factors were fixed in this study. Only the rainfall energy and soil erodibility factors can be considered to find out

For the erosion control methods selected in this study, the erosion is significantly affected by the rainfall energy. In Equation 1, the rainfall energy factor is also a function of rainfall intensity and duration, which means that the soil loss by the erosion would be decreased if the impact of rain on the soil is reduced. Hence one of the ridges was covered by the sheets of vinyl, but the furrows were not covered in this experiment. In the furrow compaction method, the furrow was thoroughly compacted by a man's weight with stepping since there was no compaction equipment available. The stepping weight is about 70kgf. As a coagulant method, both PAM-10 and PAM-40 were applied on the furrows using a spray. The numbers of 10 and 40 indicate the amount of polyacrylamid applied in kilogram per unit hectare (Flanagan, 2001).

For the control of sediment transport or delivery to the downstream of furrows, three measures were tested in this study. Since the tillage also has some contour plowing furrows along the uppermost part of the tillage, one of the furrows was directly used for the experiment as a contour plowing case. As a clump of weeds, called furrow dam, the weeds were cut out around the experiment site, and piled at the middle of the furrow. The diameter of a weed clump is about 15cm. Hence the weed pile can reduce the amount of transported sediment as a furrow-dam. Instead of the weeds, gravels whose diameter are about 5cm were also used as the same way as weed method. The difference between weed and gravel methods is the availability of the materials near the cultivated land, since the availability of materials is important in application.

3. Results and Discussion

3.1 Sediment Transport

As a result, the contour plowing is the most efficient way of reducing the sediment yield in the tillage. In this method, the sediment is not well delivered to the downstream of furrows even though soil erosion significantly occurred. The sediment reduction ratio of the contour cultivation to non-control case is 95% in weight. Furrow-dam using weeds or gravels cut the amount of sediment yield by 30% to 60%. PAM-40 can reduce the sediment by 67%, and PAM-10 shows 30% reduction in sediment yield. The larger amount of PAM application results in greater reduction of sediment transport. Since the PAM is viscous and fix the soil by coagulation, it can reduce the soil erosion. The vinyl sheet cover produced more sediment than the non-control case by 45%, which was resulted from high runoff rate without infiltration and water losses. Compaction method also shows about 50% reduction. Table 1 shows the rainfall characteristics of five events, and Table 2 summarizes the sediment yields of the implemented methods.

Table 1 Characteristics of Rainfall Events

Rainfall Event	Date	Rainfall Total Depth (mm)	Max. Intensity (mm/hr)	Avg. Intensity (mm/hr)
R1	07-09	68	15	6.2
R2	07-21	72	9	2.9
R3	07-24	175	17	5.1
R4	07-29	21	5	2.1
R5	08-08	53	22	4.8
Total		389	22	4.22

Table 2. Amount of Transported Sediment for Different Control Measures
(* Sediment Reduction Rate of Measures to Non-control Case; Unit: g)

Event	Non-Control Case	Contour Plowing	Vinyl Cover	Weed Pile	Gravel Pile	Compact-ion	PAM-10	PAM-40	Total
R1	11,300	705	13,700	7,350	4,250	3,610	5,600	3,750	50,265
R2	1,900	50	1,400	850	1,100	1,150	480	70	7,000
R3	16,300	850	29,900	14,900	8,200	10,850	20,150	8,800	109,950
R4	750	50	380	350	50	270	150	50	2,050
R5	9,700	300	12,600	3,600	1,600	2,500	1,700	500	32,500
Total	39,950	1,955	57,980	27,050	15,200	18,380	28,080	13,170	201,765
* (%)		95.1	-45.1	32.3	62.0	54.0	29.7	67.0	

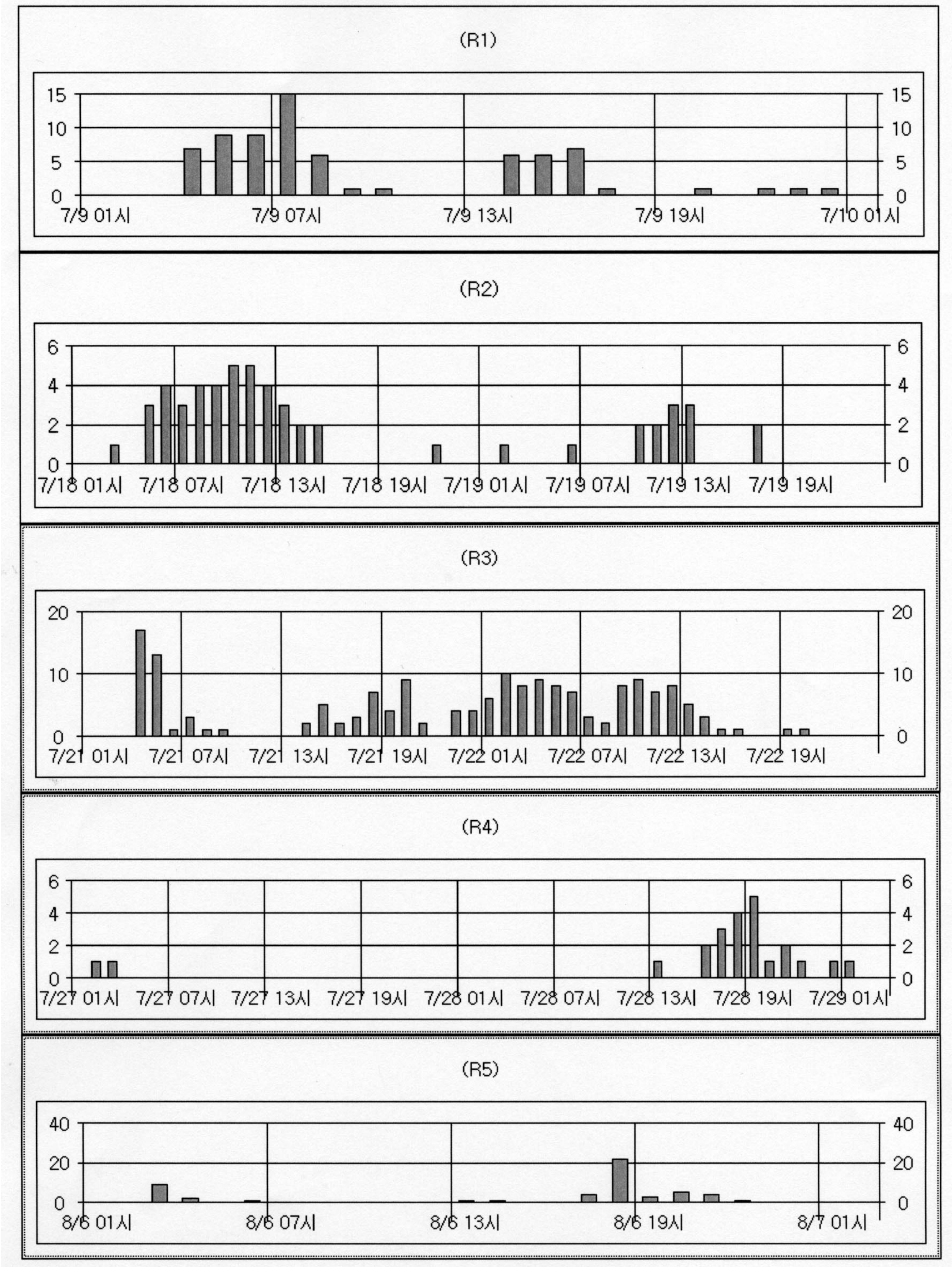

Figure 2 Rainfall depth distribution of five events

3.2 Effect of Rainfall Depth and Intensity

The amount of sediment shows significant association with the average rainfall intensity over the period of a rainfall event. The total amount of rainfall also affects the sediments. The greater maximum rainfall intensity makes a more effect on the sediments under the condition of similar average rainfall intensity. Event R1 shows similar maximum rainfall intensity and mean intensity to Event R3. Total sediment yield in Event R3 (109,950 g) is greater than that of Event R1 (50,265 g) since total rainfall depth of Event R3 is about 2.6 times as much as that of Event R1. In case of R3 and R5, rainfall intensity is similar, and rainfall depth of R3 is 3 times as much as that of R5. Hence the sediment yield in R3 is about 3 times as much as that in R5. Consequently, under the similar rainfall intensity, the sediment yield has roughly a linear relationship with total rainfall depth. Total rainfall depths of R1 and R2 are 68 mm and 72 mm, and average rainfall intensities are 6.2 mm/hr and 2.9 mm/h, respectively. The amount of sediment transported by R1 (50,265 g) is about 7 times as much as that of R2 (7,000 g) even though the total amounts of rainfall depth are similar in the two events, which means that the average rainfall intensity nonlinearly affects the soil erosion and sediment transport.

4. Conclusion

In order to reduce the sediment transport in the tillage, seven control measures were implemented at a cultivation site in the basin of Reservoir Soyang. The amount of sediment that was transported to the downstream of furrows was measured after rainfall events. There were five rainfall events during the investigation. Among the measures, the contour plowing method is the most efficient one to reduce the sediment transport. The furrow-dam with weeds or gravels can also decrease the sediment transport by 30% to 60% even though it can not reduce the amount of soil erosion. PAM-10 and PAM-40 result in a similar effect comparing to the furrow dams. Vinyl sheet cover shows an adverse effect on the sediment control. The vinyl sheet protects the soil surface from erosion, but it increases the runoff rate in the furrow. Furrow compaction method shows about 50% reduction of sediment transport. The sediment yield roughly shows a linear relationship to total rainfall depth, and nonlinear relationship to the rainfall intensity.

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Comprehensive Sediment Management in the Kurobe River

Hirokatsu Kanazawa
Director, Kurobe Office of River, Hokuriku Regional Development Bureau, Ministry of Land, Infrastructure and Transport, Japan

1. Introduction

The Kurobe River is a class A river that flows through the eastern part of Toyama Prefecture. As one of the most torrential rivers in Japan, it discharges a characteristically large volume of sediment from its upstream area. Due to the accumulation of a substantial quantity of sediment runoff over several thousands of years, a typical alluvial fan has been formed in the lower reaches of the Kurobe River. This beautifully shaped fan jutting into the ocean is recognized as a rare formation in the world. However, there is a danger that such a great quantity of sediment runoff may cause earth-flow disasters. In recent years, as upstream sediment has not been conveyed down to the river mouth due to changes in the flow regime, there has been much erosion along the coast in the area near the river.

For this reason, it is vital that a comprehensive sediment management system be established for the Kurobe River in order to secure a steady discharge of sediment, maintain stable river channels and preserve coastal areas, while at the same time preventing earth-flow disasters. As one of the countermeasures, coordinated sediment flushing has been carried out from the Unazuki Dam, completed in 2001 and managed by the Ministry of Land, Infrastructure and Transport, and the Dashidaira Dam, completed in 1985 and managed by Kansai Electric Power Co., Inc.

Many problems remain to be addressed before a comprehensive sediment management system can be established. This paper introduces some of the problems to be faced in formulating the system and outlines the coordinated sediment flushing operation and its achievements.

2. Comprehensive sediment management for the Kurobe River

(1) Profile of the Kurobe River

Figure 1 shows the Kurobe River basin. The Kurobe River, 85 km in length, is one of the most torrential rivers in Japan. It flows from Japan's highest 3,000 m level alpine mountains right down to the Sea of Japan and has an average riverbed slope of about 1/30.

The rainfall in the basin is also conspicuous in Japan. The annual rainfall in the upstream area reaches 4,000 mm, more than double the national average. Hydroelectric power generation has been utilized to take advantage of the abundant volume of water and torrential flow.

The Kurobe River has, since ancient times, meandered wildly across its alluvial fan, causing problems for people living in the area. This was particularly so when a flood levee failed and devastating damage was inflicted in an all-time record flood in August 1969. This huge flood was a turning point, and a project was formed under ministerial jurisdiction, to construct the Unazuki Dam mainly for the purpose of flood control.

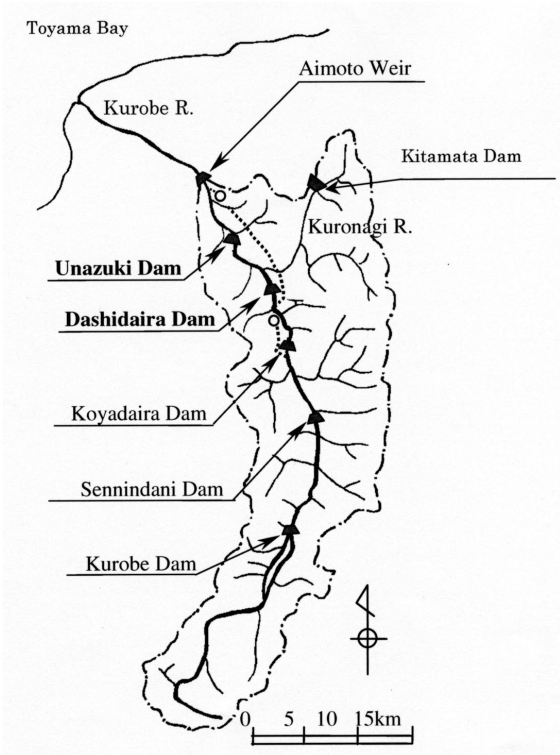

Figure 1 Kurobe River Basin

(2) Landslide and sediment runoff in the basin

The upstream area of the Kurobe River is known as the unexplored Kurobe Canyon. It is formed basically of granite and is susceptible to weathering. Because the area is relatively new in terms of geochronology, landslide scars of various sizes can be observed at as many as 7,000 locations. These landscape scars occupy nearly 5% of the total area.

A large quantity of sediment runs off along with the rainfall and freshet from these sites. The total volume of sediment runoff in the basin is estimated at about 1.4 million m^3 per year.

Many times, severe damage has been caused by this large volume of sediment runoff. The most recent instance was an extensive landslide that occurred in July 1995 in the upstream area as a result of heavy

Photo 1 Damage inflicted on the Nekomata Station of the Kurobe Gorge Railway in July 1995

rains, causing massive quantities of sediment and timber to flow into the Dashidaira Dam and nearly 6 million m^3 of sediment to accumulate in the middle course of the Kurobe River. Substantial damage was caused, not only to transport facilities such as the Kurobe Gorge Railway as shown in Photo 1, but also to power generation facilities and local tourist spots.

(3) The need for a sediment supply downstream

The Kurobe River carries a large quantity of sediment runoff, and so, it is natural that this will flow into the downstream area. For this reason, if this sediment supply mechanism is obstructed, various problems can be expected to occur.

1) Effect on the downstream course

A large quantity of sediment movement is conspicuous around the middle to lower course of the Kurobe River, and this causes the river channel to be unstable. In recent years, the degradation of the riverbed has become so bad that revetment bases have been constructed in many areas. Accordingly, if the volume of sediment supplied from the upstream course is further reduced, there will be a further degradation of the riverbed and the downstream river channel will become even more unstable.

2) Effects on the coastal area

Wave overtopping and coastal erosion have been observed since ancient times in the coastal area around the Kurobe River mouth. It is estimated that the coastal line has receded some 200 m over the past 100 years. Thus, measures are being taken to prevent further coastal erosion by installing coastal protection facilities. At the same time, it is also necessary to maintain a continuous supply of sediment from the river.

However, if the spontaneous sediment runoff is left to take its own course with no human intervention, such earth-flow disasters as mentioned above are likely to occur again and again. As far as the Kurobe River is concerned, a well-balanced comprehensive sediment management system is vital to realize as great a natural sediment runoff as possible and to prevent earth-flow disasters.

(4) Challenges in the comprehensive sediment management system

1) Strategies for sediment management

The comprehensive sediment management operations performed in the Kurobe River are only at their beginning stages. Figure 2 gives a conceptual diagram

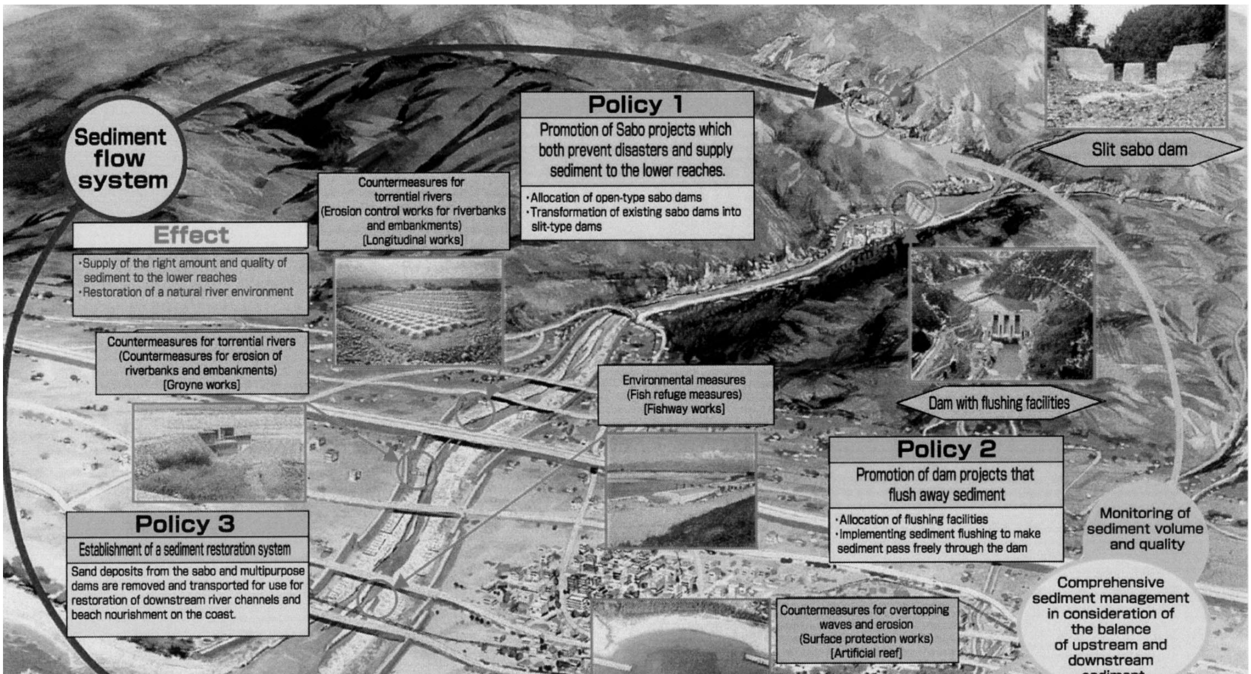

Figure 2 Conceptual diagram of Kurobe River comprehensive sediment management

Photo 2 Establishment of slit-type sabo dam (Kokurobedani sabo-dam No.1)

Photo 3 Establishment of offshore breakwaters (Yoshiwara District in Nyuzen Town)

of the overall basin management and some strategies currently underway are discussed in this section.

a) Installation of slit sabo dams
With the aim of preventing earth-flow disasters, a sabo dam is built on a mountain stream where there is a large outflow of sediment so that the quantity of sediment runoff can be controlled. However, to avoid the excessive capture of soil at times of normal flow, slit sabo dams such as that shown in Photo 2 have come into use. Slit sabo dams effectively check large quantities of sediment discharge at times of flooding, while supplying sediment downwards from between the slits at normal times.

b) Dam sediment flushing
It is usual that, when a dam is newly constructed, much of the upstream inflow sediment accumulates in the reservoir, reducing the amount of sediment supplied downwards from the dam. In the Kurobe River, which is characterized by a large amount of sediment runoff, the checking of sediment transport by the dam is most likely to have a large effect on the lower river course and coastal area. At the same time, voluminous sedimentation in the reservoir can decrease the capacity for both flood control and water supply, resulting in dam malfunction. Therefore, a sediment flushing gate was installed in both the Dashidaira Dam and the Unazuki Dam to flush soil downwards.

c) Apt gravel quarrying plan
The riverbed and riverbank from the middle to lower course of the Kurobe River consist mainly of gravel and small stones. Quarrying has been carried out extensively in the area, as the quality of the gravel is very high. The Kurobe River used to be a raised bed river, and so gravel quarrying was an effective measure for flood control. In recent years, however, with the degradation of the riverbed, there has been a restriction on these activities. However, an appropriate plan for gravel quarrying is to be closely examined in the future.

d) Improvement of coastal protection facilities to prevent beach drifting
Because the sea-bottom slope offshore from the Kurobe River mouth is steep, it is assumed that much of sediment during discharged times of flooding is transported deep under the sea and beach sediment contributes little to maintaining the coastal area. Thus, as Photo 3 indicates, coastal protection facilities such as offshore breakwaters and artificial reefs have been constructed to prevent coastal erosion and beach drifting.

e) Beach nourishment and sand bypassing
Although coastal protection facilities have been installed to prevent coastal erosion and beach drifting, the coastal area has not yet recovered fully, due to an insufficient volume of beach sediment. Therefore, the beaches are being nourished to rehabilitate the coast. Furthermore, sand bypassing is being performed to transport the sand and gravel accumulating at the river mouth for use as beach nourishment materials.

2) Survey on the sediment movement

In conducting sediment management, it is vital that the movement of sediment be grasped as basic information. Unlike water, however, it is quite difficult to establish the quality and quantity of discharged sediment. In the Kurobe River and its coastal environment, various attempts have been made to understand sediment movement. Photo 4 and Figure 3 illustrate a real-time suspended sediment densitometer and its schematic diagram.

Survey of hillside failure: filming and interpretation of aerial photos, field reconnaissance and morphometry by helicopter

Survey of sedimentation: CCTV monitoring and continuous fixed-point photography

Survey of riverbed deformation: profile and cross-leveling, granulometric analysis and installation of a riverbed degradation sensor

Survey of coastal deformation: aerial photography, sounding, granulometric analysis and follow-up checks using colored sand

Survey of water quality: turbidity and SS measure

Survey of bed load: direct sampling of sediment discharge using a backhoe

Survey of suspended sand: measurement using a real-time suspended sediment densitometer

Photo 4 Real-time suspended sediment densitometer

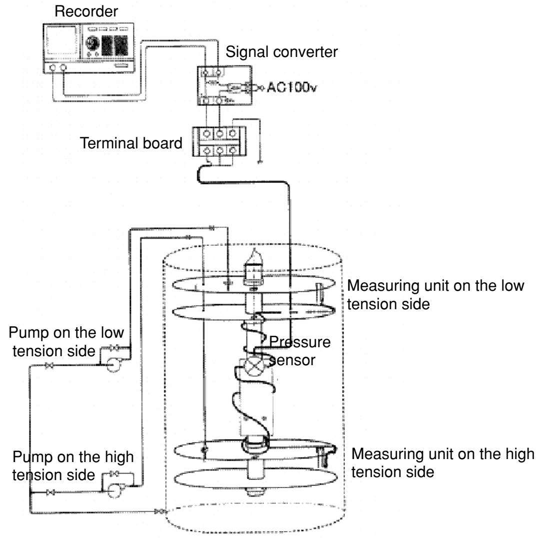

Figure 3 Schematic diagram of a real-time suspended sediment densitometer

and direct sampling of sediment discharge using a backhoe

3. Sediment flushing at dams on the Kurobe River

(1) Necessity of conducting coordinated sediment flushing in the Kurobe River

From the viewpoint of comprehensive sediment management, a downstream supply of sediment is necessary for the Kurobe River. It is also quite difficult to provide a dam with a large storage capacity for sedimentation. Accordingly, both the Unazuki and the Dashidaira Dams are equipped with sediment flushing facilities so that they may jointly conduct sediment supply to the downstream area end of a flood period in as natural a manner as possible.

(2) History of sediment flushing in the Dashidaira Dam

The Dashidaira Dam was completed 7 km upstream from the Unazuki Dam in September 1985, by Kansai Electric Power, exclusively for the purpose of power generation. The dam was equipped with sediment flushing facilities and its first operation was carried out in early December, 1991.

However, the first sediment flushing operation created problems for local residents as it adversely affected the lower reaches of the river and the coastal area. In this light, the Kurobe River Dashidaira Dam Flushing Evaluation Committee was formed. This consists of experienced scholars, administrative organs and relevant interested groups. In April 1995, the committee issued a report that published the results of their examinations and their proposals. The report concludes with the following statement, "A sediment flushing operation was initiated six years after the impoundment of the Dashidaira Dam. Meanwhile, the flow of organic material, including trees, leaves and humus had continued and accumulated in the dam together with a considerable quantity of sediment. The degradation of this material through anaerobic decom-

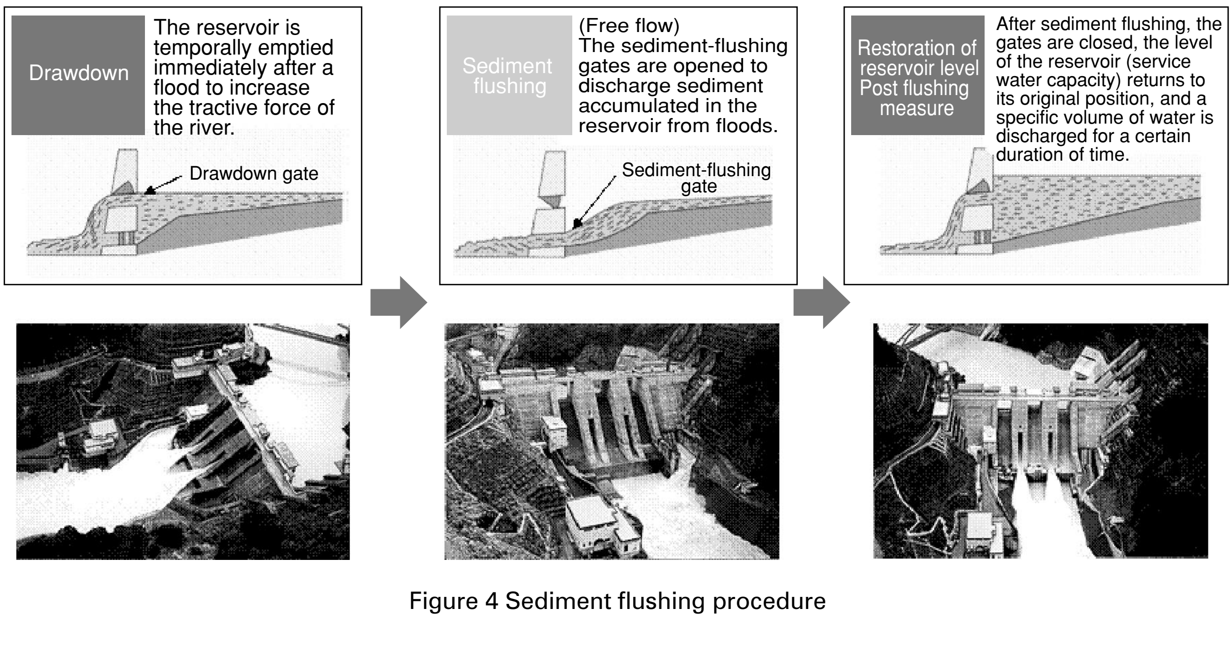

Figure 4 Sediment flushing procedure

position and the resultant impact on the basin had not been anticipated at all."

The report suggested that dam sediment flushing be conducted annually during times of river overflow and flooding, and that environmental surveys should be continued, giving priority to the alleviation of impact on the basin as far as possible.

(3) Outline of coordinated sediment flushing by the Unazuki Dam and the Dashidaira Dam

On the basis of the proposals outlined above, the committee members examined the method used for sediment flushing to reduce its impact on the basin as much as possible. As a result, the following method was judged to be the most appropriate.

Sediment flushing operations should be carried out during times of river overflow and flood between June and August to maintain the riverbed shape as far as possible without allowing an excess of sediment to accumulate in the dam reservoir. In the final stage following a flood peak, a drawdown gate would be opened to lower the dam water level. Next, sediment flushing gates should be opened to restore the natural pre-impoundment condition of the river inside the dam reservoir and to make use of the tractive force of the inflow to discharge the sediment. This state is called "free flow." Following this, the reservoir would be replenished, and a specific amount of water would be discharged from the dam as a post-flushing measure to wash down any sediment still remaining in the downstream channel.

They also suggested that there should be closer liaison between the Unazuki and the Dashidaira Dams in coordinated sediment flushing operations.

Every factor was taken into consideration, including the frequency of flooding, in consultations with interested groups engaged in inland water fishery, sea fishery and agriculture, and, as a result, it was determined that coordinated sediment flushing operations would be carried out between June and August.

There are three major differences between the current sediment flushing method and the initial method performed by the Dashidaira Dam in 1991:

(1) In the initial sediment flushing operation, the sediment discharged from the Dashidaira Dam had degraded over a period of six years. In the current method, sediment flushing operations are carried out on a yearly basis to prevent sediment degradation.

(2) The initial sediment flushing operation in the Dashidaira Dam was carried out during winter when the river was low. In the current method, the operation is synchronized with the occurrence of flooding which is large enough in scale to allow turbid water to flow through under normal conditions.

(3) In current sediment flushing operations, advance explanations and hearings are held to give interested groups, scholars and local residents a better understanding of the procedure. This kind of effort was not made for the initial operation of the Dashidaira Dam.

(4) Coordinated sediment flushing operation plan

As Table 1 indicates, coordinated sediment flushing operations are carried out based on the coordinated sediment flushing operation plan which delineates the amount and timing of sediment flushing, the flow type, its duration, and the measures to be carried out prior to and following the actions of flushing and sluicing.

1) Sediment flushing and sediment sluicing

In the coordinated sediment flushing operation plan, the operations are divided into two phases: sediment flushing and sediment sluicing.

Sediment flushing is carried out first on a flood of a certain magnitude within the sediment flushing period for the purpose of maintaining the shape of the sedimentation in the reservoir as much as possible by discharging the soil that had accumulated before the sediment flushing period.

Sediment sluicing is performed when a certain level of flooding occurs after the sediment flushing operation within the sediment flushing period, defined as the passing inflow flood sediment through.

2) Standard flow rate for sediment flushing operations

To successfully conduct sediment flushing operations on a yearly basis and maintain a flow rate of at least 130m^3/s in the free flow, the standard flow rate for sediment flushing operations has been set at 300m^3/s or over for the Dashidaira Dam and 400m^3/s or over for the Unazuki Dam (over 250m^3/s for the Dashidaira Dam only during the thawing and rainy season when the flow rate is higher).

The standard flow rate for sediment-sluicing operations depends on the flood discharge of each dam.

3) Quantity of sediment flushing

To maintain the specified level of sedimentation in the reservoir as much as possible, the excessively accumulated soil is regarded as a target volume and is therefore flushed annually.

Specifically in the case of the Dashidaira Dam, the amount of sediment accumulating after the last sediment flushing (or sediment sluicing) operation until May of the following year is assigned as the target volume. The target volume for the dam for 2005 is estimated as 540 thousand m^3. Meanwhile, the target volume for the Unazuki Dam is 0 m^3, as it is considered that the current height of the riverbed is still below that of the estimated stable riverbed in the Unazuki Dam Reservoir. Figures 5 and 6 demonstrate the shape of sedimentation in the Dashidaira Dam and the Unazuki Dam.

4) Duration of free flow

By using hydro data and hydro models derived from past floods that reached the standard flow rate, a simulation was run on the shape of the sedimentation and the grain size of the sediment as analytical factors to clarify the length of time taken for the target volume of sediment to flow down in a flood on a scale of 250-1,000 m^3/s, the figure the most likely to attain the standard flow rate. Based on this result, the duration of free flow for 2005 was determined as greater than 12 hours.

Table 1 Coordinated sediment-flushing plan

	Sediment flushing		Sediment sluicing	
	Dashidaira dam	Unazuki dam	Dashidaira dam	Unazuki dam
(1) Timing	•From June through August, flushing can be implemented in the first flood when either discharge into Dashidaira or Unazuki become larger than 300 or 400m³/s respectively. •Only in high discharges by snow melting or rainy season, flushing can be implemented in the first flood when discharge into Dashidaira becomes larger than 250m³/s. Flushing must be stopped when discharge becomes smaller than 130m³/s.		•From June through August, sluicing can be implemented in every flood when either discharge into Dashidaira or Unazuki become larger than 480 or 650m³/s respectively.	
(2) Quantity of sediment flushing	•Sediment exceeded the regular volume will be flushed out to maintain the regular sedimentation profile in a reservoir as much as possible.		•Natural floods with sediments will be passed through sediment flushing gate each time.	
(3) Flow type	•Draw-down and open channel flow		•the same as the left	
(4) Duration	• Period that is necessary to flush out sediment exceeded the regular volume to maintain the regular sedimentation profile in a reservoir as much as possible.		•Within the draw-down and open channel flow of Unazuki dam	•12 hours for draw-down and open channel flow
(5) Pre flushing / sluicing measure	•Opening a sediment flushing gate from the beginning of a flood, e.g. high water level.	•Drawing down from the latter stage of a flood control, e.g. high water level.	• the same as the left	
(6) Post flushing / sluicing measure	•In principle, stopping hydropower intake in 24 hours after the flushing and outflowing discharge as the same as inflow.	• In 24 hours after the flushing, outflowing discharge as the same as inflow from the dam and Unazuki power station.	• In 12 hours after the sluicing, outflowing discharge as the same as inflow from the dam and a downstream power station.	

5) Post flushing measure

With the aim of preventing local sedimentation along the downstream channel, the inflow in both the Dashidaira and the Unazuki Dams is continually discharged for 24 hours following the sediment flushing operations. In the Unazuki Dam, however, from 2005, an attempt will be made to promote the prevention of local sedimentation to a higher degree, with a small-scale flood of less turbid water entering the downstream channel through the effective use of dam capacity.

(5) Consensus building for the coordinated sediment flushing activity

Before performing coordinated sediment flushing operations, as Figure 7 illustrates, advance explanations and hearings are arranged for relevant parties including those from fishery and agricultural industries and administrative organs. Following this, discussions are held overseen by the third party "Kurobe River Dam Sediment Flushing Evaluation Committee," consisting of specialists in the fields of the environment, biology and fishery, and the "Kurobe River Sediment Management Council," made up of relevant municipalities and representatives of the Toyama prefectural government. At the same time, extensive use is made of public relations to disseminate information about the coordinated sediment flushing operations and to enable the local residents to gain a better understanding before the operation is begun.

(6) Environmental survey on the coordinated sediment flushing operation

The results of the coordinated sediment flushing operation are grasped and monitored through periodical investigations performed not only while the operation is in progress, but also before and after the operation (at ordinary times before and after the sediment flushing/sediment sluicing period). In addition, an extensive environmental survey is carried out during the sediment flushing/sediment sluicing operation. The findings are promptly made public and scientifically and objectively discussed and evaluated by the "Kurobe River Dam Sediment Flushing Evaluation Committee," which reports the results to the "Kurobe River Sediment Management Council." The relevant sediment flushing surveys are listed in Table 2, and Figure 8 illustrates points of observation in the environmental research.

4. Operational results of coordinated sediment flushing

(1) Achievements of coordinated sediment flushing

1) History of coordinated sediment flushing

As Table 3 shows, the first sediment flushing operation was performed in the Dashidaira Dam in 1991. From 1994 to 1995, test sediment flushing was carried out to collect data with the aim of further enhancing the accuracy and reproducibility of simulation models for contaminated substances and environmental loads in the river mouth and its surrounding coastal area, matters that were on the agenda of the Kurobe River Dashidaira Dam Flushing Evaluation Committee.

On the occasion of the July 1995 flood, nearly 3.4 million m³ of sediment had accumulated around the

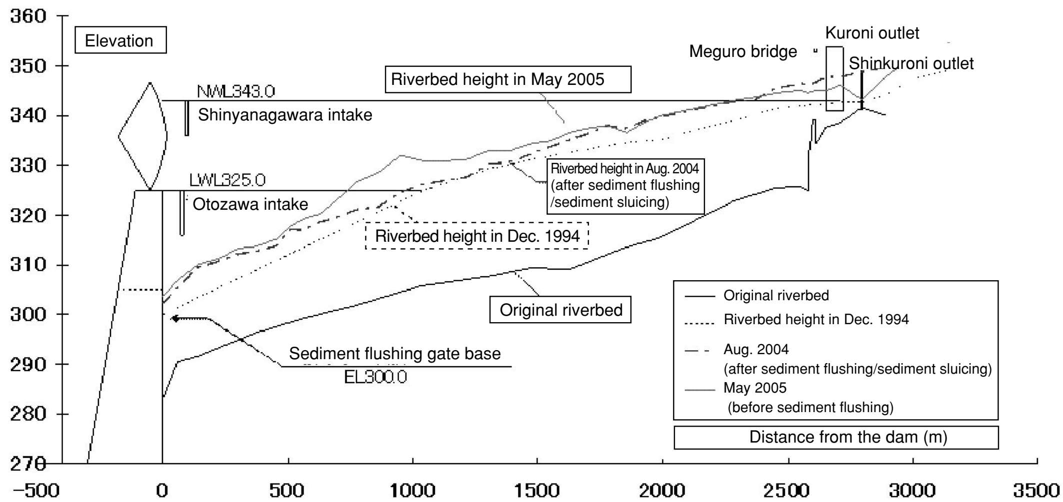

Figure 5 Sediment profile of the Dashidaira Dam (May 2005)

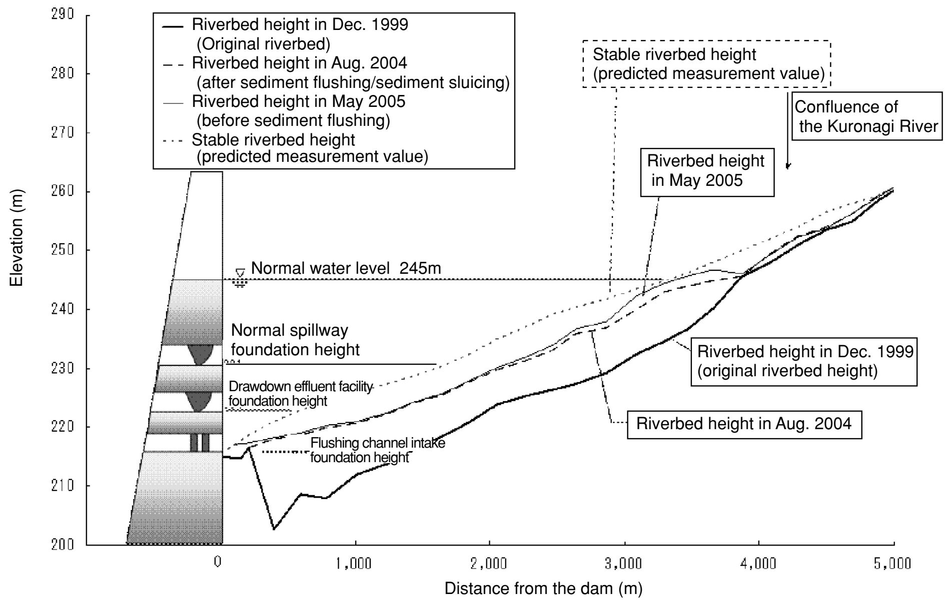

Figure 6 Sediment profile of the Unazuki Dam (May 2005)

Dashidaira Dam reservoir and about 6 million m³ in the middle course of the main part of the Kurobe River. Particularly in the Nekomata district upstream from the Dashidaira Dam, the riverbed had risen nearly 10 m, inflicting devastating damage on the Kurobe Gorge Railway and the Kansai Electric Kurobe River Power Plant No.2. Consequently, the "Coordinating Council for Kurobe River Disaster Restoration," was launched to discuss ways to prevent the recurrence of the disaster, to ensure the safety of the Nekomata district and to restore the Dashidaira Dam sediment into the state that had existed prior to the flood. On the basis of this agreement, emergency sediment flushing operations were carried out three times between 1995 and 1997 from the Dashidaira Dam.

Sediment flushing operations were implemented from the Dashidaira Dam alone for two years in 1998 and 1999 before coordinated sediment flushing operations started in 2001 with the completion of the Unazuki Dam.

2) Coordinated Sediment Flushing in 2004

From July 16, 2004, heavy rainfall stimulated by an active seasonal rain front continued over the upstream course of the Kurobe River.

As, by 9:00 PM, the inflow of the Dashidaira Dam had exceeded the standard flow rate for the scouring operation (250 m³/s in the rainy season), it was determined that coordinated sediment flushing operations would be carried out. Because the dam inflow rapidly increased as the rain intensified again during free flow operation, the coordinated sediment flushing operation was suspended at 12:00 AM on July 18. While flood treatment in the Dashidaira Dam and flood control in the Unazuki Dam were being carried out, it was determined that coordinated sediment sluicing would be

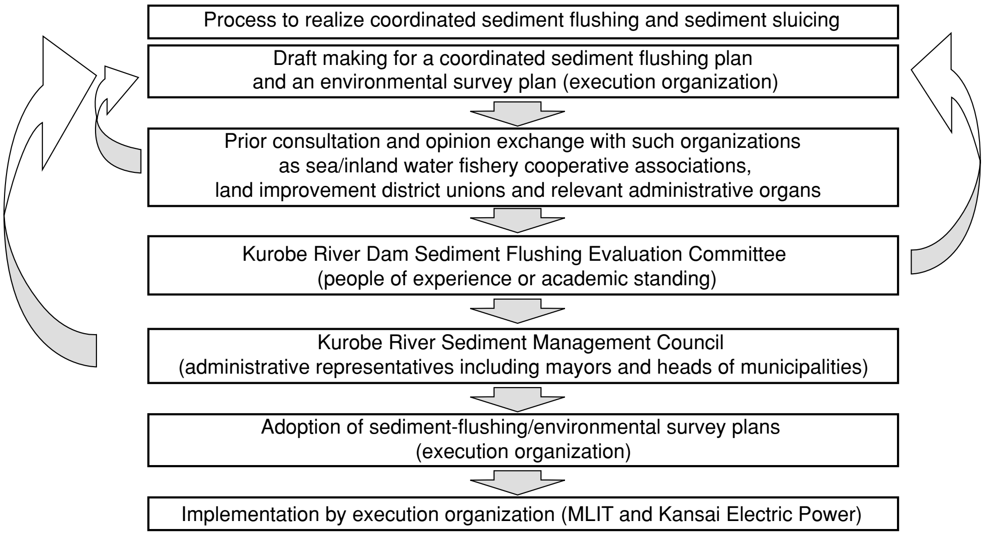

Figure 7 Process to realize coordinated sediment flushing and sediment sluicing

Table 2 List of environmental survey on sediment flushing

Month		4	5	6	7	8	9	10
Entire process				Sediment-flushing/sediment-sluicing operation period				
Research subjects			Regular research	During the sediment flushing operation			Regular research	
Dam lake	Water quality		●	●	One day after the sediment-flushing/sediment sluicing-operation		●	
	Sediment		●	●	One day after the sediment-flushing/sediment sluicing-operation		●	
River	Water quality		●	●	During and one day after the sediment-flushing/sediment-sluicing operation		●	
	Sediment		●				●	
	Aquatic organisms		●				●	
Irrigation channel	Sediment		●				●	
Coastal area	Water quality		●	●	During and one day after the sediment-flushing/sediment-sluicing operation		●	
	Sediment		●	●	One day after the sediment-flushing/sediment-sluicing operation (four major observation points)		●	
	Aquatic organisms		●				●	
Cross-leveling in the reservoir			●		● (implemented immediately after the sediment flushing operation)			

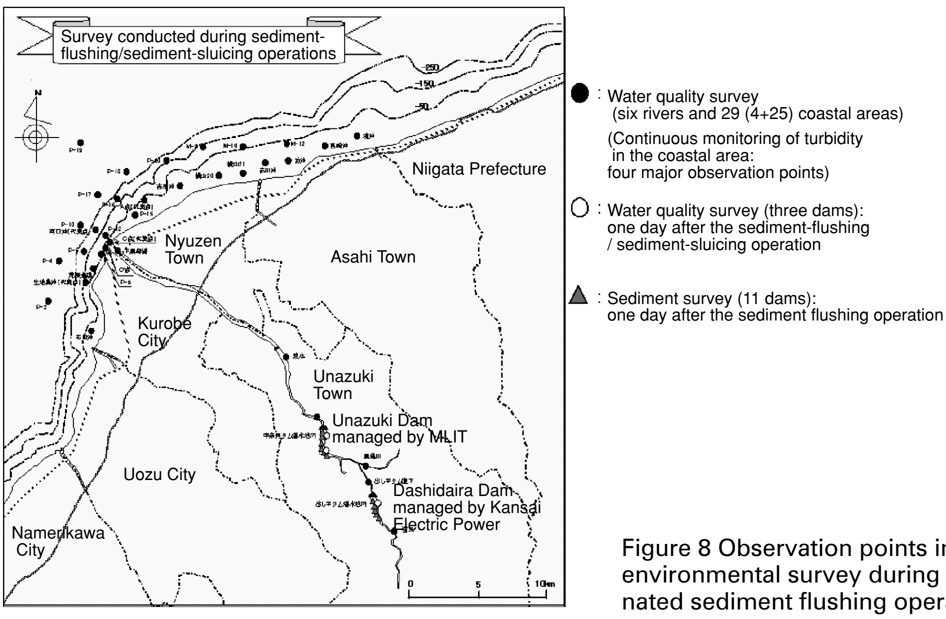

Figure 8 Observation points in the environmental survey during coordinated sediment flushing operations

Table 3 History of coordinated sediment flushing

Year	Quantity of sediment flushing (10,000 m^3)	Classification	Remarks
1991	46	Sediment flushing by the Dashidaira Dam alone	First sediment flushing
1994	8		Test sediment flushing
1995	2		
1995	172		Emergency sediment flushing
1996	80		
1997	46		
1998	34		
1999	70		
2000			No flood reaching the standard flow rate
2001	59	Coordinated sediment flushing	
2002	6		
2003	9		
2004	28		
2005	54		(Target volume)

conducted. The series of operations, from coordinated sediment flushing to coordinated sediment sluicing, lasted for 82 hours and 28 minutes until 10:22 AM on July 20. The volume of sediment discharged in 2004 amounted to 280 thousand m^3. Figure 9 and Photos 5 through 8 show the sediment flushing operations carried out in 2004.

(2) Results of environmental survey

The results of an environmental impact survey associated with the coordinated sediment flushing operation are reported to and evaluated by the Kurobe River Dam Sediment Flushing Evaluation Committee. Regarding coordinated sediment flushing operations up to this point, such comments were given as “no specific problems have been detected,” and “in general, there do not appear to be any problems resulting from environmental impact.”

In all kinds of environmental surveys, such elements as DO and SS are regarded as important indices in examining the impact on the environment. Figures 10 and 11 indicate the transition of SS and DO in the coordinated sediment flushing and sediment-sluicing operations for 2004, and Table 4 shows the maximum observation values in SS and minimum observation values in DO. Although the first sediment flushing operation of the Dashidaira Dam brought about a noticeable impact in that the DO value measured 0 mg/l in the area immediately below the dam, no conspicuous DO decrease was observed in the subsequent sediment flushing and sediment-sluicing operations. The SS values in the downstream course remained low for the incipient 2001 and 2002 coordinated sediment flushing operations, which may be because a great quantity of sediment discharged from the Dashidaira Dam had accumulated in the Unazuki Dam reservoir. As sedimentation in the Unazuki Dam reservoir has continued to accumulate, and the gradient has settled into shape, sediment is allowed to pass through smoothly and the trend of the SS value in the downstream course is to increase. However, the rise in the SS value is a temporary matter and regular follow-up surveys have observed no significant environmental impact.

The observed values for bottom COD and sulfide, the major marine indices, have not so far exceeded ordinary sludge levels in terms of the aquatic water use standard. Figures 12 and 13 demonstrate the survey results of COD and sulfide.

(3) Future challenges

In formulating the year-based coordinated sediment flushing plan, as stated earlier, interested groups and relevant organs are invited to give their opinions. Some suggest that the frequency of sediment flushing and sediment sluicing should be increased so that the discharge flow down occurs as naturally as possible. In response, opinions are exchanged and coordinated with the interested groups and relevant organs, while the effective standard flow rate and environmental impact on the downstream course are evaluated under the guidance of the Kurobe River Dam Sediment Flushing Evaluation Committee.

As it is expected that the discharge of sediment in the river channel and coastal area will incur some impact on the environment, and that there will be a temporary degradation of effluent water, including high turbidity as well as biological influence, the discovery of evaluation methods and formulation of mitigation measures require further research and continuous study. In addition to the data introduced in this paper, a substantial volume of monitoring data has been collected, and these have to be examined to analyze the impact on the environment of dam sediment flushing has to be examined.

At the same time, it is also important to evaluate the positive result of sediment flushing. In other words, dam sediment flushing not only realizes sustainable dam reservoir management, but also contributes to the maintenance of a healthy river and marine environment by continuously supplying soil to the downstream course. Quantitative evaluations are required in this regard.

In addition, it is essential to pay attention to the function of sediment to support the circulation of various substances down the river including nutrients, not merely as physical river and marine components in a narrow sense. The examination of impact mitigation measures is also required, and one of the concrete

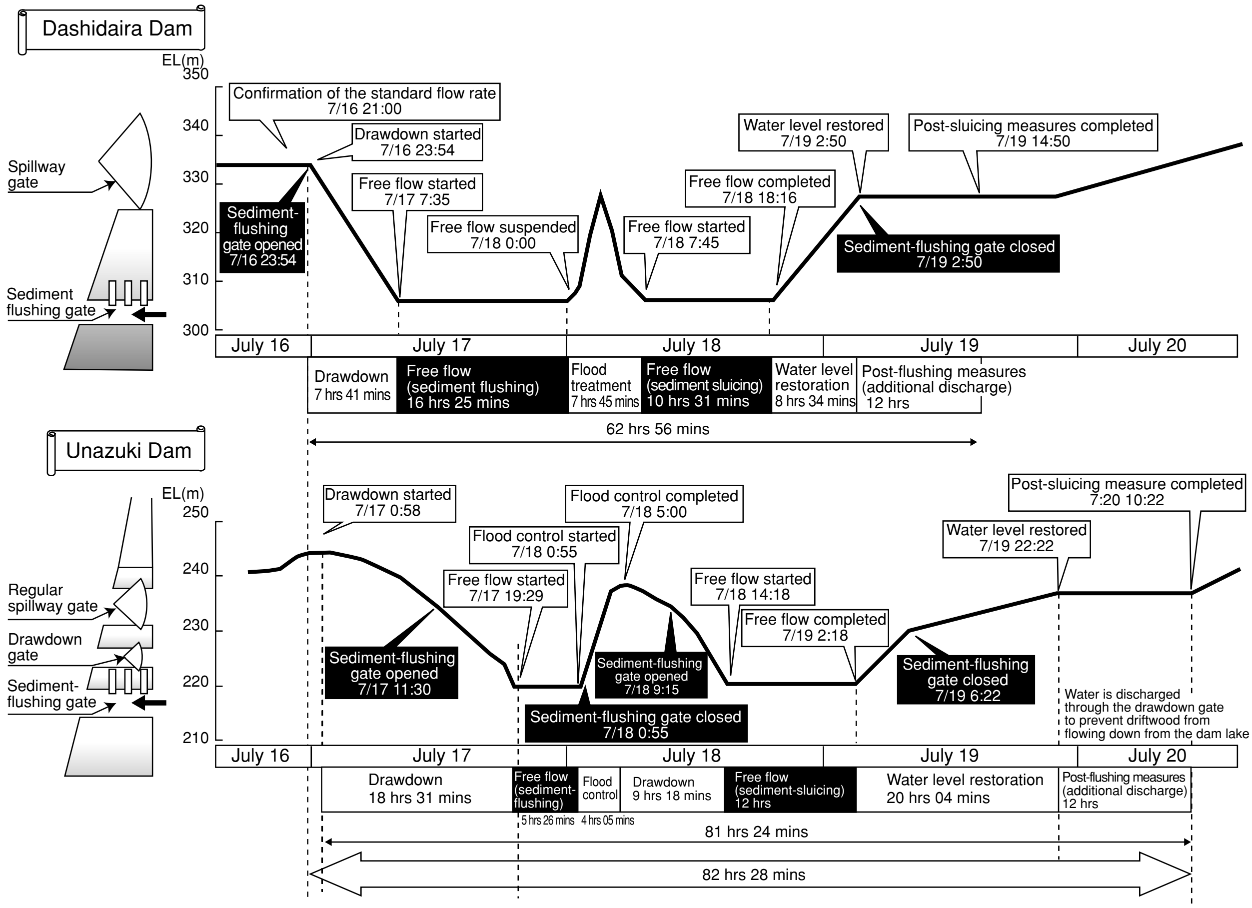

Figure 9 Schematic profile of the dam reservoir level during the 2004 coordinated sediment flushing operation

Photo 5 Dashidaira Dam
(during the free flow operation)

Photo 6 Unazuki Dam
(during the drawdown operation)

Photo 7 Kurobe River mouth during
the drawdown operation

Photo 8 Unazuki Dam reservoir during the
sediment flushing operation

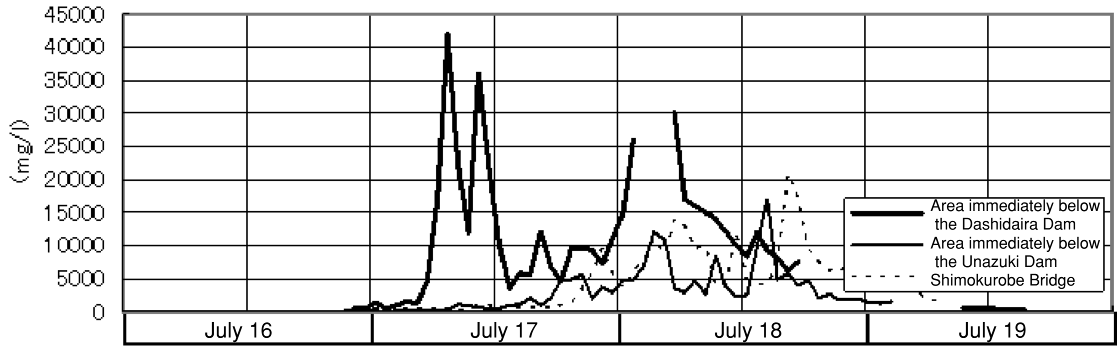

Figure 10 SS in the coordinated sediment-flushing/sediment-sluicing operations for 2004

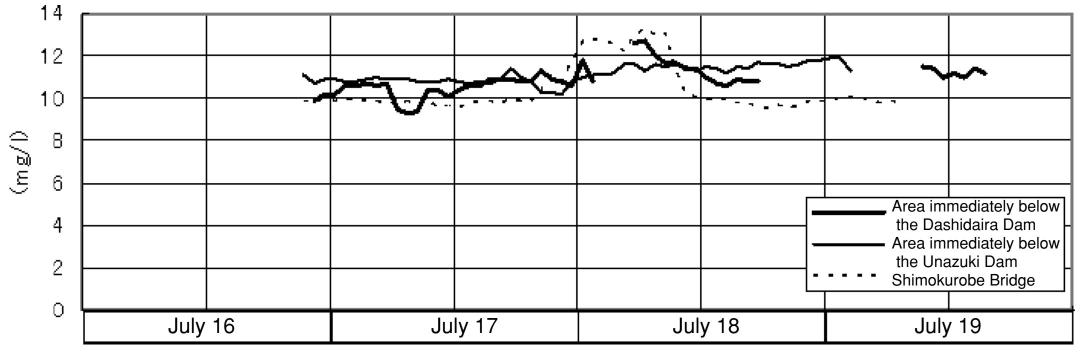

Figure 11 DO in the coordinated sediment-flushing/sediment-sluicing operations for 2004

Table 4 Maximum SS values and minimum DO values observed

Sediment flushing	SS (mg/l) (Maximum values during sediment flushing)			DO (mg/l) (Minimum values during sediment flushing)			Remarks
	Right below the Dashidaira Dam	Right below the Unazuki Dam	Shimokurobe Bridge	Right below the Dashidaira Dam	Right below the Unazuki Dam	Shimokurobe Bridge	
2004.7 Coordinated sediment flushing	16,000	17,000	21,000	10.6	11.2	9.6	
2004.7 Coordinated sediment flushing	42,000	6,800	11,000	9.3	10.2	9.8	
2003.6 Coordinated sediment flushing	69,000	17,000	10,000	11.8	11.3	9.6	
2002.7 Coordinated sediment flushing	22,000	5,400	2,800	9.5	10.5	9.5	
2001.6 Coordinated sediment flushing	29,000	3,700	2,200	11.1	10.6	9.6	First coordinated sediment sluicing
2001.6 Coordinated sediment flushing	90,000	2,500	1,500	7.2	11.4	10.2	First coordinated sediment flushing
1999.6	161,000	52,100	25,700	6.0	5.8	6.5	
1998.6	44,700	9,400	6,750	8.2	7.0	7.3	
1997.7	93,200	28,900	4,330	9.8	9.2	9.3	
1996.6	56,800	9,470	1,520	10.7	10.3	9.8	

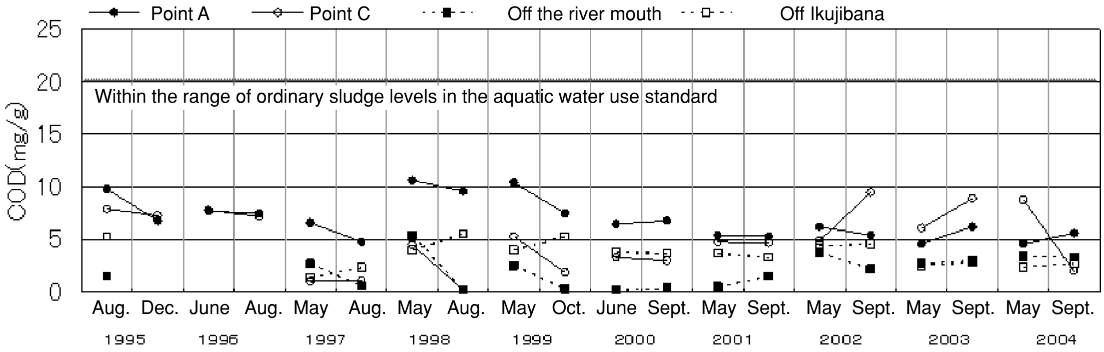

Figure 12 Marine sediment analysis results (COD)

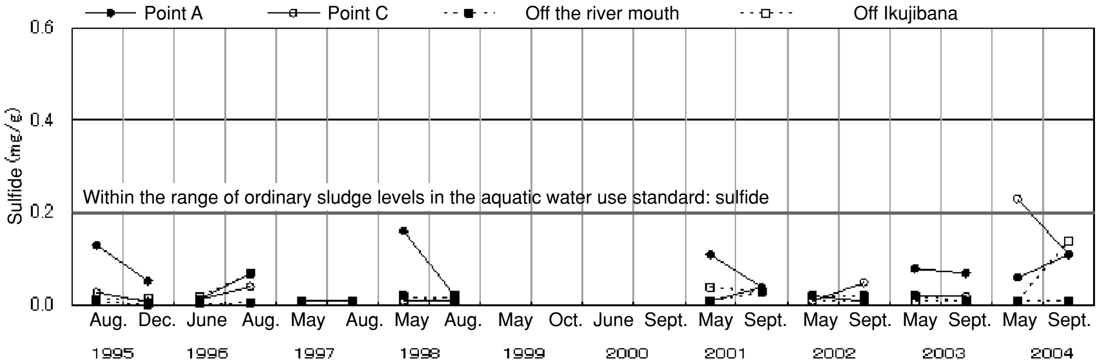

Figure 13 Marine sediment analysis results (sulfide)

Photo 9 Evacuation facility for fish
(resting water channel)

measures up to this point is the installation of an evacuation facility for fish, the (yasuragi suiro or resting water channel) shown in Photo 9. In this system, a foreland stream near the junction of the Kurobe main river and its branch river was improved to secure a freshwater zone into which fish can evacuate as the turbidity of the main river rises. As an evacuation facility for ayu (sweet fish), a positive result has been recognized.

5. Conclusion

The Kurobe River characteristically discharges a large volume of sediment and at the same time is susceptible to flood damage, and so, comprehensive sediment management is required to prevent flood disasters while at the same time maintaining the river's natural gravity flow mechanism. However, it is quite difficult to control sediment movement and a method of grasping soil dynamics as basic information has yet to be fully established. Therefore, at the same time as each sediment control measure is evolved, an attempt is made to elucidate the soil dynamics.

Regarding coordinated sediment flushing operations, environmental assessments and objective evaluations will be carried out to comprehend their impact on the environment. At the same time, sediment flushing methods will be examined and improved to find a method capable of reducing environmental impact to the minimum while the understanding of local residents will be deepened through the disclosure of information and public relations.

Study of Sanmenxia Reservoir Operation by "Clear Storage and Turbid Release" and Scour Sedimentation Equilibrium at Tongguan Riverbed

Chen Yingang, Wang Yujie
Sanmenxia Project Administration, YRCC, Sanmenxia, Henan, China

Abstract

The reservoir on the sediment-laden river is operated by the mode of "clear storage and turbid release" under such conditions that the sediment-laden reach in the reservoir reserves certain scouring capacity, a large release is available when the reservoir operates at low levels, sedimentation from return flows is controlled to an extent in the non-flood season, and large volume of floods are available in the flood season. Limitations are observed beyond these conditions. A basic relationship exists between Sanmenxia Reservoir's operation by "Clear Storage and Turbid Release" and scour sedimentation equilibrium at Tongguan riverbed, which critically depends on the contrast between the rise of the elevation $\triangle H_{natural}+\triangle H_{high\ level}$ at Tongguan in the non-flood season and the fall of the elevation $\triangle H_{flood}$. If floods are large in the flood season and the reservoir operation conforms to the above equilibrium conditions, the maximum storage level is not an important factor. In recent years, floods decrease considerably in the flood season, the fall of the elevation $\triangle H_{flood}$ induced by floods is smaller that the natural rise of the elevation $\triangle H_{natural}$ in the non-flood season, which is the essential for the increasing rise and remaining high of the elevation at Tongguan. The variation of the water-sediment regime should not lead to the denial of the scientific value and importance of the sediment-laden reservoir being operated by "Clear Storage and Turbid Release". Water levels of the reservoir need to be adjusted during sediment releasing in the flood season to maximize the potential of sediment releasing and enlarge the retrogressive scour and have it joined with the scour along the route.

Key Words: *Sanmenxia Reservoir, Clear Storage and Turbid Release, Riverbed Scour, Elevation at Tongguan, High Water Level, Sediment into Reservoir, Flood Volume*

1 Introduction

The Yellow River is a known sediment-laden river. Since 1950s, the Government has paid great attention to the river's management, and the construction of the first large reservoir, Sanmenxia Reservoir, started in the end of 1950s. At that time when people were overly optimistic about the fast progress of soil and water conservation and the effect of sediment retaining, retaining was highly addressed and releasing was neglected, after impounding in September of 1960, sedimentation had severely increased and extended upstream, resulting in rapid constriction of reservoir storage, a large rise of the riverbed at Tongguan, occurrence of inundation and alkali saline soils, affecting the flood control in the lower reaches of the Weihe River and the safety of Xi'an City. Therefore, since the mid of 1960s, enlargement, reconstruction and exploration of operation mode have been carried out for decades. The mode of CSTR (clear storage and turbid release) came into operation for the reservoir, and in the years of 1973 to 1986, sediment and water dispatching had been carried out quite well with the EAT rise and fall relatively balanced (EAT is the Elevation At Tongguan,is water level equivalent to discharge of 1000 m^3/s at Tongguan section. Basically has reflected the fluctuation situation of Tongguan riverbed). Since 1986, particularly the late of 1990s, flood volume has been reduced greatly in the flood season, sedimentation has increased, the EAT has been rising intemittently and remaining high, so that Sanmenxia Reservoir CSTR operation mode is facing new problems. The thorough study and exploration of the law about the quantitative relationship between operation indices and scour sedimentation equilibrium at Tongguan is significant to further understanding of the operation of Sanmenxia Reservoir as well as the reservoirs on other sediment-laden rivers in the world.

2 Conditions for CSTR Operation and Scope of Application

2.1 Conditions for CSTR Operation

On the basis of the river sediment theory and experience of Sanmenxia Reservoir in over forty years, three points are summarized for the conditions of the operation mode:

(1) A certain scouring capacity available in the sedimentation reach in the reservoir

In the early 1970s, with reference to the mechanism that the longitudinal gradient of the sediment-laden river channel adjusts itself along with water and sediment inflows and the variation of boundary conditions, the sediment sector believed that for the conditions for such an operation mode, a certain scouring capacity should be available in the sedimentation reach in the reservoir, i.e. the gradient J_s sediment transport balanced is obviously smaller than the gradient J0 before the existence of the reservoir (J_s/ J_0 is much bigger than 1).

(2) A large scale of discharge is available when the reservoir is operated at low levels

Theoretical analysis and the practical operation of Sanmenxia Reservoir show that a certain scouring capacity in the sedimentation reach is far enough, and the large scale flow and sediment releasing facilities are necessary when the reservoir is operated at low levels, that is, a relatively large capacity of "turbid flow discharge" is needed.

(3) Sedimentation from flow return of the reservoir is controlled to an extent in the non-flood season, and large floods are available in the flood season

First, when the reservoir storage is at high levels in the non-flood season, sedimentation from flow return of the reservoir shall be controlled to an extent for the scour in the flood season. Second, fluvial dynamics tells that the creation of riverbed is in direct proportion to the big number of flows. Only come large floods in the flood season, the said operation is able to discharge the sediment out of the reservoir being deposited in the non-flood season, presenting and maintaining annual ne long-term scour sedimentation equilibrium in the reservoir. Otherwise, sedimentation would be accumulated and no equilibrium seen.

2.2 Successful Practice of CSTR by Sanmenxia Reservoir

Sanmenxia Reservoir experienced a process full of twists and turns regarding its operation. In September of 1060, the project was initially completed and began operation by "impounding water and trapping sediment" with the maximum level at 332.58m (February 9, 1961). Thus, from September of 1960 to March of 1962, flow return passed even Tongguan, and the releasing discharge was very small (only 3084 m^3/s at 315m), particularly, the lack of outlet facilities at low levels was found, only 13% of sediment was removed as density flows. Sedimentation was serious in the reservoir, the Tonguan stage rose by 4.5m, and the storage of 1.7billion m^3 below 335m was trapped.

Therefore, after April of 1962, the operation of "flood retention and sediment discharge" had to get into effect. In the first years, only 12 original opening worked for flow discharge. Although gates were opened and flood release increased from 6.8% to 58%, alleviating sedimentation in the reservoir, outlet structures were inadequate and the sills at the opening were high so that the plentiful flow and sediment of 1964 brought about severe sedimentation. Sedimentation continuously developed upstream. With enlargement and reconstruction of outlet structures, the discharging capacity strengthened, a scour of 400 million m^3 took place downstream Tongguan with the EAT down by 1.9m. In October of 1971, the outlet structures were modified again with the original 12 openings, additional two tunnels, three sediment releasing steel pipes, eight sediment outlets, which reached discharge of 9059 m^3/s at 315m.

For further exploration of a scientific operation mode for the reservoir, CSTR began in November of 1973, i.e. when sediment inflow is low in the non-flood season, water is stored for ice control, irrigation in spring, power generation; and in the flood season, particularly, with flood events, the reservoir level is lowered to discharge flood and sediment being deposited in the non-flood season out of the reservoir, realizing a regulation of both water volume and sediment. From November of 1973 to October of 1985, the maximum reservoir level reached 326m and 324m respectively for ice control and irrigation in spring. Although the high water levels kept for a long period, and sedimentation from flow return passed Tongguan, flood inflows were at high level and of large volume, CSTR played a role, and scour and sedimentation relatively balanced at Tongguan. The EAT at the section rose and fell between 326.64 and 327.20m. By the end of the flood season in 1985 and 1973, the EAT was respectively 326.64m.

2.3 Scope of Application and Limitations of CSTR to Reservoir on Sediment-laden Rivers

The conditions for operation of CSTR summarize the scope of application and possible limitations in practices; with the conditions varied, operation of CSTR will not effectively realize the scour sedimentation equilibrium. The experience of Sanmenxia Reservoir has proved for over a decade.

The experience of Sanmenxia Reservoir is as follows. After 1986, especially 1990s, inflow conditions obviously changed in the flood season, and floods largely reduced, which led to the increasing rise of the water level and remaining high at Tongguan. During this period, openings nos. 9 and 10 were added in 1990, the maximum reservoir level reached 322m for ice control and irrigation in spring in the non-flood season after 1993, dredging had been gradually enhanced after 1997, openings nos. 11 and 12 were added in 1999 and 2000, the maximum reservoir level reached 320.5m for ice control and irrigation in spring in the non-flood season after 2000, the discharge capacity exceeded 3600m^3/s below 300m, being favoring the reduction of siltation at Tongguan riverbed, still completely keeping within limits of the water level at Tongguan. The EAT reached 328.30m by the end of the flood season in 2001. From the end of the non-flood season to the beginning of the flood season in 2002, with successive small but sediment-laden floods in the Weihe River, and the reservoir was operated at low levels, serious sedimentation and a large rise (about 0.8m) occurred at the Tongguan reach. The EAT created a historical record of 329.20m, did not fall to that before the large rise by the end of the flood season.

The operation of CSTR once successfully applied by Sanmenxia Reservoir meets new problems, and what is the cause? Could this deny the scientific value and effect of the operation mode for the reservoir on the sediment-laden river? The author believes that further analysis and study are necessary in view of the quantitative relationship between the operation and the scour sedimentation equilibrium at Tongguan.

3 Relationship between the Operation and the Equilibrium

3.1 Relationship between the High Storage Operation and Tongguan Riverbed Rise in Non-flood Season

A large quantity of historical investigation data and studies show that before the Sanmenxia Reservoir is built, the variation of the EAT features as: (1) The law of "going up in flood season and coming down in non-flood season" is presented in the year; (2) The rise in the non-flood season is greater than the fall in the flood season, indicating a slight ascend in the historical period. For example, from the Three States period (years 155 to 220) to the reservoir construction (1960), a 14m thick medium and fine silt layer[1] had deposited at Tongguan; in the twenty years before the construction, the average rise was 0.35m in the non-flood season and the average fall was 0.28m in the flood season, balancing an average rise of 0.07m[2]. That is, in the thousands of years before the reservoir, without the effect of sedimentation from backwater of the reservoir, both in the non-flood season and in the historical period, the riverbed had risen mainly because the variation of sediment and water regime caused scour and sedimentation and adjustment of the riverbed.

With the project completed, when the reservoir is operated at high water levels in the non-flood season, the EAT rising has multiple reasons, including the unfavorable effect by sedimentation from backwater with high water level operation.

Under certain conditions, the rise △H of EAT approximates the thickness △h of the sedimentation mass near Tongguan, i.e. △H ≈ △h. The length L between sections closely downstream Tongguan and average width B vary relatively less, and suppose the volume △V of sedimentation mass of a river reach closely downstream Tongguan, and density ρ of sedi-

Table 1 Quantitative Correlation between EAT Rise △H and the Operation Factor W_s (at and over a water level) in the non-flood season

Year	ΔH	$Ws_{\geq 324}$	$Ws_{\geq 323}$	$Ws_{\geq 322}$	$Ws_{\geq 321}$	$Ws_{\geq 320}$	$Ws_{\geq 319}$	$Ws_{\geq 318}$	$Ws_{\geq 317}$	$Ws_{\geq 316}$	$Ws_{\geq 315}$
1973-197	0.55	1232	3325	6560	8992	10630	10862	11320	11485	11714	11918
1974-197	0.53	0	3403	4314	4889	5750	7083	7825	8849	9428	9778
1975-197	0.67	471	911	1096	1852	4531	7056	7456	8630	9336	9989
1976-197	1.25	4720	6415	6581	6763	6943	7220	7505	7861	8513	9071
1977-197	0.51	677	1424	3466	4966	5390	6686	7005	7767	9113	9417
1978-197	0.67	2198	2796	6444	6953	7866	9020	9584	9859	10144	10776
1979-198	0.20	0	2010	2395	3796	5347	6693	6754	6915	8247	9006
1980-198	0.24	0	1027	2127	2840	4311	5447	5551	5628	5783	6046
1981-198	0.50	0	2191	4270	5268	5966	6698	7836	8613	9147	10083
1982-198	0.33	0	3961	6397	6838	7735	8886	9533	9962	10517	10598
1983-198	0.61	594	2048	2856	4717	5676	6360	7002	7568	7797	7941
1984-198	0.21	429	1401	1844	2158	2474	7031	7679	8507	9296	9835
1985-198	0.44	0	0	368	667	1055	3312	4445	4935	5784	12799
1986-198	0.12	0	1538	1985	2407	4029	5083	5862	6431	7686	8127
1987-198	0.21	152	1647	2173	3847	4309	5261	5629	6378	6841	6933
1988-198	0.54	695	5461	5999	6281	6800	9236	10428	11373	12135	13187
1989-199	0.40	0	4461	5354	6078	9103	10020	10884	11271	12548	15011
1990-199	0.42	0	1637	2571	3049	3566	3595	7230	8205	14216	14697
1991-199	0.50	0	2082	4238	7459	7866	8232	8823	9760	12887	13302
1992-199	0.46	0	0	0	1351	3488	4627	5891	6968	7483	13611
1993-199	0.17	0	0	1169	3206	3932	5966	6489	7235	8350	11129
1994-199	0.43	0	0	0	1679	2498	3722	5467	8561	10779	12110
199519-9	0.18	0	0	0	3546	5102	6092	6475	7682	9418	12953
1996-199	0.17	0	0	0	1358	3519	5165	5543	7309	7556	7675
1997-199	0.26	0	672	2237	6153	11279	15181	16510	17642	18153	18763
1998-199	0.34	0	0	0	0	873	3373	7521	9803	11259	12459
Average	0.42	429.5	1862	2863	4120	5386	6843	7779	8661	9774	11050

Note: ΔH、Ws respectively in metre and ten thousand ton.

The simple relation between EAT rise □H and sediment inflow W_s at and over a water level (≥315m, 316m,…324m) in the non-flood season has been analyzed (see table 2).

Table 2. Regressive Analysis Statistics between
EAT Rise $\triangle H$ and Sediment Inflow W_s at and over a Water Level in the Non-flood Season

Sediment Inflow Ws ≥ a Water Level	Correlation Coefficient	Marked Level	Intercept	Regressive Coefficient
$Ws_{H\geq324}$	0.8171	48.20	0.338	0.1912
$Ws_{H\geq323}$	0.6028	13.70	0.270	0.0805
$Ws_{H\geq322}$	0.5363	9.69	0.261	0.0552
$Ws_{H\geq321}$	0.4100	4.85	0.250	0.0411
$Ws_{H\geq320}$	0.2691	1.87	0.289	0.0242
$Ws_{H\geq319}$	0.1389	0.47	0.334	0.0124
$Ws_{H\geq318}$	0.1284	0.40	0.325	0.0121
$Ws_{H\geq317}$	0.0907	0.20	0.345	0.0086
$Ws_{H\geq316}$	0.0524	0.07	0.375	0.0045
$Ws_{H\geq315}$	0.0056	0	0.414	0.0005

mentation mass, thickness is then calculated by $\triangle h = \triangle V / BL$. As $\triangle V$ is related to sedimentation weight W_s in a certain time duration, i.e. $\triangle V = W_s / \rho$, it is then $\triangle H \approx \triangle h = \triangle V / BL = W_s / \rho BL$, in which ρ, B, L are constants, sediment inflow (or sedimentation weight) W_s in a certain duration with high reservoir level is the critical factor that causes the EAT rise, and $\triangle H$ and W_s are basically in linear. Table 1 shows the correlation between $\triangle H$ and W_s at and over a water level.

Based on the mathematic analysis, correlation coefficients of sample and population are not always identical. For two random variables whose populations are disrelated ($\rho=0$), because of sampling, correlation coefficient r is not always zero, may be greater than zero, or another random variable. Therefore, it shall be checked by some rules.

The analysis included 26 samples from the data of the non-flood season. According to the rules and standards for judging and checking their correlations, an with different levels of truth ($\alpha=0.01, 0.02, 0.05, 0.10$), $\triangle H$ and Ws have the minimum coefficient of simple relation r_d respectively 0.4994, 0.4569, 0.3914 and 0.3324 (see Figure 1).

Based on the results in Table 2, correlation coefficients of $\triangle H$ and W_s can be transferred to a curve of correlation coefficients showing $\triangle H$ and operation at and above a water level (see Figure 2).

Figures nos.1 and 2 result in the data in row 3 of Table 3, showing, with different levels of truth, $\triangle H$ and W_s are correlated at corresponding water levels, that is, when checking correlation coefficients of samples and determining population correlation, if $\alpha=0.01$, 0.02, 0.05, 0.10, W_s and $\triangle H$ are correlated at different critical water levels, respectively 322.24, 321.83, 321.21 and 320.56m. With maximum reservoir levels strictly controlled, and with specifications and precise judgment and check standards, the reservoir level below 320m in the non-flood season basically eliminates the effect of reservoir operation on sedimentation at Tongguan. Thus, both the EAT rise in the non-flood season and the EAT accumulated rise in the year are due to the same causes in comparison to that in nearly twenty years before the project.

According to the 1973~1999 year material in non-flood season, the comes water volume and sedimentation weight, the EAT in Early Flood Season and in Late

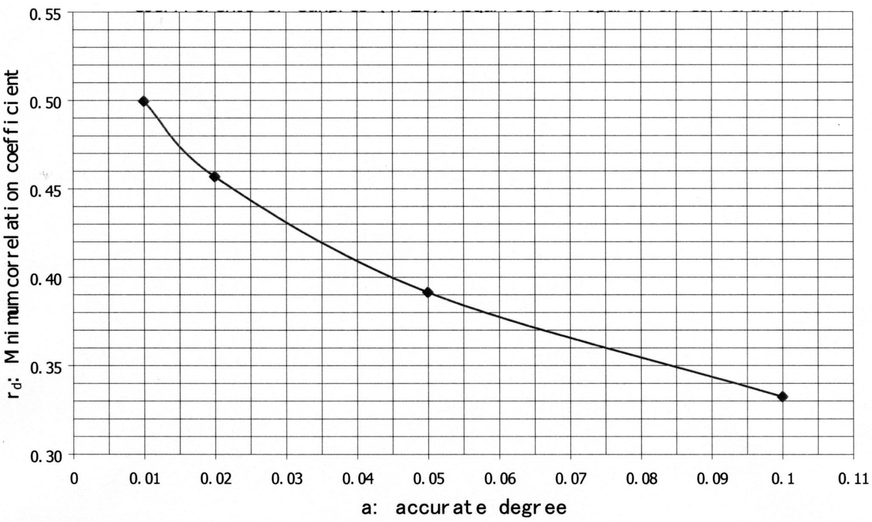

Fig. 1. With different Accurate Degrees, Minimum Correlation Coefficients of Samples (n=26) Required by Population Correlation

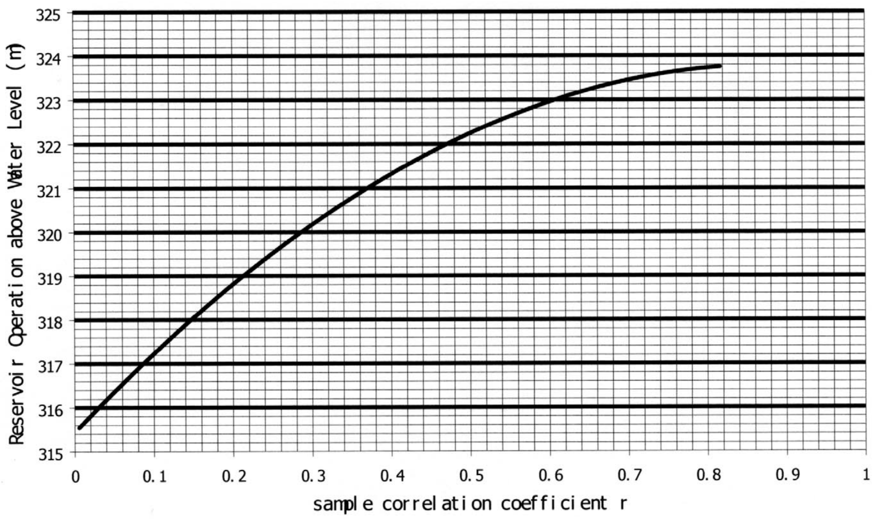

Fig. 2. The EAT Rise △H and Reservoir Operation above a Water Level (Sediment Inflow) Correlation coefficient Curve

Table 3. With Different Accurate Levels, the minimum simple correlation coefficient, r_d, and Corresponding Reservoir Levels

Different Accurate Level α	0.01	0.02	0.05	0.10
minimum simple correlation coefficient r_d	0.4994	0.4569	0.3914	0.3324
Ws and △H are correlated at water levels (m)	322.24	321.83	321.21	320.56

Table 4. Statistics of EAT Falls, △H, and Flood Volume, W, above a Discharge in the Flood Season

Year	EAT Early Flood Season	EAT in Late Flood Season	Fall △H	$W_{Q\geq1500}$	$W_{Q\geq2000}$	$W_{Q\geq2500}$	$W_{Q\geq3000}$
1974	327.19	326.70	0.49	47.8	26.9	19.3	5.4
1975	327.23	326.04	1.19	294.7	270.8	231.2	181.6
1976	326.71	326.12	0.59	301	255	219	196
1977	327.37	326.79	0.58	105.2	68.8	45.7	36.4
1978	327.30	327.09	0.21	186.6	151.2	112.2	82
1979	327.76	327.62	0.14	184	159	105	62
1980	327.82	327.38	0.44	54.4	28.1	4.7	0
1981	327.62	326.94	0.68	323.6	303.5	263.6	223.6
1982	327.44	327.06	0.38	133	93.3	39.9	15.1
1983	327.39	326.57	0.82	305.9	275.8	245	209.1
1984	327.18	326.75	0.43	269	227	183	152
1985	326.96	326.64	0.32	197	164	128	114
1986	327.08	327.18	-0.10	64.1	43.9	31.9	11.3
1987	327.30	327.16	0.14	9.1	5	3	3
1988	327.37	327.08	0.29	128	96.9	72.6	45.7
1989	327.62	327.36	0.26	161	141.3	125.8	70.6
1990	327.76	327.60	0.16	75.4	22	12.7	3.5
1991	328.02	327.90	0.12	5.3	2.3	2.3	0
1992	328.40	327.30	1.10	69.4	48.3	26.7	12.2
1993	327.76	327.78	-0.02	77.3	44	10.5	5.8
1994	327.95	327.69	0.26	61.4	34.6	24.5	19.7
1995	328.12	328.24	-0.12	49.8	18.8	7.5	5.3
1996	328.42	328.07	0.35	72.5	35.2	13.1	10.6
1997	328.24	328.02	0.22	8	5	3.1	3.1
1998	328.28	328.12	0.16	30.4	17.8	6.4	4
1999	328.46	328.12	0.34	17.2	8.3	0	0

Flood Season, along with Tongguan-Guduo section of river gradient and so on. The union analysis result (see Table 2), under the condition that high level operation of Sanmenxia reservoir in the non- flood season (surpasses 320.56m), The corresponding backwater silts volume (or sedimentation weight Ws) to the EAT rise quantity △H influential physics statistical model is:

$$\triangle H = 0.250 + 0.0411 W_{s \geq 321} \quad (1)$$

$$\triangle H = 0.261 + 0.0552 W_{s \geq 322} \quad (2)$$

$$\triangle H = 0.270 + 0.0805 W_{s \geq 323} \quad (3)$$

$$\triangle H = 0.338 + 0.1912 W_{s \geq 324} \quad (4)$$

3.2 Relationship between Flood Inflow and Fall of Tongguan Riverbed

For sediment-laden rivers, the riverbed is created mainly during flood events, flood volume and flood peak discharge are two major physical factors facilitating scour of the riverbed and EAT fall. As the actual flood movement consumes energy in a very complicated way, it is difficult to establish a simple expression about riverbed scour and sedimentation and affecting factors based on conversation of quality and energy equations. If the flood mass entering the Tongguan section at a unit time in a flood event is regarded as a whole or system, internal force is not considered, only in view of the total scouring force and impulse onto the whole riverbed, it is believed that scour of the riverbed mainly relates to impulse of flows, the expression thus becomes simple.

Suppose the average scouring force onto the riverbed at a unit time in a flood event is F, total time is T, flood mass entering the Tongguan section at a unit time is M, average discharge is Q, average density of flood mass is ρ, sloping angle of Tonggu-Guduo section in the beginning of the flood is α, and gradient is J, total scouring impulse is $P=F\times T=Mg\sin\alpha\times T=Q\rho g\sin\alpha\times T$.

As $Q\times T=W$; α is small, $\sin\alpha\approx tg\alpha=J$; when sediment content, S (kg/m^3), is bigger, ρ is expressed as $1+0.000624S$, total impulse, $P=W(1+0.000624S)gJ$. As g is gravity acceleration (constant), impulse factor P is expressed as $W(1+0.000624S)J$.

The author made a great quantity of analysis about the relationship between △H and $W(1+0.000624S)J$, and found that they two are closely related, particularly close between △H and W. Table 4 only lists the EAT fall, △H, and flood volume, W, above a discharge in the flood season of 1974 to 1999.

At Tongguan riverbed in the flood season, scour and sedimentation feature as: (1) Scour happens in the general food process; when severe sedimentation occurs during falling of water levels, slight sedimentation occurs through the flood process. (2) When small (or normal) floods contain a large proportion of sediment (or which lasts a long time), i.e. when sedimentation is greater than scour, it presents slight sedimentation in the whole flood season. The EAT variation basically conforms to the above situation, and Table 4 shows the law. For reflecting the average regime in the flood season, special locations (slight sedimentation) are not excluded.

Generally speaking, flows less than 1000m^3/s is unfavorable to the EAT in the flood season. According to the data of 1974 to 1999, without variation of S and J and their effect on Tongguan, regressive calculation results are shown in Table 5. Based on the data in Table 5, the EAT falls, △H, and flood volume, W, above a discharge (1500m^3/s、2000m^3/s, 2500m^3/s, 3000m^3/s) are related in expressions nos. 5, 6, 7 and 8.

3.3 EAT to Operation at High Storage Level in Non-flood Season and Flood Volume in Flood Season Relationship

As summarized above, the CSTR operation of Sanmenxia Reservoir and the EAT equilibrium is concluded as a relation between natural rise of the EAT, ($H_{natural}$, rise from high water level operation, ($H_{high\ level}$, and fall from scour in flood season (H_{flood}. There are four cases:

(1) If $\triangle H_{flood} > \triangle H_{natural} + \triangle H_{high\ level}$, and even the reservoir is operated by the CSTR rule, the EAT will decline;

(2) If $\triangle H_{flood} = \triangle H_{natural} + \triangle H_{high\ level}$, the reservoir being operated by the CSTR rule will lead to a relative equilibrium of the EAT rise and fall;

(3) If $\triangle H_{natural} < \triangle H_{flood} < \triangle H_{natural} + \triangle H_{high\ level}$, the EAT may go to ascend, by lowering the maximum operation levels in the non-flood season, the EAT rise and fall will become balanced.

(4) $\triangle H_{flood} < \triangle H_{natural}$, whether the operation levels

Table 5 The EAT falls , △H, and flood volume, W, above a discharge in the flood season of 1974 to 1999, Regressive Analysis Statistics

Flood Volume, W	Correlation Coefficient	Marked Level	Intercept	Regressive Coefficient
$W_{Q\geq1500}$	0.5331	9.53	0.156	0.00166
$W_{Q\geq2000}$	0.5494	10.38	0.186	0.00181
$W_{Q\geq2500}$	0.5570	10.79	0.209	0.00206
$W_{Q\geq3000}$	0.5590	10.91	0.226	0.00241

$$\triangle H = 0.156 + 0.00166 W_{Q\geq1500} \quad (5)$$

$$\triangle H = 0.186 + 0.00181 W_{Q\geq2000} \quad (6)$$

$$\triangle H = 0.209 + 0.00206 W_{Q\geq2500} \quad (7)$$

$$\triangle H = 0.226 + 0.00241 W_{Q\geq3000} \quad (8)$$

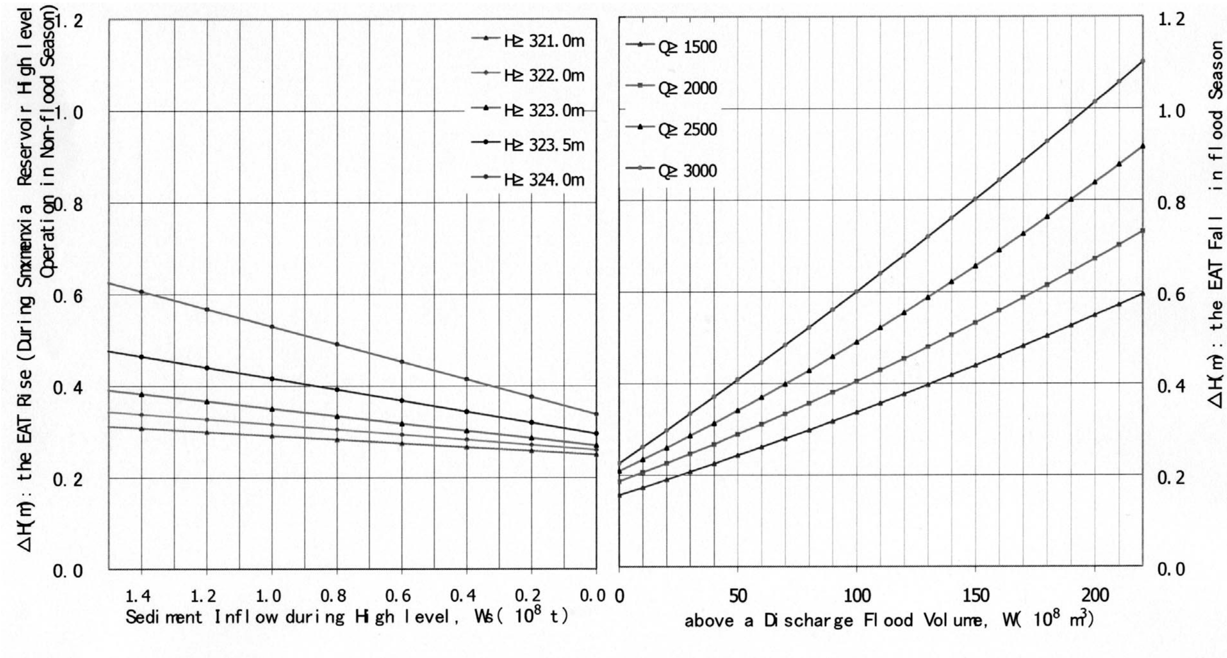

Fig. 2. The EAT Rise △H and Reservoir Operation above a Water Level (Sediment Inflow) Correlation coefficient Curve

rise or fall, the EAT will go to ascend.

According to the expression nos. 1, 2, 3, 4, 5, 6, 7 and 8, a linear diagram is derived for the reservoir high level operation, flood volume in flood season and EAT. Except for the two factors of sediment inflow above a reservoir level in the non-flood season and flood volume above a discharge, without considering the increase of sediment inflow above a reservoir level leading to slight reduction of the rising rate of the EAT, with considering for the same flood volumes, increasing discharge leading to gradual increase of the falling rate of the EAT (i.e. sediment-laden large floods have strong scour, at present, Tongguan-Shijiatan section has a larger gradient and the project has a stronger outlet capacity), the linear diagrams corresponding to the expression nos. 5, 6, 7 and 8 are corrected by using relevant characteristic values, and Figure 3 is obtained for the comprehensive relationship between the reservoir operation level, flood value and balance of the EAT (see Figure 3).

Generally speaking, if the numerical values in Figure 3 are neglected, the figure reflects the basic relationship between the CSTR operation of the general sediment-laden river reservoir and scour and sedimentation equilibrium in the reservoir.

For Sanmenxia Reservoir, such a relationship must not simply determine the maximum operating level, but reflect the contrast between $\triangle H_{natural} + \triangle H_{high\ level}$ in the non-flood season of a year or many years and $\triangle H_{flood}$, the EAT fall in the flood season. If the relationship between the equilibrium conditions is met, the maximum operating level will not be critical; when in the non-flood season, the highest operating level has minimum effect ($\triangle H_{high\ level}=0$), with flood volume being reduced in the flood season, $\triangle H_{flood}<\triangle H_{natural}$, i.e. it fails to meet the equilibrium conditions, and it is also not favorable to make the EAT fall simply by controlling the operation of the reservoir.

3.4 Reasons of the EAT Remaining High in Recent Years

Observations show that from 1974 to 1985, flood volumes with discharge over 1500m^3/s, 2000m^3/s, 2500m^3/s and 3000m^3/s reached respectively 20.0 billion m^3, 16.8 billion m^3, 13.3 billion m^3 and 10.6 billion m^3, being close to the original design volumes (annual flow about 25.5 billion m^3 in the flood season and flood volume about 20.0 billion m^3 in the flood season). Even if the highest reservoir levels are respectively at 326m and 324m for ice control and irrigation in spring, as the reservoir reserves some scouring capacity, practices have proved that the operation of the reservoir by the CSTR mode is able to realize the scour and sedimentation equilibrium and a relatively steady stage at Tongguan.

From 1993 to 1999, flood volumes with discharge over 1500m^3/s, 2000m^3/s, 2500m^3/s and 3000m^3/s reached respectively 4.52 billion m^3, 2.34 billion m^3, 0.930 billion m^3 and 0.693 billion m^3. Since 2000, inflow discharge over 2000m^3/s creates flood volume less than 0.7 billion m^3. That is why the maximum reservoir level has been down to below 320.5m, but the EAT remains increasing or still high.

4 Conclusions

(1) The CSTR operation of Sanmenxia Reservoir and the Scour and sedimentation equilibrium at Tongguan is related by critically depending on the contrast between the annual and long-term rise of the EAT in the non-flood season, $\triangle H_{natural} + \triangle H_{high\ level}$ and the

EAT fall in the flood season, ($\triangle H_{flood}$. When the equilibrium conditions are met, the maximum reservoir level will not be a critical factor.

(2) In recent years, the big reduction of flood volume is the basic reason why the EAT remains high, that is, the fall due to floods, $\triangle H_{flood}$, is smaller than the rise in non-flood season, $\triangle H_{natural}$. The sedimentation and rising of the riverbed in the Weihe Lower Reaches are also caused by obvious reduction of flood volume and obvious increase of sediment concentration in the last twenty years.

(3) The increasing decline of sediment and water regime in the Yellow River Basin has been the basic and common cause for sedimentation and rising of the riverbed in the middle and lower reaches. The variation of sediment and water regime should not undermine the scientific value and importance of the CSTR operation of reservoirs on sediment-laden rivers.

(4) With unfavorable sediment and water conditions, the reservoir operation levels stipulated by the "Four-province Meeting" in 1969 shall be adjusted. Before floods enter the reservoir, it is necessary to lower the level to below 292m, and maximize the potential of sediment releasing with all outlets opened, to extend retrogressive scour and make it joined with the scour along the route.

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Examples of Measures being taken for the Preservation of the Sediment Transport Systems in Rivers Downstream of Dams

Akira Kubota
Water Resources Engineering Department, Japan Water Agency, Japan

Kentaro Kido
Water Resources Engineering Department, Japan Water Agency, Japan

1. Introduction

In general, the following points are indicated as the effects of dam construction to sediment transport system:

- Increase in the danger of floods due to the riverbed aggradations in the upstream of the dam.
- Progress of the riverbed degradations in the downstream reach of the dam due to the interruption of sediment transport.
- Prolonged period of turbid water in the reservoir.
- Functional deteriorations of river structures due to the riverbed degradations or riverbed aggradations.
- Coarsened riverbed material in the downstream reach of the dam and the effects of the fixed flow course to river environment.

Because of the above-mentioned reasons, it would be necessary to consider the sediment problems in reservoirs not only from the viewpoint of the maintenance of the reservoirs' storage functions but also from the viewpoint of the sediment management that aims at the preservation and restoration of the river, including the ecosystem. As measures against the sediment problems, the management of the sediment qualities and quantities in the reservoirs and the method that properly supplies sediment material in the downstream reaches of dams are being conducted.[1]

This paper delineates the examples of experimental measures for the preservation of the sediment transport system in the downstream of those dams that are managed by Japan Water Agency and the examples of studies and analyses being conducted for the preservation of future sediment transport system in the downstream of a dam being constructed. This study intends to clarify and examine the themes for the evaluation of concrete measures for the preservation of the sediment transport system in the future by analyzing these examples.

2. Studied Dams

Concerned dams in this paper are those that were constructed and are being managed by Japan Water Agency and those for that sediment nourishment and preservation measures are being taken in the downstream reaches of them. The locations of those dams are shown in Fig. 1.

The conditions of the sediment in the downstream of dams are strongly affected by area factors in general. This tendency is conspicuous in areas along a large geological tectonic line, such as the Itoigawa-Shizuoka tectonic line and the Median Tectonic Line. When looking into the total sediment rate in a dam reservoir (a ratio of the amount of the sediment in a reservoir to the total reservoir capacity; see Fig 2[2]) in each river system, it is especially large in the Tenryu River and Ooi River systems and the second largest is in the Kiso River and Kurobe River systems.

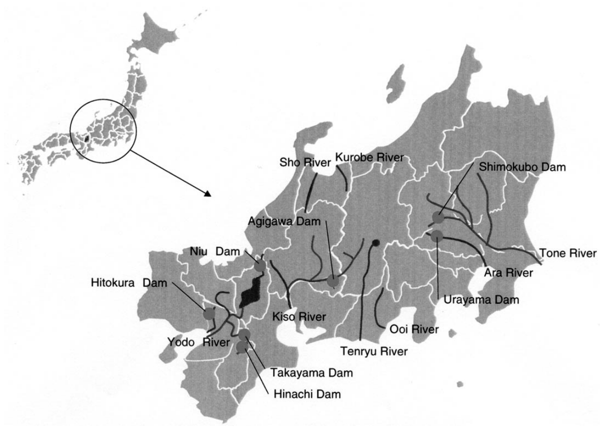

Fig. 1 Locations of Dams and Rivers Examined by this Study

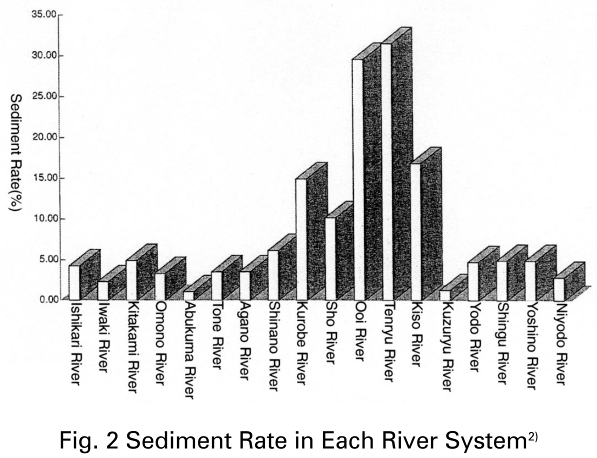

Fig. 2 Sediment Rate in Each River System[2]

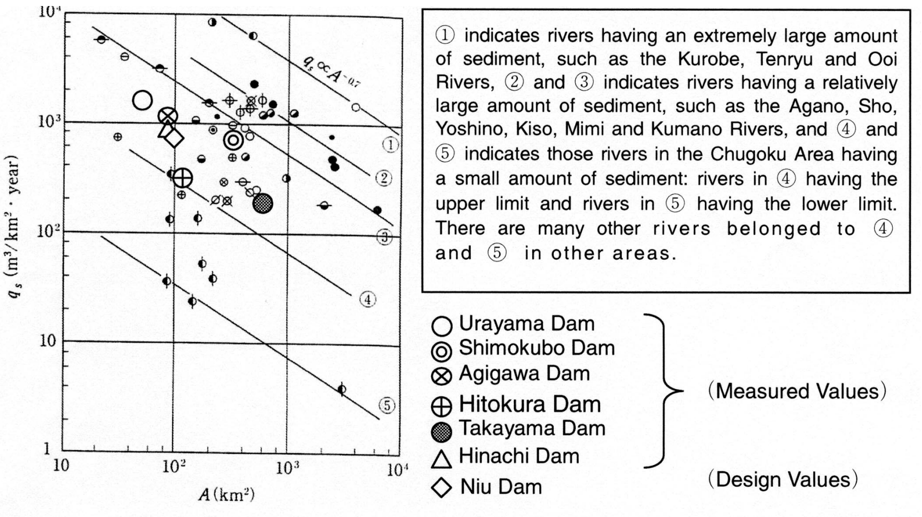

Fig. 3 Relationship between the Size of a Catchment Area and the Amount of the Annual Average Sediment in the Catchment Area[3)]

Fig. 3[3)] shows the relationship between the catchment area of each dam and the amount of annual average sediment produced in the catchment area extracted from the observed sediment data in Japan. Dams discussed in this paper are also included in this figure. In this figure, ① indicates those rivers having the largest amount of sediment, ② and ③ having a large amount of sediment and ④ and ⑤ having a small amount of sediment.

Dam reservoirs having extremely large amount of sediment, such as those in the Tenryu River system and the Kurobe River system, large-scale reservoir sedimentation control measures, such as scour gates and outlets and sediment bypass, are provided.

On the other hand, concerned dams in the paper are built on rivers having a medium amount of sediment transport in Japan. For this reason, the clarification of themes was made not in view of the sedimentation prevention measures in the reservoirs in river systems having an extremely large amount of sediment transport but in view of the sediment transport preservation measures in rivers having a medium amount of sediment.

Japan Water Agency is experimentally applying the following two methods to dams under its management:

① A method to transport and agitate riverbed material by temporarily increasing reservoir discharge amount (hereinafter referred to as flush discharge) without sediment supply in the downstream reaches (Takayama Dam and Hinachi Dam).

② A method to artificially supply riverbed material in the downstream reaches and transport and agitate the sediment by the flush discharge and flood control discharge (Agigawa Dam, Hitokura Dam, Urayama Dam, and Shimokubo Dam).

3. Example of the Effects of Flush Discharge only (Takayama and Hinachi Dams) [4), 5)]

3.1 Outline of these Dams and the Purpose of Flush Discharge

The outline of Takayama Dam and Hinachi Dam is listed in Table 1. The Kizu River, downstream of these dams, is a typical sand river. Area residents and specialists have presented the following opinions regarding the river environment in the downstream of these dams:

- Flow regime in the downstream of these dams became more uniform than before dam construction due to the effects of these dams.
- Algae, that is the main feed of Ayu fish (Japanese sweet fish), is hard to grow fresh.
- Dirt sticks on algae and makes it inappropriate for the feed of Ayu fish.

To respond to the above-mentioned opinions, flush discharge was conducted during the period when the reservoir level was to be lowered to the limit level set for the flood season (hereinafter referred to as a drawdown period).

3.2 Study Method

The study was conducted under the maximum flush discharge of 40 m³/s from Takayama Dam (the flow velocity was in the range of 0.78 to 0.87m/s at the study point) and of 20 m³/s from Hinachi Dam (the flow velocity was in the range of 0.91 to 1.10 m/s at the study point).

The study purpose was to investigate sediment transport and the conditions of algae grown on rock. The study boundary was an approximately 10km section of the river in the downstream of Takayama Dam and an approximately 5km section (up to the confluence of the Shorenji River) in the downstream of Hinachi Dam.

Table 1 Outline of Takayama Dam and Hinachi Dam

Name of Dam	Takayama Dam	Hinachi Dam
Name of River System and River	Yodo River System, Nabari River, a tributary of Kizu River	
Catchment Area of Nabari River	615km^2	
Operation period since dam completion	35 years	6 years
Dam Location (distance from the confluence with Kizu River)	about 0.5km	about 37km
Dam's Catchment Area	615km^2	75.5km^2
River Conditions at Dam Site:		
River slope	about 1/250	about 1/200
Inflow rate(normal water discharge ,2002)	8.5m^3/s	1.43m^3/s
Discharge rate (normal water discharge,2002)	10.16m^3/s	1.58m^3/s

Photo 1 Takayama Dam

Photo 2 Hinachi Dam

3.3 Study Results

The study results of Takayama Dam were as follows:

- Movement of gravel having a maximum diameter of 26mm was confirmed by the sediment transport investigation.
- No great changes could be seen in the chlorophyll-a survey result and no peeling of algae due to flush discharge was confirmed. However, existed predominant species of algae were taken over by others and change in the river environment could be seen.

The study results of Hinachi Dam were as follows:

- Movement of gravel having a maximum diameter of 10mm was confirmed.
- As the same at Takayama Dam, no great changes could be seen in the chlorophyll-a survey result and no peeling of algae due to flush discharge could be confirmed. However, as shown in Fig. 4, the survey result of the ratio of chlorophyll-a to pheophytin-a before and after flush discharge revealed the increase in the chlorophyll-a content. It is considered that this phenomenon means increased algae activities, because chlorophyll-a changes into pheophytin-a after algae die.

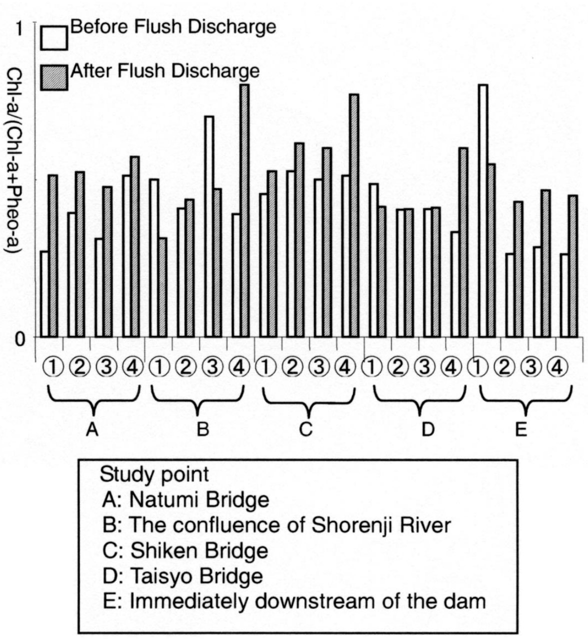

Fig. 4 Change in the Activation of Algae on Rock before and after Flush Discharge (Hinachi Dam)

4. Examples of Sediment Transport Effects (Agigawa Dam, Hitokura Dam, Urayama Dam, and Shimokubo Dam)

4.1 Agigawa Dam[6)]

4.1.1 Outline of Agigawa Dam and the Purpose of Measures Taken

The outline of Agigawa Dam is listed in Table 2. Area residents and specialists have presented the fol-

Table 2 Outline of Agigawa Dam

Name of Dam	Agigawa Dam
Name of River System and River	Kiso River System, Agi River
Catchment Area of Agi River	133km^2
Operation period since dam completion	14 years
Dam Location (distance from the confluence with Kiso River)	8.0km
Dam's Catchment Area	81.8km^2
River Conditions at Dam Site:	
River slope	about 1/80
Inflow rate(normal water discharge ,2002)	1.61m^3/s
Discharge rate (normal water discharge,2002)	1.74m^3/s

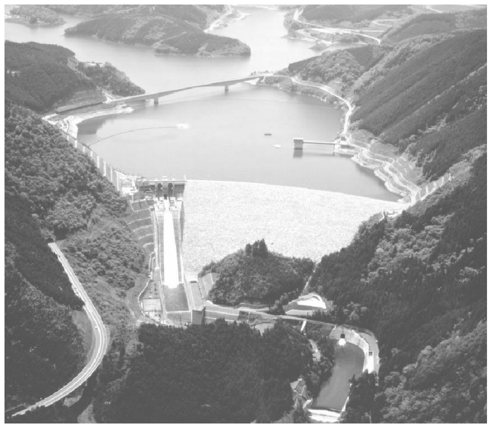

Photo 3 Agigawa Dam

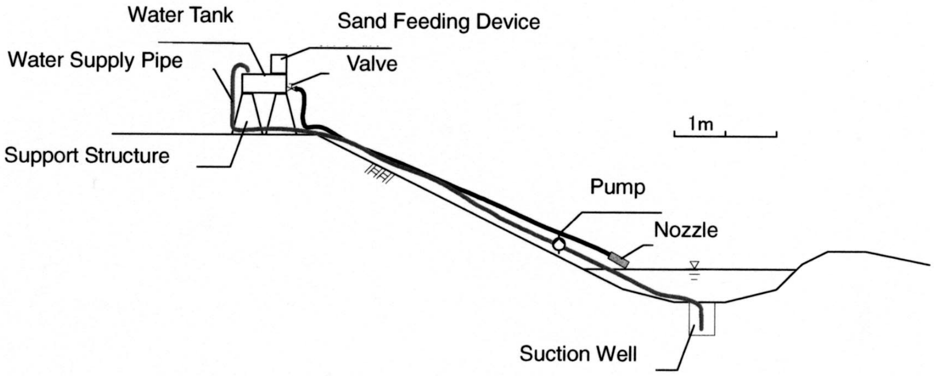

Fig. 5 Equipment Arrangement for Impact and Peeling Test of Algae on Rock

lowing opinions regarding the river environment in the downstream of the dam:

- Riverbed material immediately downstream the dam has become coarser.
- The sand in the river should be flushed so that algae on rock can grow for the feed of Ayu fish.

Based on the above opinions, the sediment material deposited by the check dam (a dam constructed on the upstream of Agigawa Dam Reservoir for water quality preservation purpose) was transported and fed into the downstream reach of the dam in 2004. Before this sediment restoration, the impact and peeling test of algae on rock was conducted as shown in Fig. 5. Gravel mixed water was ejected to algae on rock and the amount of peeled off algae was quantitatively studied.

4.1.2 Impact and Peeling Test of Algae on Rock

Three cases of the test were conducted by changing the amount of gravel in flush water to compare the peel rate of algae on rock with each other. The test results are shown in Fig. 6. The following tendency can be seen from the figure:

① A larger amount of algae is peeled off when gravel-mixed water is ejected onto algae than when water only was ejected (zero gravel concentration).
② When water only is ejected onto algae, the water velocity does not affect to algae peel-off rate.
③ Algae peel-off rate reaches a maximum figure at a certain gravel concentration.

4.2 Hitokura Dam[7)]

4.2.1 Outline of Hitokura Dam and the Purpose of Measures Taken

The outline of Hitokura Dam is listed in Table 3. The Ina River used to be famous as a rare urban river for Ayu fishing. Area residents in the downstream area of the dam have a strong desire to restore the river to the original state so that many Ayu fish will be able to live again as it used to be. For this reason, Japan Water Agency supplies gravel in the downstream of the dam to make spawning grounds, dumps large pieces of rock into the river and repeats flush discharge in order to grow algae on the riverbed. This program is being carried out under the discussions and cooperation with the downstream region's Fishermen's Cooperative Association, NPO groups, community representatives, and related agencies as well as with the participation of area residents.

4.2.2 Study Method

A maximum amount of 20m^3/s flush discharge was made by utilizing a discharge during a reservoir drawdown period. Gravel produced by downstream riverworks was spread uniformly in the river so that the riverbed could be completely submerged when a flush discharge of 20m^3/s was made. The river section studied was an approximately 5km section downstream of the dam (up to the confluence with the Ina River). The main purpose of the study was to investigate the conditions of the algae on the riverbed.

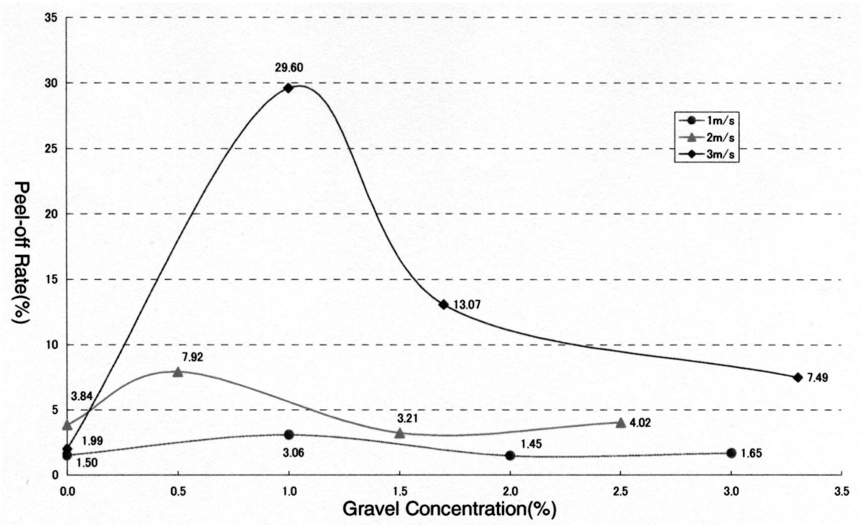

Fig. 6 Peel-off Rate of Chlorophyll-a in the Impact and Peeling Test of Algae on Rock

Table 3 Outline of Hitokura Dam

Name of Dam	Hitokura Dam
Name of River System and River	Yodo River System, Hitokura Ooroji River, a tributary of Ina River
Catchment Area of Hitokura Ooroji River	132.3km^2
Operation period since dam completion	22 years
Dam Location (distance from the confluence with Ina River)	5.0km
Dam's Catchment Area	115.1km^2
River Conditions at Dam Site: River slope Inflow rate(normal water discharge ,2002) Discharge rate (normal water discharge,2002)	 about 1/200 0.95m^3/s 1.43m^3/s

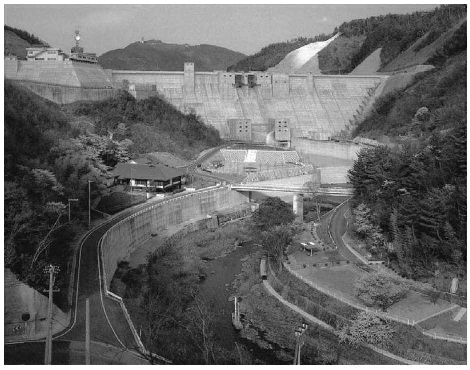

Photo 4 Hitokura Dam

4.2.3 Study Results

Flush discharge was effective to peel off algae from the rock in the downstream of the dam (see Photo 5).

4.3 Urayama Dam[8), 9)]

4.3.1 Outline of Urayama Dam and the Purpose of Measures Taken

The outline of Urayama Dam is listed in Table 4. Ugui fish (dace) live in the Urayama River downstream the dam. The fish is popular for fishing in the campsites along the river. Ugui fish like to spawn on clean riverbed gravel having a 2 to 4mm diameter. For this reason, gravel deposited on the upper part of the reservoir was moved into the downstream reach of the dam in order to prevent the coarsening of riverbed material and the deterioration of fish habitat.

4.3.2 Study Method

A tracer study was conducted to learn if the artificially transported riverbed material was used as a spawning ground. The study was made in an approximately 2.3km river section downstream of the dam (up to the confluence with the Ara River).

4.3.3 Study Results

Spawning of Ugui fish (dace) was confirmed on the artificially relocated gravel-mixed riverbed (see Photo 7). It is considered that the relocating gravel would be effective to maintain the good spawning ground.

4.4 Shimokubo Dam[10)]

4.4.1 Outline of Shimokubo Dam and the Purpose of Measures Taken

Shimokubo Dam is outlined in Table 5. The approximately 1.5km river section downstream of the dam is designated as a national scenic spot and natural monument called as "Sanba Seki Kyo (gorge)." Due to the lowering of the water level and the interruption of sediment supply, the riverbed in this section has been lowered and the landscape has been deteriorated. For this reason, river maintenance discharge begun in 2001 and experimental gravel nourishing has been carried out since 2003. The purpose of this measure is to restore the riverbed and the landscape of "Sanba Seki Kyo" by the cleansing effect. The activities are carried out under discussions with area residents and administrative organs and by taking into consideration the directions and advice of specialists.

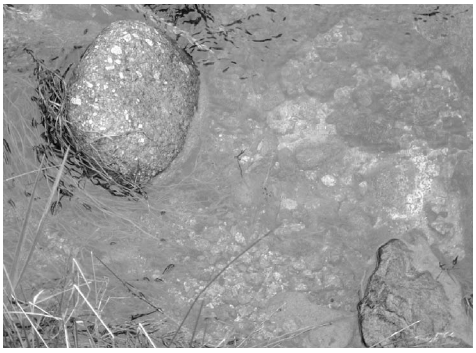

Photo 5 Riverbed Condition Before (left side) and After (right side) a Flush Discharge

Table 4 Outline of Urayama Dam

Name of Dam	Urayama Dam
Name of River System and River	Ara River System, Urayama River
Catchment Area of Urayama River	$59.6km^2$
Operation period since dam completion	6 years
Dam Location (distance from the confluence with Ara River)	about 2.3km
Dam's Catchment Area	$51.6km^2$
River Conditions at Dam Site	
River slope	about 1/80
Inflow rate(normal water discharge ,2002)	$1.01m^3/s$
Discharge rate (normal water discharge,2002)	$0.72m^3/s$

Photo 6 Urayama Dam

Spawning Ugui Fish

Condition of Spawning Ground

Photo 7 Spawning Ground of Ugui Fish made with Artificially Placed Gravel

Table 5 Outline of Shimokubo Dam

Name of Dam	Shimokubo Dam
Name of River System and River	Tone River System, Kanna River
Catchment Area of Kanna River	$407km^2$
Operation period since dam completion	36 years
Dam Location(distance from the confluence with Tone River)	about 30km
Dam's Catchment Area	$322.82km^2$
River conditions at Dam site	
River slope	about 1/130
Inflow rate(normal water discharge ,2002)	$3.02m^3/s$
Discharge rate (normal water discharge,2002)	$3.60m^3/s$

Photo 8 Shimokubo Dam

4.4.2 Study Method

Riverbed material was obtained at the sediment trap dam site. The material was transported and placed in the river downstream of the dam in order to make it to flow when reservoir water was discharged. The study included a periodical landscape survey by photographing. The study area was an approximately 3km section of the river downstream of the dam (Tosen Bridge point).

4.4.3 Study Results

A certain landscape improvement was confirmed, such as the restored shininess of Sanba Seki rock by the cleansing effect (see Photo 9). In addition, some riverbed restoration was confirmed.

4.5 Results of Measures Taken at Dams managed by Japan Water Agency

Points to be evaluated in the above-mentioned measures can be clarified as follows:

① When paying attention to the changes in and activation of predominant algae grown on rock at the sites of Takayama Dam and Hinachi Dam, certain effects were achieved by flush discharge. At the sites of Hitokura Dam and Agigawa Dam, peel-off effects of algae on rock by sediment transport were confirmed.

② At the site of Urayama Dam, attention was paid to the maintenance of the fish spawning ground. Sediment supply to the spawning ground by sediment transport was confirmed.

③ At Shimokubo Dam site, special attention was paid to the area's specific tourism resource (scenic spot of natural monument). Sediment transport could accomplish certain effects on restoring the landscape.

④ At Agigawa Dam site, a basic experiment was conducted to learn the relationship between flow velocity, sediment concentration and the peel-off rate of algae on rock prior to the implementation of the sediment transport and important data were obtained.

⑤ At Shimokubo Dam and Hitokura Dam sites, meetings with area residents and specialists were held to exchange opinions and measures to be taken were decided upon and implemented under discussions with them.

5. Examination of the Preservation of Sediment Transport at a Dam under Construction[11), 12)]

The outline of Niu Dam is listed in Table 6. The Niu Dam is being constructed on the Takatoki River of the Yodo River system. The Takatori River is an important spawning ground of various species of fish, including Ayu fish. Thus, the following studies and analysis are being conducted to learn the sediment transport mechanism in the Takatoki River and obtain basic data necessary to take sediment transport preservation measures after completing the dam construction:

Visual observation of the change in the river topography (river channel, water course, sand bars, riverbed, etc.) and vegetation, preparation of the database of river features prior to dam construction, and periodical photograph taking of the conditions in the river section between the dam construction site and the river month before the dam construction are being conducted. Photograph taking is conducted at the intervals of 1 to 2km from existing bridges with a frequency of more than once a week. It is possible to continuously learn the changes in the river flow and riverbed condition through these activities (see Photo 10).

In addition to the above study, possible effects to the sediment, change in riverbed features and the amount and quality of sediment transport in the Takatoki River to be created by Niu Dam after its completion are quantitatively being analyzed by using the sediment transport prediction model that has been developed for the Takatoki River system.

6. Concluding Summary

For Future Measures to be taken to Preserve Sediment Transport System:

Based on the results of the above-mentioned studies, the following five points are considered as important that measures for the preservation of sediment transport system should be taken in the future for those dams that are built on rivers having a medium

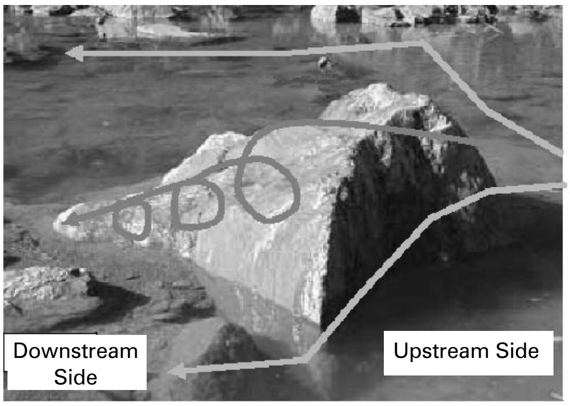

Photo 9 Cleansing Effects and Cleaned Rugged-Rock Surface

Table 6 Outline of Niu Dam

Name of Dam	Niu Dam
Name of River System and River	Yodo River System, Takatoki River, a tributary of Ane River
Catchment Area of Takatoki River	212.0km^2
Dam Location (distance from the river mouth)	about 30km
Dam's Catchment Area	93.1km^2
River Conditions at Dam Site	
River slope	about 1/130
Inflow rate(normal water discharge ,2003)	8.56m^3/s

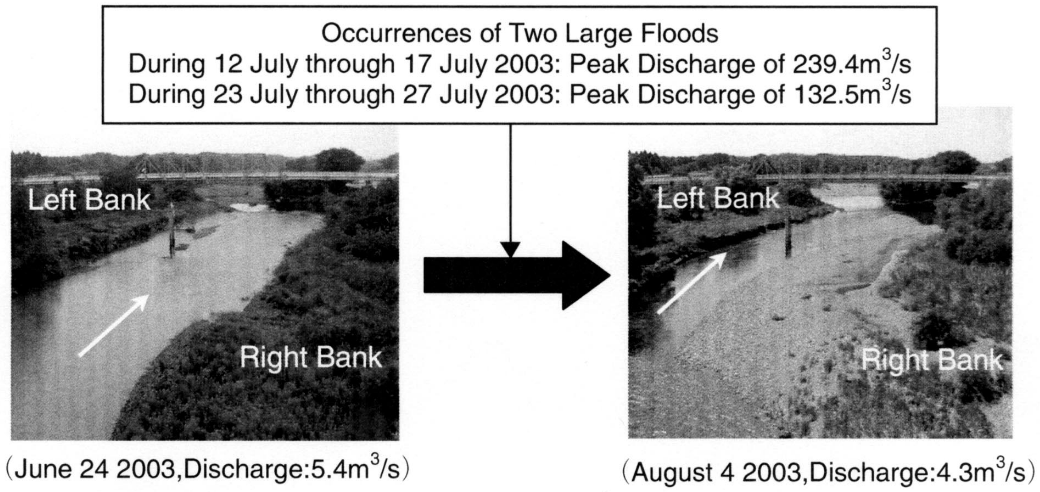

Photo 10 Riverbed Condition Before and After Flood

amount of sediment transport, such as those mentioned above:

(1) Set up of Targets

As targets for sediment transport management, prevention of riverbed degradation, fixed flow course and coarsening of riverbed material, acceleration of the peel-off of algae grown on riverbed rock, and preservation and restoration of landscape are pointed out. Judging from the results of this study, a certain effect to the peel-off of algae was accomplished only by controlling the amount of flush discharge. However, by combining with sediment material supply, flush discharge may result in the improvement of peel-off effect of algae, supply of fine material in river course, and restoration of small sand bars.

(2) Selection of the Method of Sediment Material Supply to Suite the Objective

For the improvement of the river environment in a limited section of the river downstream of a dam, it would be sufficient to conduct sediment material supply in the river immediately downstream of a dam and the drawdown operations of the reservoir. However, when considering a larger scale sediment transport, there would be a limit in the tractive forces to be created by flush discharge from the dam because of the limitation of reservoir discharge equipment and reservoir capacities. Thus, it would be desirable to determine a location to supply sediment material and the amount of it to suite the river course and flow regime.

(3) Careful Monitoring

Discharge from a dam is related to weather condition. Thus, artificial sediment supply and transport should be conducted to suit river conditions that vary from time to time by monitoring the river conditions, preparing a sediment supply plan and modifying the plan from time to time to suit the conditions. To supply a sufficient amount of sediment material, it is important to conduct a continuous monitoring of downstream river environment in detail and prepare the database.

(4) Cooperation and Understanding from Various Aspects

It is important to maintain close communications and cooperation with concerned administrative organs and river water users, fishermen and tourism industries and area residents.

(5) Development of more economical and effective method

As mentioned above, it is considered that the further economical and effective method for sediment material management may be developed by accumulating scientific knowledge related to the effects of sediment transport given to river environment.

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Observation and Analysis on Sedimentation of Xiaolangdi Reservoir

Qu Zhangbin　Xiao Qiang
Yellow River Water & Hydropower Development Cor., MWR, China

Preface

Xiaolangdi Multipurpose Dam Project is located at the mouth of the last gorge in the middle reach of the Yellow River, 130.8 km upstream of Sanmenxia Reservoir, 4 km and 128.2 km downstream of the relocated Xiaolangdi Hydrographic Station and Huayuankou Hydrographic Station respectively. About 694,155 km^2 catchment area is under the control of Xiaolangdi dam, 92.3% of the total. The reservoir compiles a long-term average runoff of 28.146 billion m^3, 87% of the total, and controls the total sand of almost 100% transported to the Yellow River. The project is key in harnessing the problem of sedimentation at the middle/lower reaches of the Yellow River. Since put into operation, e.g. sand retaining and flow/sand regulation, Xiaolangdi project plays a significant role in d/s flood control, slows down the rising of d/s riverbed, hence becoming one of the most important measures to keep the healthy life of the Yellow River.

I. Characteristics of the Reservoir

Xiaolangdi reservoir level is designed to reach El.275m in maximum, with a total storage capacity of 12.65 billion m^3 (41.1% comes from branch streams). The surface water is 272.3 km^2 in area in maximum (40.3%, amounting to 110.4 km^2, is contributed by branch streams). About 159.7 km (branch valley is excluded) belongs to branch stream-accumulated sections, 1.24 times of the main stream. The reservoir, while being at El.275m, is 905 km long in perimeter.

The canyon-like riverbed of the main stream of the Yellow River is narrow on the u/s and wide on the d/s. It is 65 km long and 200～400m wide from Sanmenxia Station to the outlet of Banjian River, while it is 65 km long and 500～1000m wide from Banjian River to Xiaolangdi dam. The valley of Balihutong, 26 upstream of Xiaolangdi dam, is 4 km long, 200～300m wide in minimum at the bottom and 200～300m wide on the top. The riverbed covered by the sandstone is 1% in gradient.

II. Operational Style of the Reservoir

Operation of the Xiaolangdi multipurpose dam project is composed of 3 stages, i.e. early stage of sand retaining, late stage of sand retaining and normal operational stage.

Early stage of sand retaining: the sedimentation weighs less than 2.1～2.2 billion m^3 (lower than El.210m);

Late stage of sand retaining: there are definite division of high beaches from low slots in the reservoir (El.254m closer to the dam), sedimentation weighs 7.55 billion m^3;

Normal operational stage: keep a reservoir volume of 4.05 billion m^3 above El.254m, perform a long term flow/sand regulation by using the water of 1.05 billion m^3 stored in the low slot below El.254m.

III. Characteristics of sedimentation

The closest u/s station to Xiaolangdi reservoir is Sanmenxia hydrographic station. From 1952 to 2000, the annual runoff of Sanmenxia reservoir is 38.2 billion m^3 in average, transporting the sand of 1.23 billion t (2.256 billion t in maximum with sand content of 911kg/m^3 in 1977). The sand input to the reservoir from Sanmenxia dam is mainly sourced from valley flushing by heavy rain in the flood season (0.4%, amounting to 47 million t, is from other sources) .

IV. Profile of sedimentation

Guided by the principle of "dense profiling closer to the dam and big branch streams, sparse far away from the dam and small branch streams", there are in total 174 profiles laid out in Xialangdi reservoir, vertical to the contour line at El.200～275m. There are altogether 56 profiles laid out for the main stream, of which, 16 Nos for the upper reservoir with 3.42 km in space, while 40 for the lower one with 1.79 km in space (the line where it is 71.67 km away from the dam is regarded as the boundary to separate the upper reservoir and the lower one). 65 profiles are arranged for 21 branch streams from the left bank, while 53 ones for 19 from the right bank.

V. Measurement on sedimentation

At present, we adopt terrain method for measurement on sedimentation, changing the traditional profiling way. The main instrument used for the measurement is Geoswath Strip Echo Sounder made in U.K, and the boat is accurately positioned by GPS, which is controlled by Hypack navigation software. Measuring in the traditional profiling way, we performed 4 measurements which failed to meet the anticipated target, for the selected profiles could not represent the underwater terrain and the changed conditions of sedimentation especially in the tundish zone, 4.2 km away from the dam, the work was time consuming and money was lost. However, after the measurement method is introduced, the situation is greatly improved.

VI. Observation and analysis on sedimentation

By measuring the underwater terrain in the backwater zone, learn and grasp the quantity, location, shape, composition of the sedimentation, and make clear how the sedimentation changes in front of the dam.

(I) Quantity of the water in and out

The reservoir volume in 2004 was less than that in 2003, 8.263 billion m^3 in maximum at El.261.99m (April 1, 2004) and 1.86 billion m^3 at El.218.63m in minimum (Aug.30, 2004). The flow into the reservoir in 2004 is 16.64 billion m^3 (39.6% is compiled from July to October), 1.387 billion m^3 per week, 2840 m^3/s in maximum on July 7, 2004. The flow out of the reservoir in 2004 is 20.66 billion m^3 (33.4% is discharged from July to October), 1.722 billion m^3 per week, 2680m^3/s in maximum on July 9, 2004.

(II) Sand in/out

The clay/sand into the reservoir mainly happened during flow/sand regulation event from June to August 2004, 524 kg/m^3 in maximum on Aug.22, 2004. During the 3rd flow/sand regulation from June16 to July13, 2004, 44 million t sand was brought and 5.5 million t was discharged out of the Xiaolangdi reservoir; while for the 4th one, 170 million t in and 150 million t out.

(III) Density current and its vertical distribution

On Jul. 9 and Aug.1, 2004, the density current repeatedly moved to the front of the dam, forms a muddy reservoir till the end of November 2004. By sampling from a fixed position in front of the dam, it is found that there was no major difference of sand content at various elevations, however after the end of August 2004, the muddy layer dropped, deeper and denser, shaped like an inclined fold line. From the end of August to the end of September, the muddy layer dropped by 2.1m~3.1m on a weekly basis, turning thinner and thinner in front of the dam; from the end of September to the end of October, the muddy layer dropped by 0.5m per week, shaped like a bench, and deposits 300 kg/m^3~400 kg/m^3 at the bottom of the reservoir.

(IV) Distribution of the muddy sand

Even though the reservoir is deeper and denser, there is no major change in the size of sand particles, 0.004mm~0.005mm on Aug.19 and Sep.7, 2004 when the reservoir was muddy, 0.005mm~0.006mm in general; 0.004mm~0.005mm on Sep.28 and Oct.26, 2004, 0.006mm in general. This shows that while the density current moves to the front of the dam, no matter the muddy layer is rising or dropping, the composition of the sand is the same, namely, fine silt sand. If we timely discharge the muddy reservoir, this kind of sand can be flushed directly to the ocean, with nothing to do with the d/s riverbed.

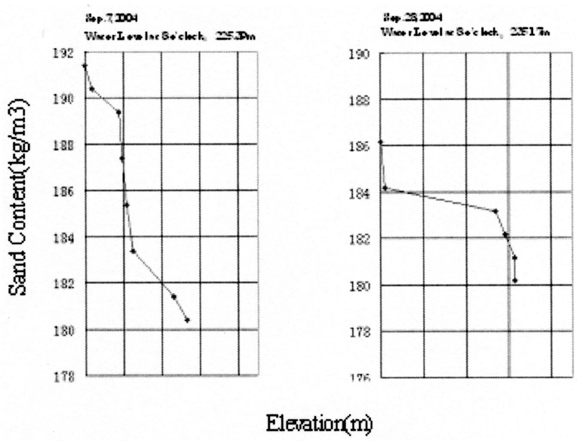


Fig. 1. Layout of the Sand Content in the Muddy Layer in front of the Dam, 2004

(V) Longitudinal sedimentation on the main stream

The post flood reservoir volume of 11.08 billion m^3 in 2004 is 152 million m^3 less than that before the flood season, with 126 million m^3 deposited on the main stream of the Yellow River, 83% of the total in this period.

(i) Quantity and longitudinal distribution of the sedimentation

Compared with the sedimentation level before the flood season, it was increased by 5.5m (1.45 km away from the dam). The sand was over 10m thick between 42.51 km and 51.87 km from the dam, even 17.3m in maximum, 46.05 km away from the dam. Flushing the riverbed happened 62.11 km from the dam, while the 10m deep into the riverbed happened 71.67 km from the dam, the far upstream, the better flushing. By the end of November 2004, the vertical gradient of the riverbed is 0.69%, which is much less than 1% in 1999 when the reservoir was not impounded.

(ii) Analysis on shape of sedimentation

The vertical sedimentation is subject to the characteristics of sand/water, shape of surface water, changes in operational level in front of the dam and the terrain of the reservoir among other conditions, of which, the characteristics of sand/water and the changes in operational level in front of the dam are the most important factors. Because of the flow/sand regulation, the reservoir level is relatively high, when Sanmenxia reservoir releases big flow continuously in a certain period, it is easy to form a delta of sedimentation in the reservoir, which was the situation of xiaolangdi reservoir in 2004.

Two tests, made in 2004, suggested that the longitudinal sedimentation is shaped like a delta. The top of the delta is at El.229.5m, 60.15 km away from the dam, which moved downward by 14.38 km in comparison with its position before flood season (74.53 km away from the dam). The front slope of the delta, 10.1 km

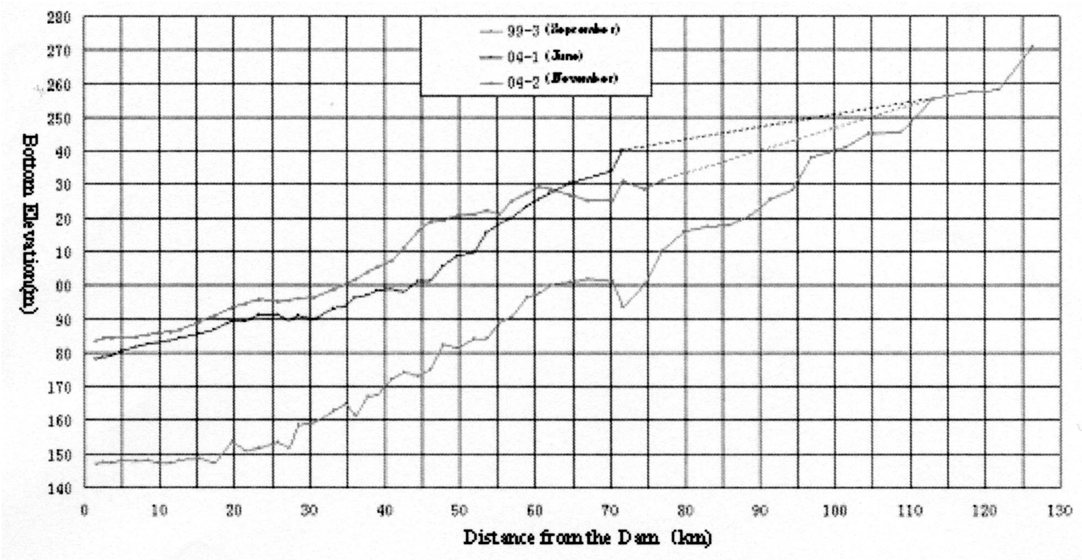


Fig. 2. Elevation Changes of the Yellow River Slot within the Xiaolangdi reservoir

long, was located from 70.25 km to 60.15 km with the top elevation of 225.4m; the rear slope, 14.1 km long, was from 60.15 km to 46.05km with bottom elevation of 219.0m; the tail section is from 70.25 km upward. The front and rear slopes of the delta are –0.41% and 7.4% in gradient.

(iii) Quantity of the delta

The quantity of the delta is 456 million m^3, of which, the front slope sedimentation of 327 million m^3 (71.7% of the total) and rear one of 129 million m^3 (28.3% of the total).

(iv) Horizontal sedimentation on the main stream

Between 1.45 km and 51.87 km from the dam, the profile shows horizontal sedimentation, which is 51.87 km long. The beach and slot separates clearly 51.87 km away from the dam, with height differences of 6m, 10m and 11.1m in the areas of 56.85 km, 62.11 km and 71.67 km from the dam respectively. Since Xiaoalngdi reservoir has a long impoundment period to store the clear water during non-flood season, the backwater is far upward, the reservoir level and amplitude change slowly, and also the reservoir turns muddy in the flood season, the uniform horizontal sedimentation is easy to happen. On this condition, the top of the delta sedimentation is easy to move, which facilitates the forming of the density current.

(VI) Sedimentation on branch streams

There is little change in the sedimentation volume on the branch streams, except for a slight rising of riverbed adjacent to the estuary. The longitudinal sedimentation is like a cone, slow in gradient. Beach and slot separate at the estuary of the u/s river, say, Yunxi River (56.34 km from the dam), Banjian River (63.87 km from the dam), Haoqing River (58.84 km from the dam); horizontal sedimentation was found in the d/s Dongyang River (30.84 km from the dam). However, the estuary was not plugged.

(VII) Composition of sedimentation

In comparison with the sand granule before the flood season of 2004, the middle-sized sand turned fine, 19.7 km in front of the dam, which is less than 0.01mm; no orderly phenomenon was found between 19.7 km and 58.9 km; the sand turned coarse from 62.1 km upward, which is more than 0.02mm, showing that the 4th and 5th flow/sand regulation were satisfactory. The sand of the reservoir tail moved downward, most of which is silt sand. In light of above, by making use of Xiaolangdi reservoir's coarse retaining and fine sand discharging, coarse sand can be deposited as close as possible to the dam, which will facilitate the forming of the sedimentation bedclothes, reduce seepage of dam foundation and discharge the fine sand out of the reservoir.

VII. Closing remarks

(I) Xiaolangdi reservoir has a sedimentation volume of 1.48 billion m^3 by the end of 2004, when we called sand retaining early stage.

(II) The top position of the delta sedimentation is subject to the level closer to the dam, scope changes and operational style of the reservoir, changeable in direction during wet and dry season, which moved downward after flow/sand regulation performed this June.

(III) The level changes in sedimentation in front of the dam is subject to the operation of discharging tunnels. It is suggested to operate the relatively low discharging tunnels when the density current ran up to the dam and the reservoir turned muddy.

(IV) It is a most important solution to make use of the density current for sedimentation reduction and natural life extension of xiaolangdi reservoir. In order to keep the long term operational volume of the reservoir, it is suggested to reduce the water level to flush the sedimentation, provided that the condition is suitable.

Sediment Management Measures at the Miwa Dam

Yasushi Enomura
Director, Mibugawa River Comprehensive Development Office, Ministry of Land, Infrastructure and Transport, Japan

1. Outline of the Miwa Dam Redevelopment Project

Originating in Lake Suwa, the Tenryu River flows in the central portion of Japan's main island, conjoining branch streams from the southern and central Japan Alps, passes through several gorges and alluvial plains before finally pouring into the Pacific Ocean. With a total basin area of 5,090 km^2 and a total river length of 213 km, it is one of the most precipitous rivers in Japan. Characterized by rainfall in steep mountain areas, the river runs down rapidly and carries a large quantity of sediment in its upstream course. For this reason, it has been called the "Wild Tenryu," frequently causing damages to food production, notably in 1961, 1982 and 1983. Figure 1 shows a diagram of the river basin.

The Mibugawa River Comprehensive Development Project focuses on the redevelopment of the existing Miwa Dam located on the Mibu River, the Tenryu River system's largest tributary.

In the Miwa Dam redevelopment project, sediment excavation in the reservoir and permanent sediment management measures were formulated to maintain as well as reinforce the flood control and water supply functions of the Miwa Dam, a multipurpose concrete gravity dam with a height of 69.1 m, a crest length of 367.5 m and a gross storage capacity of approximately 30 million m^3. The dam was completed in 1959 by the then Ministry of Construction (now Ministry of Land, Infrastructure and Transport) for the purpose of flood control, irrigation and power generation.

During the last 46 years, the dam has contributed to the region's safety and industrial development by preventing flooding, providing water for nearly 2,500 ha of downstream paddy fields and supplying electricity to meet the demand of some 43,000 households.

Yet, the flood-induced sediment inflow has brought about such an accumulation of sedimentation that effective storage capacity is barely secured. If large-scale flooding occurred, it is possible that there would be damage to the flood control and water supply functions of the Miwa Dam. Thus, excavation in the reservoir as well as measures to prevent further sedimentation (permanent sediment management) are vitally important. Photo 1 illustrates the sedimentation, and Figure 2 is a chart of yearly sediment change.

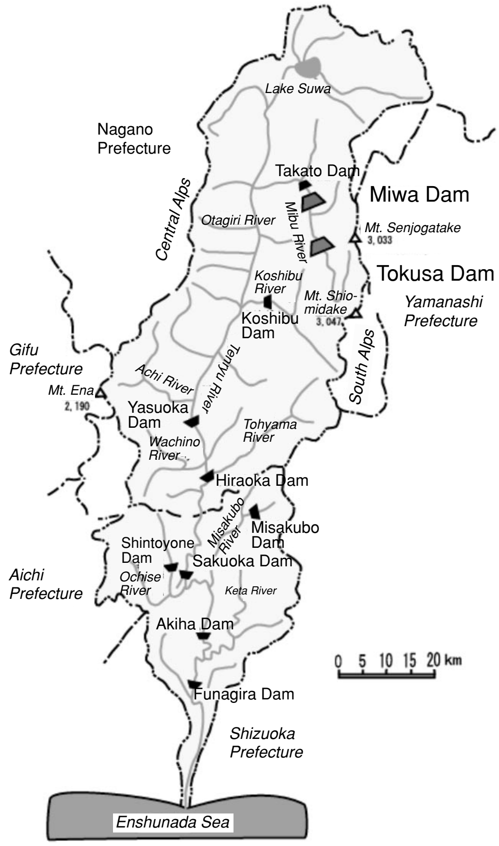


Figure-1 Tenryu drainage basin

An study on the execution plan was launched in 1987, and construction work started in 1989 under the Mibugawa River Comprehensive Development Project. Regarding permanent sediment management measures, a check dam was installed in 1993, and a flood bypass tunnel and a diversion weir were completed in 2005. Work on sediment excavation was begun in 1999 and nearly two million m^3 of sediment has been excavated so far to restore the reservoir capacity. The excavated soil sediment is used effectively in public works as banking materials for relocated roads and farmland consolidation.

Photo-1 Miwa Dam during dewatering (the photo taken in 1989)

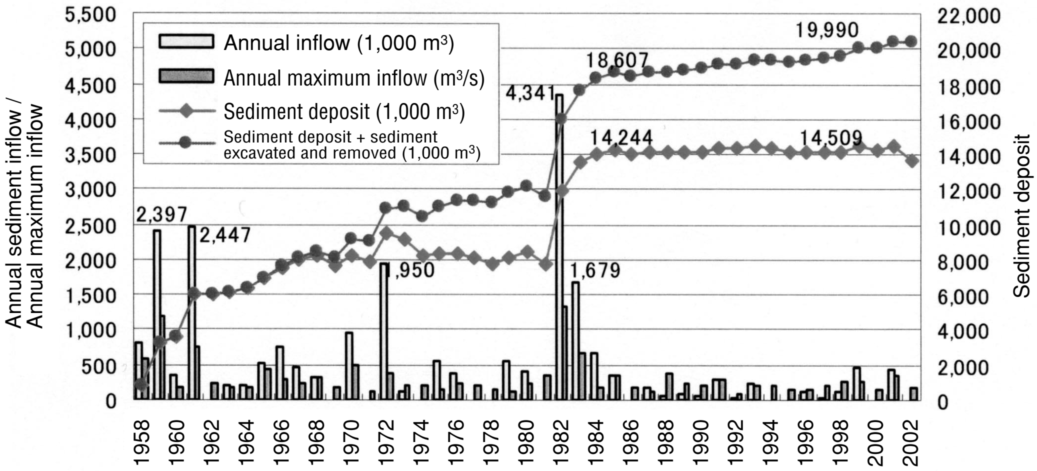


Figure-2 Yearly sediment change

2. Overview of the permanent sediment control facilities

The permanent sediment management measures aim at checking the reduction in reservoir capacity that has occurred as a result of sediment flowing into the Miwa Dam from flooding by trapping it upstream of the reservoir or bypassing it down below the reservoir.

Because the inflow sediment in the Miwa Dam characteristically includes many fine-grained fractions (wash load), a plan was made for the check dam in the upstream reaches above the reservoir to trap all the coarse-grained fractions (bed load and suspended load) so that only the wash load would be bypassed down below the dam. The bed load and suspended load trapped in the upstream reaches was to be collected by privately owned gravel gathering agencies for utilization as construction materials. Figure 3 is a chart of the annual volume of sediment removed (annual average expectation value).

The permanent sediment management facilities consist of the installations described below. Figure 4 shows a map of the permanent sediment control facilities.

① Check dam

From the total inflow sediment, this facility traps bed load and suspended load. Its sediment storage capacity is 200 thousand m³. The sediment is excavated and removed by private gravel agencies.

② Diversion weir

The diversion weir combines the functions of training and trapping the sediment. Wash load, which flows down through the check dam, is guided together with floodwater from a bypass channel into a bypass tunnel. When large-scale flooding occurs, the facility

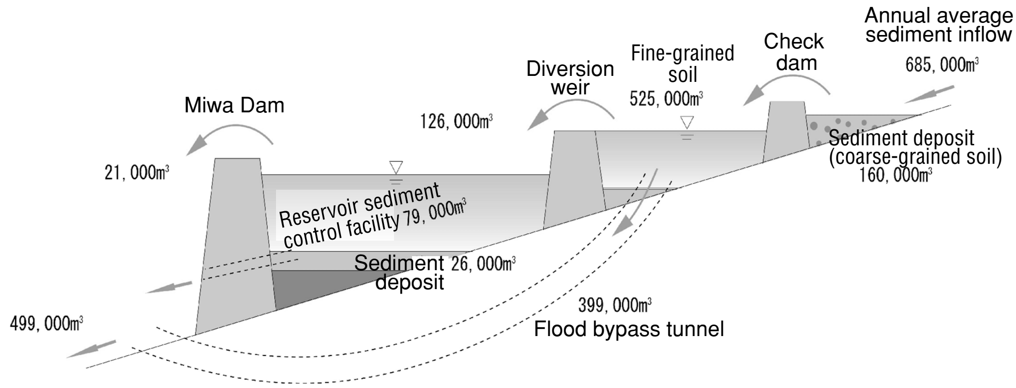


Figure-3 Predicted annual sediment discharge

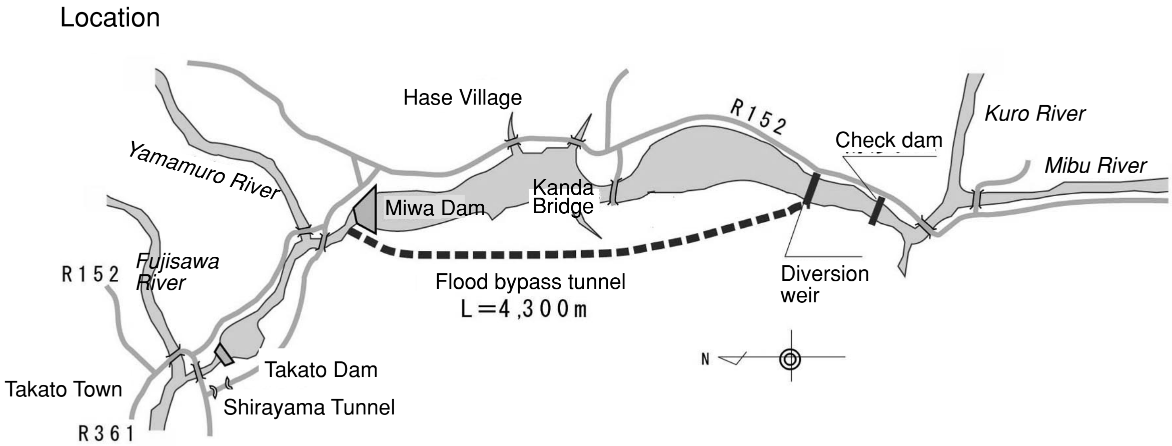


Figure-4 Location map of the permanent sediment control facilities

allows a training dike and a trap weir to catch coarse-grained fractions that overflow from the check dam to prevent them from flowing into the flood bypass tunnel. The storage volume of the diversion weir is 520 thousand m³.

③ Flood bypass tunnel

The facility bypasses wash load together with floodwater to the area downstream of the dam. Its maximum flow rate is about 300 m³/s.

④ Reservoir sediment control facility

A facility is currently being designed that will discharge wash load, which flows into the reservoir together with floodwater, into the reaches downstream of the dam, as a flood control measure.

3. Outline of the construction works

1) Diversion weir

The diversion weir installed in the upstream reaches of the Miwa Dam reservoir is a concrete gravity structure 20.5 m in height and 244.5 m in length. The construction works were begun in October 2001 and nearly 53,800 m³ of concrete was placed into the body of the structure in February 2005. By the end of fiscal 2004, the bypass channel on the left side connecting the bypass tunnel intake to the diversion weir, the main and secondary gates and the diversion weir access bridge were completed. The ancillary structures, including a power supply device, a gate operating apparatus and a monitoring facility were finished by May 2005.

Because it was necessary to carry out work on the diversion weir inside the reservoir, the execution period was restricted to six months during the non-flood season between October and March. Construction efficiency is also affected by the fluctuation in the reservoir level. For this reason, the reservoir level was lowered during the non-flood period to secure the construction term while the reduced energy output was compensated for by the power plant on the Miwa Dam. Furthermore, to facilitate concrete placement under circumstances where the average temperature was below zero, an appropriate winter concreting measure (use of warm water for mixing and jet heater after placement) was adopted to ensure high quality. Photos 2 and 3 show the diversion weir under construction and after completion.

2) Flood bypass tunnel

The flood bypass tunnel was excavated on the left mountain side of the Miwa Dam reservoir. It extends

Photo-2 Construction of the diversion weir

Photo-3 Current diversion weir

Photo-4 Construction of the tunnel entrance

Photo-5 Current tunnel entrance

nearly 4,300 m and the finished sectional area is approximately 47 m^2.

The section to be excavated basically consists of Mesozoic granodiorite formed in an area known as the Ryoke belt, and was constructed using the NATM method. The work was launched in February 2001 and all the lines were totally penetrated by March 2004.

Because the width of the finished sectional area of the tunnel is 7.8 m, too narrow for heavy dump trucks to pass, a conveyer belt was used to remove the excavated rock and mud. The use of this method provided an even greater enhancement of safety management, as it made the construction process shorter and improved the working environment as the conveyer belt emitted far less exhaust gas than vehicle transportation would have, had it been used.

The ventilation system for the tunnel construction consists of a central air supply and an end dust-collecting system placed near the facing site to filter out the dust particles that the facing work would produce.

There was some concern about incidental water leaking in during the facing work, as the tunnel was to be excavated alongside the Miwa Dam reservoir, below the water level of the reservoir, and construction was to be carried out in the proximity of the median tectonic line. Therefore, a seismic reflection method (TSP method) using elastic waves and a hydraulic rock drill search boring method was applied to anticipate any fracture zones and geologically transitional areas. As a result, there was no significant water inflow. Thus, the excavation was carried out smoothly, according to the construction schedule.

Later, the tunnel was lined and the concrete floor slabs were placed. Work on the tunnel outfall was begun in June 2004 and was completed by the end of May, 2005. Photographs 4 and 5 illustrate the flood bypass tunnel under construction and after completion.

4. Test operation/monitoring plan

In the construction of the permanent sediment management facilities in the Miwa Dam, the flood bypass tunnel, the diversion weir and the check dam have been completed, and test operations have been underway since June 2005.

The current Miwa Dam operation rules have been implemented so that fixed release-rate operations are applied when inflow the exceeds 300 m^3/s and fixed release-volume operations are carried out when the outflow rate exceeds 500 m^3/s. As a result, water,

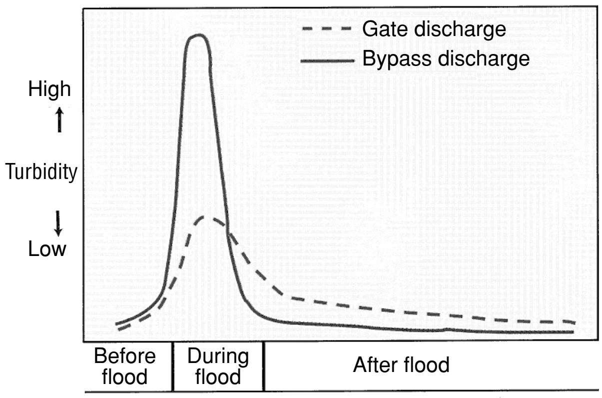


Figure-5 Conceptual diagram of change of turbid water

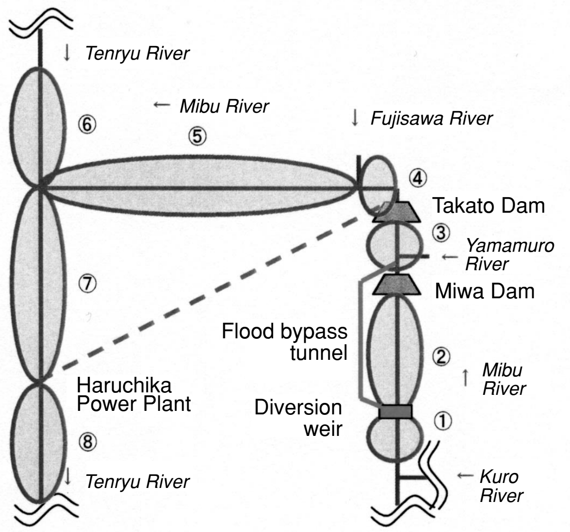


Figure-6 Area surveyed

which had been previously discharged by the main gate of the Miwa Dam, is now partially released through the bypass gate without flowing into the reservoir.

Specifically, bypass operations are performed when the Miwa Dam is required to conduct a gate discharge during flooding, which is expected to occur four or five times a year at a peak inflow rate of $100m^3/s$ or over. Floodwater is discharged from the bypass gate at a rate of up to $300m^3/s$, and the bypass operation is terminated after the inflow rate reaches its peak and declines below $100m^3/s$.

Regarding operational requirements including the inflow rate to start and finish bypassing, the monitoring research results obtained from test operations are analyzed and fed back in order to draw up the most effective operation plan.

In order to examine the scouring effects and evaluate the impact on the downstream environment, the following items are investigated in the monitoring survey.

① Evaluation of the sediment balanced plan

The measured values and assumed values regarding the following four items were compared to evaluate the validity of the plan.

- the amount of wash load flowing into the Miwa Dam
- bed load and suspended load trapped by the diversion weir and the check dam
- sedimentation in the Miwa Dam reservoir
- sedimentation in the Takato Dam reservoir located immediately downstream from the Miwa Dam

② Evaluation of the facility structure

The characteristics of separated flow at the diversion weir, the sedimentation in the flood bypass tunnel and equipment abrasion are examined.

③ Evaluation of effluent turbidity

In comparison with conventional gate discharge through a reservoir, the water washed down by the bypass discharge is of higher turbidity (see Figure 5). Therefore, it is expected that the turbidity of water in the reservoir will be reduced so that the time taken to discharge turbid water from the dam after flooding should be shortened. These effects are being evaluated.

④ Evaluation of impact on local organisms

The impact of changing turbidity in the dam discharge on downstream organisms is being evaluated.

The river section between the upstream area of the Miwa Dam on the Mibugawa River and the confluence of discharge from the Haruchika Power Plant with the Tenryu River is divided into eight areas (see Figure 6), where the following items are being observed: the flow rate, water quality (temperature, DO, SS and SS grain size), the sediment volume in the area downstream of the dam, the riverbed materials, organisms (fish species, zoobenthos and attached algae) and the dry riverbed (water route, vegetation zone and sedimentation). In addition to the regular times of inspection, observations are carried out for certain durations during and after the bypass discharge operation to grasp any changes in the sediment regime and the environment induced by the bypass operation.

A period of approximately five years is allotted for monitoring after the commencement of test operations, during which time the items surveyed and inspection frequency may be reexamined appropriately, depending on the situation. A preliminary survey was begun in 2004, one year prior to the test operation, to compare results with those obtained while operations were in progress. These data will be scientifically and objectively evaluated by the management follow-up committee for dams in the Chubu region organized by the Chubu Regional Development Bureau.

5. Conclusion

The major facilities for the permanent sediment management measures of the Miwa Dam redevelopment project, such as the diversion weir and the flood bypass tunnel, were completed and test operations begun. This is the first time that a bypass tunnel has been used to control sediment in a multipurpose dam in Japan and its achievement has attracted a great deal of attention. In addition to restoring and maintaining the functions of the dam, it is expected that the sediment control measures will cause sediment to flow down as naturally as possible from the perspective of the comprehensive sediment management of the basin.

It is hoped that we will be able to contribute to the dam and river basin management not only for this dam, but also for other dams and river systems, by closely studying the results and impact of the permanent sediment control facilities.

Reservoir Area Development Measures and Some Examples in Japan

Katsuhito Miyake
Director, First Investigation Div., Water Resources Environment Technology Center, Japan

Tadashi Kitamura
Director, Management Planning Div., Japan Water Agency, Japan

Hideya Ohta
Director, Coordination Division, Land Department, Japan Water Agency, Japan

1. Necessity and a purpose of measures for a Reservoir area development

Inhabitants of lower-stream areas benefit from dam projects in various ways such as flood-defense, water supply (domestic and industrial) as well as maintenance of minimum volume of water flow necessary for river transportation, fishery, landscape, prevention of salt and closure of estuaries, protection of river management facilities and so on.

On the other hand, it is inevitable that people living on the site of dam structure and reservoir should be displaced. Thus dam projects bring about disintegration of community life and loss of identity based on territorial and ancestral background. Displaced inhabitants need to re-start their livings against these negative backgrounds. And, in addition to physical changes caused by construction works over a long period, displacement of inhabitants makes it impossible to sustain the life-style and economic activities based on community. Decline in local tax revenue (income tax and corporate tax) also creates serious difficulties for provision of basic public service, such as care for the elderly and education.

In this way, dam projects bring about fundamental changes in natural, economic and social conditions of water-source areas. Their influences persist long after the works are completed and operations of the dams are started.

This disparity makes it all the more difficult for displaced inhabitants to accept their sufferings as well as for water-source areas to willingly cooperate with the projects.

These circumstances made it necessary to improve the compensation system and resettlement assistance measures. As institutional frameworks for compensation, the Guideline for Compensation for Losses Caused by the Acquisition of Land for Public Purposes and the Guideline for Compensation for Losses Caused by Public Works Projects were instituted in 1962 and 1967, respectively. As resettlement assistance measures, the Law Concerning Special Measures for Reservoir Area Development (hereafter referred to as the “Special Measures Law”) and the so-called “Three Laws for Electric Power Development” were enacted in 1973 and 1974, respectively.

This articles show the outline of reservoir area development measures and some examples in Japan.

2. Process of Japanese measures for a Reservoir area development

2.1. Compensation to property to land acquisition

2.1.1 Uniform Compensation Standard

Delivery of compensation for purchase of properties is implemented under <Uniform Compensation Standard for Purchase of Properties for Public Purposes (Cabinet Decision, 1962/6/19)>. Compensation under this standard is considered to correspond to the <just compensation> under article 29 of the Japanese Constitution

Article 29 of the Japanese Constitution

The right to own or hold property is inviolable. Law, in conformity with public welfare shall define property rights. Private property may be taken for public use upon just compensation therefor.

2.1.2 Background of the Establishment of Uniform Standard for Compensation

In postwar Japan, rapid economic growth was accompanied by expansion of public investment for infrastructure development. Corresponding rise in land price enhanced expectations for compensations on the part of landowners, which made land purchase for public works more difficult. Given time limits for purchase negotiations, settlements of compensation were often arbitrary. Spread of information about arbitrary settlements made landowners in other projects more demanding to create a vicious circle of project undertakers forced to make further arbitrary concessions. Thus, it came to be globally recognized that difficulty in purchase of property is the main stumbling block of infrastructure development, leading to the establishment of a just and uniform compensation standard applicable to all public works projects.

2.1.3 Substances of Uniform Standard for Compensation

a) Compensation corresponding to the value of properties taken for projects

Taking of properties includes extinction of fishery rights and mining rights. The basic principle is that compensation should be equal to the objective value of the properties taken, that is, their normal market value. This means that neither spiritual values the owners attach to inherited properties nor special uses of the properties should not be taken into account.

The principle of <normal market value> is expected to be applied not only to negotiated purchases but also to expropriation of properties under Land Expropriation Law (Law No.219 of 1951).

For determination of normal market value, <Real Estate Appraisal Standards> was officially established in 1964.

b) Incidental indemnities

Incidental indemnities cover costs of removal of buildings and timbers, those resulting from abolition or suspension of operations made necessary by the taking of properties as well as decline of value incurred on the reminder when only part of a piece of land is taken for a project.

2.2 Public Compensation

Execution of public works projects often causes abolition or suspension of public facilities for which undertakers of the projects should be held responsible. For example, roads running on the site of dam structure or its reservoir should be abolished and dislocated alongside the submergence area at the expense of the undertaker of the dam project.

Normal market value principle is not applied for this type of compensation. Public facilities are in service for the interests of their users and their utilities cannot always be measured in terms of market values of properties constituting the public facilities.

Thus, <Uniform Public Compensation Standard for Execution of Public Works (Cabinet Decision, 1967/2/21)> was established to ensure that functions of public facilities that should be abolished or suspended are properly restored so long as it is technically and economically viable.

2.3 A measure for resettlement and vitalization of reservoir area

2.3.1 Special Measures Law for Water-Source Areas (Law No.118 of 1973)

By improving living environment and industrial infrastructures in water-source areas that undergo enormous changes of their fundamentals as well as preventing water pollution in reservoirs of dams, the purpose of this law is to ensure stability of living and improvement of welfare thereby promoting construction of dams and water level controlling facilities to contribute to water resources development and preservation of national territory.

So this law integrates measures to address the issue of vitalization of water-source areas and restoration of living which compensation (including public compensation) by the undertakers of dam projects alone cannot satisfy. (Fig. 1)

2.3.1.2 Designation of Dams and Decision of Water-Source Area Development Plan

As of April 2005, there are 94 dams and 1 water level controlling facilities for lakes designated by cabinet order under the law. Among them, 12 dams and the 1 water level controlling facility are undertaken by the JWA (Japan Water Agency). 26 dams (including 2 JWA dams) and the 1 water level controlling facility are designated under the law to be eligible for augmented rate of subsidies by the State. Delimitation of water-source area and decision of water-source area development plan have been completed for 84dams and the 1 water level controlling facility.

24 projects that can be included in the water-source area for dams

Land improvement, erosion defense, flood defense, road, small-scale water supply system, sewerage, compulsory education facilities, medical clinics, housing esta te development, public rental house, forestry road, collective-use facilities for modernization of operation of agriculture, forestry or fishery, facilities for protection or exploitation of natural parks, community center and other facilities for conservation and utilization of folk cultural properties or tangible cultural properties, sports or recreational facilities, nursery house and other facilities for children, welfare facilities for the cared residence of the elderly, welfare facilities for the handicapped and the elderly, facilities for cable broadcasting or radiotelephone, fire fighting facilities, facilities for disposal of water resulting from stock raising, facilities for disposal of physiological wastes, waste disposal facilities (underlined are projects to which application of augmented rate of subsidies of the state is envisaged.)

2.3.1.3 Project Package under the Water-Source Area Development Plan

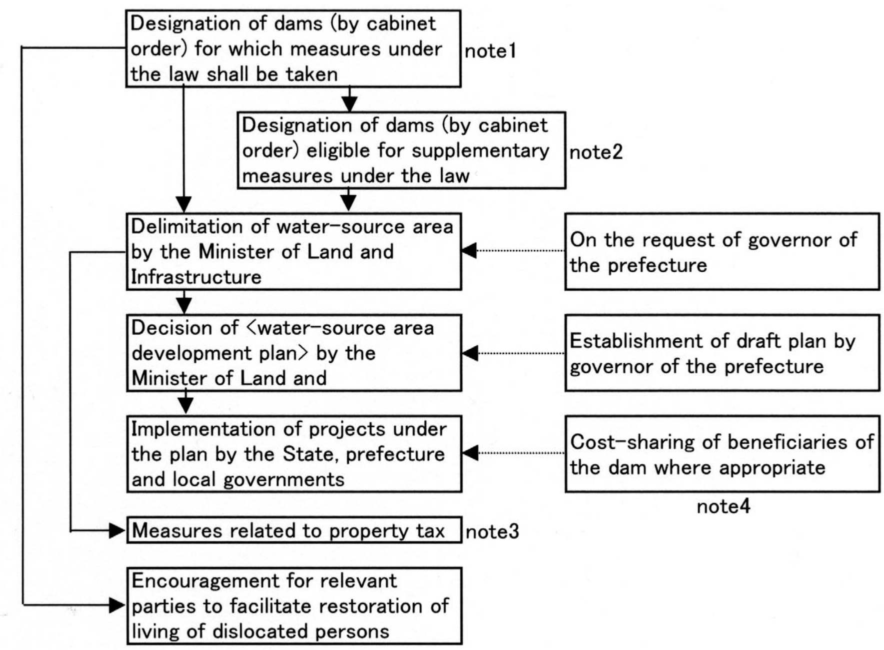
Next table is an overview of 72 water-source area development plans for dams decided by the end of FY1995. No data are available for the other water-source area development plans decided after then.

Overview of water-source area development plan

• estimated project cost for the 72 plans and its distribution among interested parties (million yen)

total aggregate	627,879	
(per project)	8,721	
State	311,079	(49,5%)
prefectures	143,556	(22.9%)
local governments	159,693	(25.4%)

Fig. 1 Flow chart of the scheme



note1: condition for designation
no less than 20 houses or no smaller than 20ha farmland shuold be submerged
(in Hokkaido region, 60ha farmland instead of 20ha)
note2: condition for designation
no less than 150 houses or no smaller than 150ha farmland shuold be submerged
(in case the dam provides extra-ordinary benefits to prefectures that do not contain the water-source area of the dam,75 houses or 75ha farmland)
designation entails application of augmented rate of subsidies by the State for projects under the plan
note3: revenue slump caused by reduced rate of property tax shall be filled by the State
note4: local governments who share cost of a project under the plan can negociate with beneficiaries to agree on cost sharing.
The Minister responsible for the project may intervene as mediator on the request of a negociating party.
<beneficiaries>
water-supply undertakers, industrial water-supply undertakers, electric generators
local governments comprising water-supply area of the dam
local governments comprising flood defense area of the dam

others 13,551 (2.2%)
Others include beneficiaries assuming cost sharing under the law.

- Road is included in most of the 72plans. Other projects included in more than half of the 72plans are land improvement, flood defense, sports or recreational facilities, community center and other facilities for conservation and utilization of folk cultural properties or tangible cultural properties, small-scale water supply system and forestry road.

Source: <Practical Manual for policies on Water-Source Areas>p.202-205, Taisei Publishing Co., 1996 Japan

2.3.2. Aides from Water-Source Area Foundations

Local governments of water-source areas and those of beneficiary areas jointly establish water-source area foundations. Their missions are restoration of living of displaced inhabitants and vitalization of water-source areas.

As for vitalization of water-source areas, their provision of aides can be directed to those dams that are not eligible for the designation under the Special Measures Law for Water-Source Areas. The flexibility of their aide package represents their complementary role to the measures implemented under the same law.

Up to now, there are 13 water-source area foundations, 8 of which have received investment to their core assets by the Ministry of Land, Infrastructure and Transport.

2.3.2.1 Mission of Water-Source Area Foundations (Vitalization of water-Source Areas)

- provision of subsidies for small projects that are not included in the water-source area development plan
- support for various events held in water-source areas such as exchanges between water-source areas and lower-stream beneficiary areas

2.3.2.2 Financial Sources of the Foundations

Business expenses related to the missions of the foundations are financed by contributions from beneficiaries of the dam concerned (local governments). Interests produced by the core funds finance administrative expenses including wages for employees of the foundations.

2.3.3 Water-Source Area Vision

Besides infrastructure development, sustainable engagement of interested parties is essential for vitalization of water-source areas after the end of constructions. In this light, since FY2001, the Ministry of Land and Infrastructure and JWA has undertaken <Water-source area vision> for the dams constructed and operated by them. Key concepts of water-source area vision are ① use of natural riparian environment and cultural heritages ② enhancement of awareness of the importance of water-source area's vitality to maintain quality and quantity of water provided by the dam. Up to now, water-source area visions are established or being drafted for 98 dams. The works for the establishment for water-source area vision are undertaken jointly by the operator of the dam, local authorities, residents of the water-source area, representatives of agriculture, forestry, industry and commerce as well as relevant academicians or specialists.

2.3.4 Dam open to the local community

As public demand for the natural environment and recreation continues to grow, it is being hoped that dams, reservoirs and the surrounding areas are made to play an important role in enhancing the vitality of the regions in which they are located by promoting the use of them as green open spaces and taking measures to conserve the natural environment.

A "dam open to the local community" project aims to enhance the vitality of an area by hearing the opinions of the local community, making effective use of the creative efforts of local residents, making the dam even more open to the local community, improving the dam according to the local needs with the assistance of the organizations concerned, and using the dam as a core element for regional revitalization.

2.3.5 Encouragement and Support for Voluntary Activities

There are varieties of activities that are better promoted voluntary than by operators of dam or under administrative framework. In order to encourage and support these voluntary activities for the purpose of vitalization of water-source areas, <Water Resources Environment Technology Center> (http://www.wec.go.jp) introduced in FY2000 <Water Source Area Support Program>.

Water Sources Area Support Program

The program aims at continuous activities that contribute to independent and sustainable development of water-source areas. Organizations eligible for aids under these programs are corporations (non-governmental organizations, private companies, non-profit organizations), associations (such as volunteer associations) as well as elementary and junior high schools. Financial aid of 500,000yen/year is delivered to each acknowledged organization over a period of 3 years. By FY2003, 22 organizations are acknowledged under this program.

(See Fig. 2.)

3. Example: Hiyoshi Dam

3.1 Hiyoshi Dam

3.1.1 Data on Hiyoshi Dam

(See Table 1, Photo 1, and Fig. 3)

3.1.2 Reservoir area development plan for Hiyoshi Dam

In the case of the Hiyoshi Dam project, a total of about 350 ha or land including about 85 ha of agricultural land and 216 houses was to go under water. In order to prevent serious impacts on the production capability, living environment or other aspects of the adjacent areas, in December 1983 the reservoir area extending over Hiyoshi-cho, Yagi-cho and Keihoku-cho was designated as a "reservoir area" under the Special Measures Law. In March 1984, the Reservoir Area Development Plan for Hiyoshi Dam involving 47 projects was adopted. Key events related to the Reservoir Area Development Plan are summarized below.

Table 1 Data on Hiyoshi Dam

Dam operation and maintenance office	Naka, Hiyoshi-cho, Funai-gun, Kyoto-fu (right bank
Location	35° 08' 51" North, 135° 31' 01" East
River	Katsura River in the Yodo River System
Purpose/Type	FNWI/concrete gravity dam
Dam height/length/volume	70.4 m/438 m/800,000 m3
Catchment area/reservoir area	290K m3/274ha
Total storage capacity/effective storage capacity	66,000,000 m3/58,000,000 m3
Dam owner	Japan Water Agency
Commencement/completion	1971/1997
Name of reservoir (lake)	Lake Amawaka
Random information	*Special Measures Law*: Hiyoshi (Article 9, Designated Dams), total submerged area: 274 ha, the number of submerged houses: 188, the area of submerged agricultural land: 94 ha, date of dam designation: June 2, 1956, date of reservoir area designation: December 6, 1983, date of adoption of plan: March 5, 1984
	Dam open to the local community: (Designated in April 1993)
	The 100 Best Dam Lakes in Japan: Selected one of the 100 Best Dam Lakes designated by Water Resources Environment Technology Center (published on March 16, 2005)

Fig. 2 Evolution of reservoir area development measures

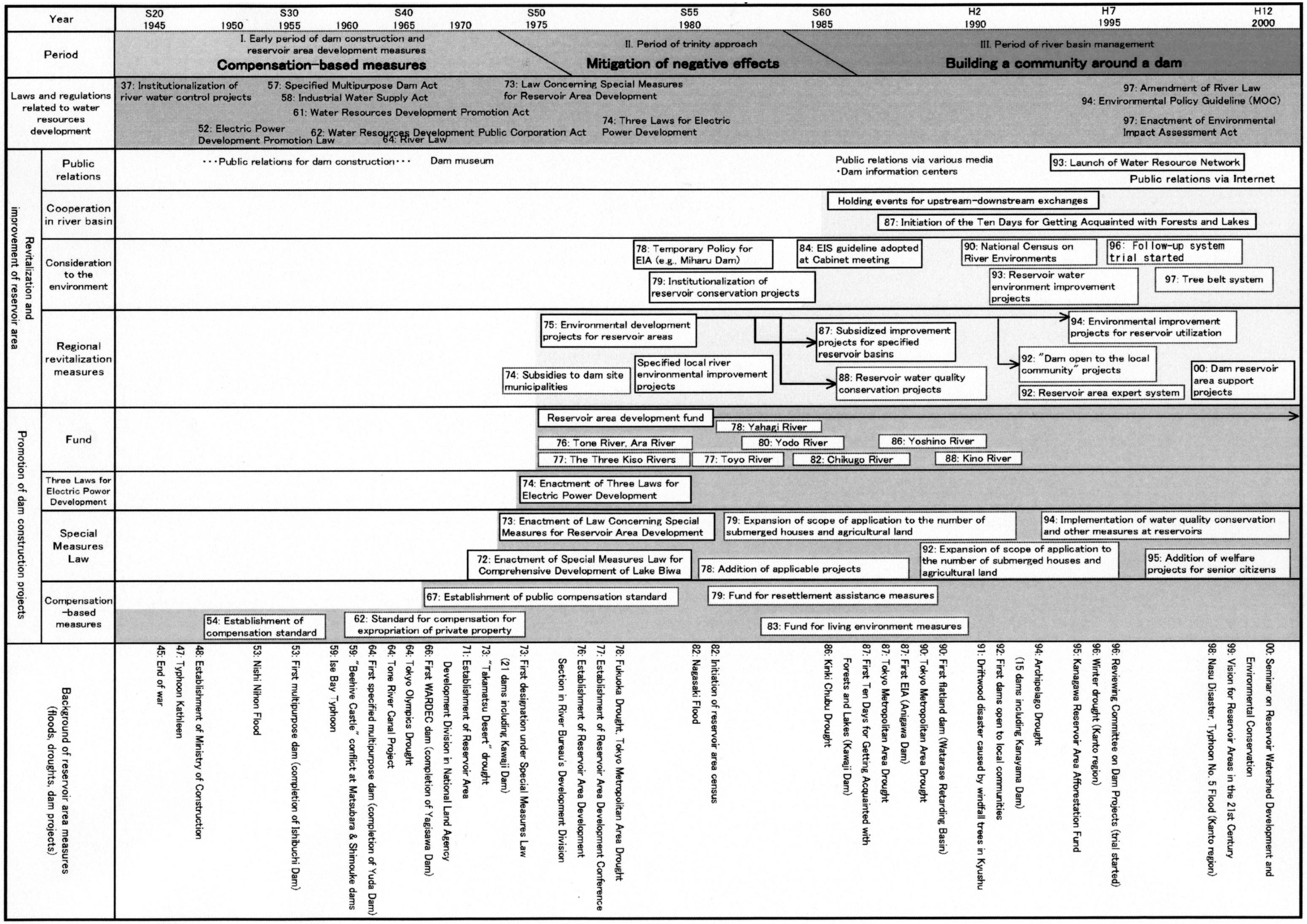


Photo 1 Hiyoshi Dam

- June 1981
 Hiyoshi Dam received designation under Articles 2 and 9 of the Law Concerning Special Measures for Reservoir Area Development.
- December 1983
 Notification on "reservoir area" designation
- March 1984
 Adoption of the Reservoir Area Development Plan under Article 4 of the Law Concerning Special Measures for Reservoir Area Development
 The Kyoto, Osaka and Hyogo prefecture governments agreed on the funding shares to be borne by the downstream communities under Article 12 of the Law Concerning Special Measures for Reservoir Area Development and signed on a memorandum.
- The funding shares to be borne by the downstream communities were paid to the reservoir-area municipality.

Under the reservoir area development plan, measures were taken to mitigate the effects of dam construction on the production capability and living environment of the adjacent areas, stabilize the life of local residents and enhance their welfare. The plan includes a wide range of including land improvement, road improvement, erosion control, flood control, small water-supply systems, joint-use facilities, and sports and recreation facilities. These projects are shown in Table 2)

At the Hiyoshi Dam, a number of projects related to the Special Measures Law that are not classified as subsidized public works projects or Special-Measures-Law projects are implemented to supplement the Special-Measures-Law projects. These projects are shown in Table 3)

For the Hiyoshi Dam project, the Yodo River Reservoir Area Development Fund was established in March 1980 with the aim of contributing to the stabilization of the affected local residents and the growth and pros-

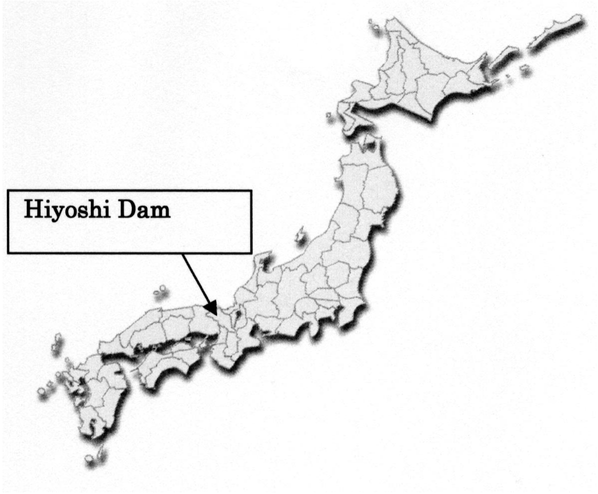


Fig 3 Hiyoshi Dam

Table 2 Reservoir area development projects ("Special Measures Law" projects)

(Amounts in Thousand Yen)

no.	Name of project	Implementing body	Total project cost
1	Reorganization of organization-operated farmland in Tonoda area	Hiyoshi Mun.	115,070
2	Comprehensive development of agricultural infrastructure in Kihata and Kozumi areas	Hiyoshi Mun.	427,649
3	Small-scale farm reorganization in Shimoutsu area	Keihoku Mun.	38,777
4	Small-scale farm reorganization in Chuji area	Keihoku Mun.	47,741
5	Small-scale improvement of old reservoir (Higashi-okajiri Pond)	Yagi Mun.	31,000
6	Improvement of old prefecture-operated reservoir (Tomisaka Pond)	Kyoto. Pref.	642,300
7	Preventive erosion control in Tonoda area	Kyoto. Pref.	8,636
8	Corrective erosion control in Nakasegi area	Kyoto. Pref.	19,833
9	Preventive erosion control in Nakaseki area	Kyoto. Pref.	6,835
10	Corrective erosion control in Kozumi area	Kyoto. Pref.	12,868
11	Preventive erosion control in Kozumi area	Kyoto. Pref.	18,097
12	Improvement of the Tawara River	Kyoto. Pref.	1,166,752
13	Improvement of the Goma River	Kyoto. Pref.	992,272
14	Smaller stream improvement for the Katsura River	Kyoto. Pref.	399,438
15	Improvement of the Akeshi River	Kyoto. Pref.	206,627
16-1	Improvement of Prefectural Road Kyoto Hiyoshi–Miyama Line	Kyoto. Pref.	2,917,000
16-2	Improvement of Prefectural Road Kyoto Hiyoshi–Miyama Line	Kyoto. Pref.	2,888,049
16-3	Improvement of Prefectural Road Kyoto Hiyoshi–Miyama Line	Kyoto. Pref.	2,413,000
17-1	Improvement of Prefectural Road Sonobe–Hiraya Line	Kyoto. Pref.	234,000
17-2	Improvement of Prefectural Road Sonobe–Hiraya Line	Kyoto. Pref.	1,947,000
18	Improvement of Prefectural Road Chuji–Hiyoshi Line	Kyoto. Pref.	847,176
19	Improvement of Prefectural Road Atago–Yuzuki Line	Kyoto. Pref.	603,400
20	Improvement of Municipal Road Asahiyama Line	Hiyoshi Mun.	584,912
21	Improvement of Municipal Road Kihata Line	Hiyoshi Mun.	461,110
22	Improvement of Municipal Road Kashiwagi Line	Hiyoshi Mun.	1,184,000
23	Improvement of Municipal Road Omukai Line (provisional name)	Keihoku Mun.	4,252
24	Improvement of Municipal Road Omukai Line (provisional name)	Yagi Mun.	4,265
25	Improvement of Municipal Road Kojo Line (provisional name)	Keihoku Mun.	73,550
26	Improvement of Municipal Road Shimoutsu Line (provisional name)	Keihoku Mun.	464,200
27	Improvement of Municipal Road Kamiyoshi–Hidokoro Line	Yagi Mun.	418,866
28	Construction of Kamiyoshi small drinking water supply system	Yagi Mun.	140,000
29	Reconstruction, etc., of the main building of Tonoda Junior High School	Hiyoshi Mun.	255,605
30	Construction of forest road Ikegaya Line	Keihoku Mun.	315,300
31	Construction of Rural Environment Improvement Center	Hiyoshi Mun.	185,000
32	Construction of rice processing facilities	Hiyoshi JA	257,538
33	Construction of Agricultural Product Processing Center	Hiyoshi JA	115,718
34	Construction of freshwater aquaculture facilities	Hiyoshi Mun.	15,465
35	Construction of fishpond facilities	Keihoku Mun.	363,662
36	Construction of Kozumi meeting facilities	Hiyoshi Mun.	32,354
37	Construction of Hiyoshi Dam Memorial Park (provisional name)	Hiyoshi Mun.	1,459,314
38	Construction of rest facilities for cyclists	Hiyoshi Mun.	77,072
39	Construction of "Fumin no Mori" forest park	Kyoto. Pref.	1,091,537
40	Construction of Tonoda sports ground	Hiyoshi Mun.	106,291
41	Reconstruction of Central Day Care Center	Hiyoshi Mun.	174,579
42	Construction of fire-fighting facilities	Hiyoshi Mun.	46,222
43	Construction of fire-fighting facilities	Keihoku Mun.	7,889
Total			23,822,221

Note: Total project cost as of the end of fiscal 2001. The costs of ongoing projects are estimated costs of completed projects (as of the end of fiscal 2001). The costs of municipal projects include funding shares for water utilization provided under Article 12 of the Law Concerning Special Measures for Reservoir Area Development.

Table 3 Reservoir area development projects
(Projects related to the "Special Measures Law")

(Amounts in Thousand Yen)

no.	Name of project	Implementing body	Total project cost
1	Reorganization of prefectural farm in Hiyoshi area	Kyoto Pref.	1,256,700
2	Improvement of the Tawara River	Kyoto Pref.	263,359
3	Improvement of the Kozumi River	Kyoto Pref.	58,650
4	Improvement of Prefectural Road Sonobe–Hiraya Line	Kyoto Pref.	4,167,877
5-1	Prefectural Road Kyoto Hiyoshi–Miyama Line (Tonoda work section)	Kyoto Pref.	539,000
5-2	Prefectural Road Kyoto Hiyoshi–Miyama Line (Shimo-honoda work section)	Kyoto Pref.	195,000
6	Improvement of Prefectural Road Tomita–Goma Depot Line	Kyoto Pref.	196,000
7	Improvement of Municipal Road Kihata Line	Hoyoshi Town	671,726
8	Improvement of Agricultural Road Katano–Honoda Line	Hoyoshi Town	479,400
9	Improvement of low-traffic trunk agricultural road in Goma	Kyoto Pref.	569,500
10	Construction of Forest Road Teratani Line	Hoyoshi Town	32,100
11	Construction of Forest Road Kakutani Line	Hoyoshi Town	6,600
12	Construction of "Fumin no Mori" forest park	Kyoto Pref.	1,373,033
13	Construction of municipal gymnasium	Hiyoshi Town	800,110
14	Reorganization of organization-operated farm in Kamiutsu area	Keihoku Town	169,024
15	Reorganization of organization-operated farm in Akeshi area	Keihoku Town	466,558
16	Smaller stream improvement for the Katsura River (Chuji to Kashiwara)	Kyoto Pref.	300,000
17	Improvement of the Hosono River	Kyoto Pref.	53,559
18	Improvement of Prefectural Road Yagi–Shuzan Line	Kyoto Pref.	1,508,518
19	Improvement of Prefectural Road Chuji–Kumata Line	Kyoto Pref.	307,046
20	Improvement of Prefectural Road Miyanotsuji–Kamiyoshi Line	Kyoto Pref.	515,864
21	Improvement of Municipal Road Nishi-uno Line	Keihoku Town	516,107
22	Improvement of Municipal Road Utsu–Segi Line	Keihoku Town	25,784
23	Construction of forest road Tonodani Line	Keihoku Town	4,088
24	Construction of forest road Mikodani Line	Keihoku Town	3,443
25	Construction of forest road Takatani Line	Keihoku Town	74368
26	Construction of forest road Tanotani Line	Keihoku Town	4558
27	Reconstruction of Utsu Day Care Center	Keihoku Town	77,822
28	Reorganization of prefectural farm in Kamiyoshi area	Kyoto Pref.	1,333,300
29	Improvement of Nishi-okajiri reservoir	Kyoto Pref.	850,000
30	Smaller stream improvement for the Katsura River (Yagi-cho, etc.)	Keihoku Town	463,837
31	Erosion control for the Okutsuhara River	Kyoto Pref.	25,000
32	Local improvement of the Umada River	Kyoto Pref.	351,113
33	Improvement of Municipal Road Aoto–Hidokoro Line	Yagi Town	331,000
34	Construction of a branch road off the forest road Umenoki Line	Yagi Town	3,700
	Total		17,993,744

perity of the areas to be submerged, and various assistance projects are being implemented.

These projects include providing interest subsidies to affected local residents who wish to acquire real estate such as substitute lots, paying the expenses for occupational training in cases where job conversion is necessary, providing living counselors, and conducting studies on measures to be taken to revitalize the affected areas.

Funds for Hiyoshi Dam

As of the end of fiscal 1997 (Amounts in thousand yen) (Table 4)

In April 1993, Hiyoshi Dam was designated as the first "dam open to the local community." Since then, efforts have been underway, in cooperation with three local towns (Hiyoshi-cho, Yagi-cho, Keihoku-cho), to improve the environment of the dam and reservoir area in order to enhance the vitality of the region, aiming to achieve the basic goal of "creating a healthy, culturally rich community founded on the natural environment."

The process of "dam open to the local community" planning is summarized below.

- April 1993
  Received the "dam open to the local community" designation from the Director-General of the River Bureau, Ministry of Construction.
- July 1993
  The Hiyoshi Dam Environmental Improvement Council was established.
- February 1994
  The "Dam Open to the Local Community" Plan drawn up by Hiyoshi-cho was approved by the Director-General of the River Bureau.
- February 1995
  The plan was expanded to involve three towns (Hiyoshi-cho, Yagi-cho, Keihoku-cho) in the dam area.

The projects implemented under the "Dam Open to Local Community" Plan for Hiyoshi Dam are shown in Table 5.

3.1.3 Vision for the Reservoir Area

A "Vision for the Reservoir Area" is an action plan that the local government and residents of the dam reservoir area and the dam owner and the dam manager jointly draw up with the cooperation of the local governments of the downstream areas and the administrative organizations concerned in order to enhance the vitality of the reservoir area as an independent, sustainable community by making effective use of the dam.

A vision is drawn up by establishing an organization composed of the dam owner and the dam manager, local governments in the river basin, local residents, administrative organizations concerned, academic experts, etc., and using a method that makes it possible to reflect the opinions of the stakeholders in the reservoir area.

Table 4 Funds for Hiyoshi Dam
As of the end of fiscal 1997

(Amounts in thousand yen)

Name of project	Planned amount	Actual amount	Period (year)	Funding entity
Resettlement assistance projects	254,000	206,816		Kyoto Pref.
1. Real estate acquisition subsidizing projects	127,000	97,466		31.3% (1.16t/s)
(1) Project for subsidizing interest for the Kyoto Prefecture system	126,000	96,958	1985 to 1987	Osaka Pref.
(2) Project for subsidizing interest for relocation in Hiyoshi-cho	1,000	508	1985	42.6%(1.576t/s)
2. Occupational training allowance subsidizing project	9,000	0		Hyogo Pref.
3. Livelihood counseling personnel cost subsidizing project	19,000	10,350	1985 to 1987	26.1%(0.964t/s)
4. Special financial assistance project	99,000	99,000	1985	Itami City
				5.7%(0.21t/s)
Regional revitalization projects	65,000	65,000		Hanshin Water
1. Local museum construction project	50,000	50,000	1997	Supply Authority
2. Study subsidy projects	15,000	15,000	1985	20.4%(0.754t/s)
Total	319,000	271,816		

Table 5 Projects implemented under the "Dam Open to Local Community" Plan for Hiyoshi Dam

Name of project	Implementing body	Year	Relationship with the "open to local community" dam
Dam memorial park construction project	Hiyoshi Town	1995–1999	"Special Measures Law" project
Project for the construction of rest facilities for cyclists	Hiyoshi Town	1993–1996	"Special Measures Law" project & Specified local river environmental improvement project
Lake surface leisure facilities construction project	Hiyoshi Town	1997–1998	"Special Measures Law" project
Camp site construction project	Hiyoshi Town	1996–1998	"Special Measures Law" project
Local museum construction project	Hiyoshi Town	1996–1998	Yodo River Fund project
Youth outdoor activity facilities, etc.	Hiyoshi Town & Keihoku Town	1996–	"Special Measures Law" related project
Stream park construction project (provisional name)	Keihoku Town	1996–1997	"Special Measures Law" project
Segi Dam reservoir area development project	Yagi Town	1995–	Specified local river environmental improvement project
"Fumin no Mori" forest park construction project	Kyoto Pref.	1996–1998	"Special Measures Law" project
Dam construction project	Japan Water Agency	1982–1997	* Improvement of Prefectural Road Hiyoshi–Miyama Line * Ring road construction for making effective use of municipal roads including Amawaka Line, Yamago Line and Omukai Line * Construction of a gallery in the dam body, observation platform, project presentation hall, etc.

Work to draw up visions for 23 dams in Japan (dams managed by the Ministry of Land, Infrastructure and Transport and dams managed by Japan Water Agency) began in 2001. The vision for Hiyoshi Dam was drawn up and announced and published toward the end of March 2002. Currently, the plan is being implemented through joint efforts led by the local governments.

3.1.4 Usage

According to dam reservoir usage survey results, the number of visitors to Hiyoshi Dam in 2000 (from the spring of 2000 to the winter of 2003), the first year of survey, was about 870,000. The number of visitors in 2003 was 530,000, a decrease of about 340,000. (Fig. 4)

According to o survey results by type of use in 2003, "use of facilities" such as museums and hot springs accounts for about 38 percent, followed by "outdoor activities" using forest parks, sports grounds, etc., account for about 23 percent. "Boating" use ranked at the bottom, accounting for about less than 0.1 percent.

Comparison of the results of the two surveys reveals that the percentage of "outdoor activities" increased, indicating that the number of users of parks and other facilities that are integrated with the dam body is on the increase. (Fig. 5)

Characteristics of the dams in Japan that attract the greatest numbers of visitors (five most used dams) and the types of facilities and places that attract many users are depicted in Photos 2, 3 and 4.

3.2 Iwaya Dam (Water-Source Area Vision)

In addition to its flood defense function, Iwaya dam provides agricultural water, domestic water and industrial water totaling 45.69t/s to Kiso river area including Nagoya city.

Its water-source area vision, established in March 2003, installs in itself operation management cycle <plan → do → check → action> following the principle of policy evaluation (Figure 6).

3.3 Sameura Dam (Water-Source Area Vision)

Sameura dam is constructed and operated by JWA. Besides the important role it plays for the flood defense of Yoshino river basin, it provides agricultural water 18.4t/s, domestic water 7.44t/s as well as industrial water 14.11t/s. It is also used for electric generation.

In the Sameura dam water-source area vision established in February 2002, priority was set on promoting integrated use of its reservoir lake, the largest in Shikoku region. Accordingly, <reservoir surface use plan> was established in January 2005. (Figure 7)

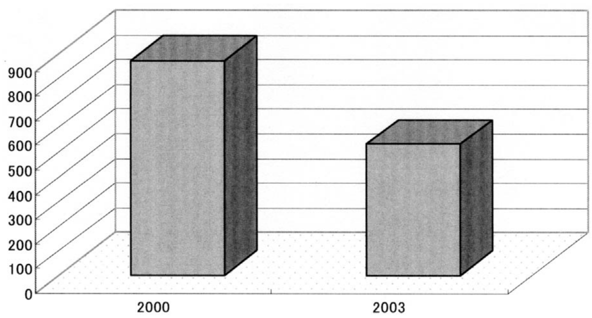


Fig. 4 Changes in the number of visitors per year (numbers in thousands)

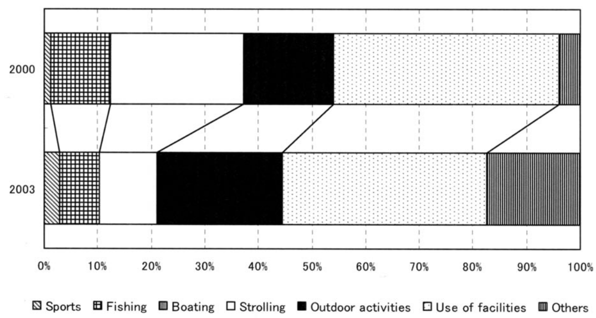


Fig. 5 Changes in the percentages of different reservoir uses

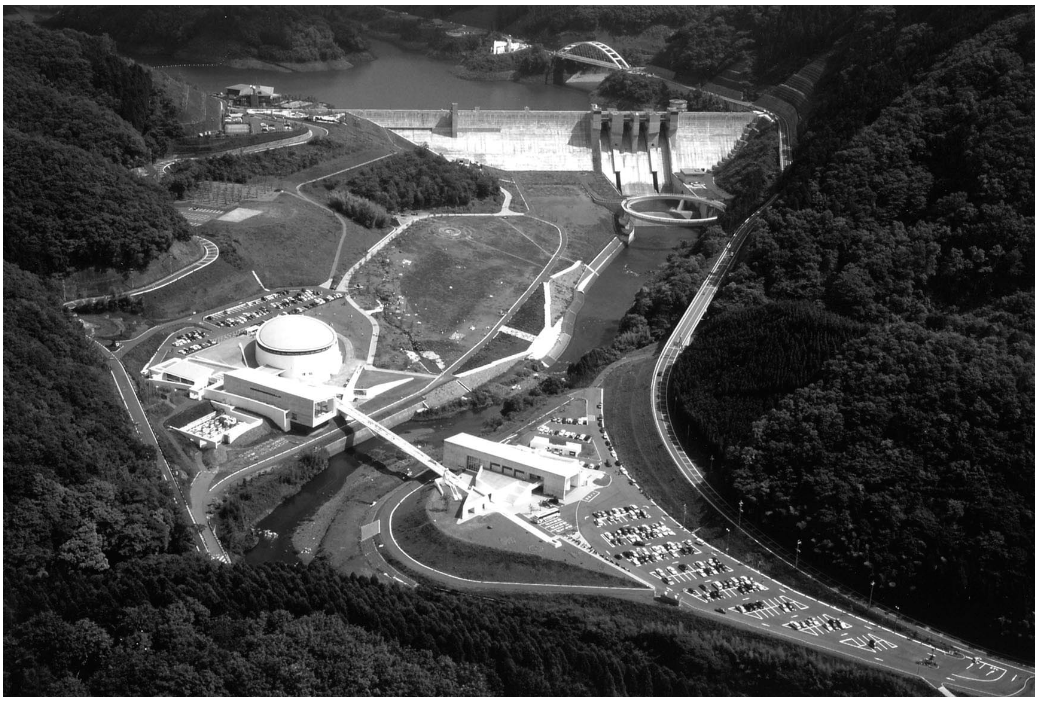


Photo 2 Hiyosh dam and springs park

Photo 3 Visitor Center

Photo 4 "Fumin no Mori" forest park

<outline of the reservoir lake use plan>

α basic principles

protection of dam's utilities

Necessary measures should be taken lest use of reservoir surface not create such troubles as;

Hinderance to the dam operations

Physical damages to operational facilities or reservoir banks

Diminution of reservoir capacity or reduction of outlet volume due to structures for reservoir surface use

protection of environment

Attention should be paid in order to prevent degradation of envirinment such as deterioration of reservoir's water quality, landscape decline and production of pollutive wastes.

inexclusiveness

Reservoir surface of dams are created for public purposes and creates a part of river space. Under the river law(Law No.167 of 1964), oppertunities for the use of river space should be open to the general public.

So delivery of reservoir surface use licence should make sure that this principle of its inexclusiveness is not compromised.

β practical rules for reservoir surface use

Users by boats are requested to obtain membership of the Sameura dam reservoir users' council (annual membership fee is 4000yen).

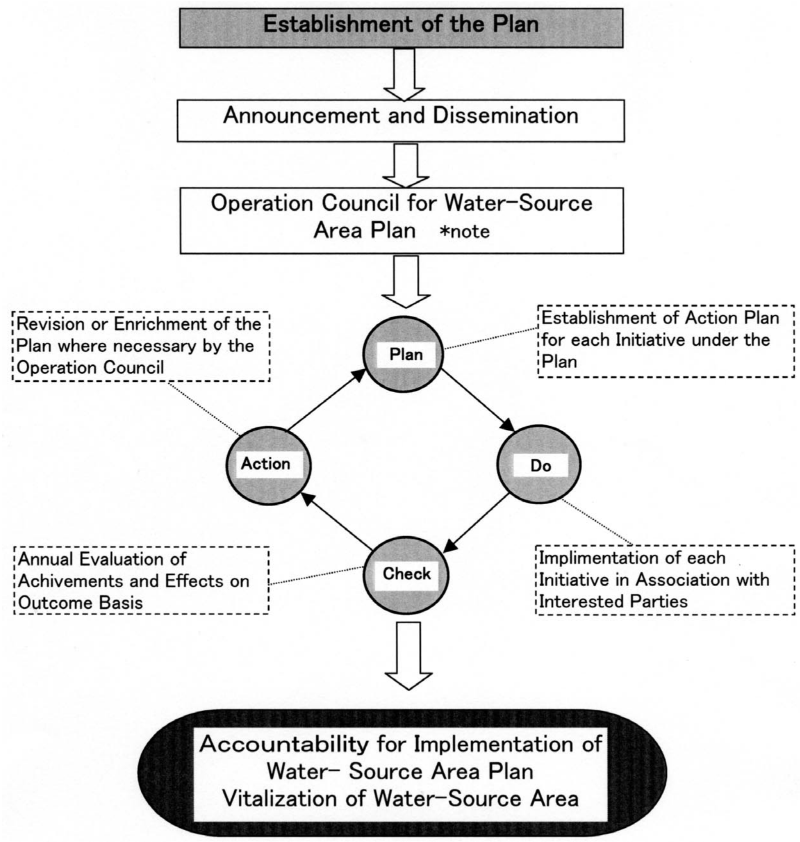
Camping and swimming are prohibited.

Reservoir surface use is possible when the inflow volume to the reservoir does not exceed 100t/s.

Designation of access route to the reservoir

Designation of non-parking areas in the reservoir side

Prohibition of sctivities causing damages to the



Note: Operational council is composed of those responsible for implementation of each initiative under the water-source area vision, that is, the operator of the dam, local authorities, representative of agriculture, forestry and commerce and other relevant parties

Fig. 6 Establishment of Iwaya Dam Water-Source Area Plan

Photo 5 Reservoir Surface
750ha (Approx.)

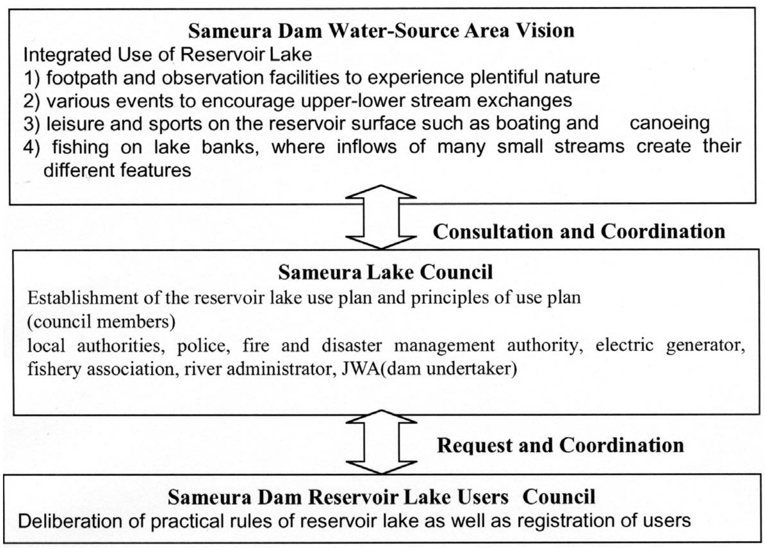


Fig. 7. Organization for the Establishment of the Reservoir Surface Use Plan

water quality or muisances to other users.

Users are strongly advised to carry back home their wastes.

4. An example of a reservoir area support business

- Misogawa Dam (constructed and operated by JWA)
  Acknowledged Organization: association for promotion of upper-down stream exchanges

Performance of folk entertainment by children of Kiso river basin, exchange of messages, installation of time capsule, monuments with poems and promotion of summer-camp as field of exchanges.

- Tamagawa Dam (constructed and operated by Ministry of Land and Infrastructure)
  Acknowledged Organization: forum on the history and culture of Tama river basin

This forum makes researches of the history and cultures of Tama river basin area including field works on its life, industry, economy and culture, thereby presents proposals to integrate its rich natural and historical resources into community building.

- Nibutani Dam (constructed and operated by the Hokkaido Development Bureau of the Ministry of Land and Infrastructure)
  Acknowledged Organization: Hidaka town forest lovers' association

Its activities center on consciousness-building on the importance of forest protection including fire prevention. The association also organizes environmental education programs for pupils in the river basin such as rafting, forest trekking and observation of mushrooms.

5. Challenges for the future

5.1 Severe Economic and Financial Circumstances

Quite naturally, financial resources that can be affected to measures for water-source areas are not unlimited in the context of severe economic and financial circumstances. Especially, difficulty lies in obtaining funds for measures after dam construction is completed and operation of dam is started. Besides delivery of large blocks such as government grants and installment paid by beneficiaries, various initiatives should be promoted to meet ground needs of water-source areas.

5.2 Increasing maintenance cost

Many of the municipal governments in the reservoir areas feel that dam construction has been beneficial because facilities in the reservoir areas have been improved. Struggling with increased maintenance cost, however, many municipal governments wish to receive financial assistance from dam owners.

In cases where a management organization has been established, if the management organization is unable to obtain financial assistance, it may become impossible to maintain the environment of the dam reservoir area.

Another challenge is to find ways to maintain facilities whose attractiveness will decrease over time as they become old.

The reason for this kind of problem may be that the facilities constructed in connection with dam projects may be too large to support financially for the local governments of the reservoir areas or that a labor shortage results as volunteer workers engaged in the management of facilities become older. Some say that changes should be made to the current system so that facilities in dam reservoir areas can be maintained as "parks" under the City Planning Law.

There are cases where dam owners provide assistance such as providing information on projects eligible for subsidies and reducing labor cost by mechanization. In many cases, however, more improvements are hoped for.

5.3 Attractiveness of adjacent facilities

Decreasing attractiveness of adjacent facilities because of, for example, the natural deterioration of the facilities, failure of the facilities to meet the needs of the time and inadequacy of functions (e.g., adjacent roads, parking lots, toilets, information signs) is being seen as a problem.

The local governments in the reservoir areas hope to make effective use of the adjacent facilities for the purpose of regional development. They hope, therefore, that effective use is made of dams by, for example, tying up with local events or building tourist routes so that people can visit the reservoir areas.

The "Vision for the Reservoir Area" that has been drawn up for many dams in the country since 2001 aims to make effective use of adjacent facilities and enhance their attractiveness by reflecting the opinions of local communities in the facility plans. The "Vision for the Reservoir Area" scheme is expected to help make the adjacent facilities more attractive.

5.4 Upper-Lower Stream Exchanges

For sustainable and stable use of domestic and industrial water, it is necessary to promote mutual understanding and partnership on the relevant issues such as the roles of forests, protection of river water quality, impacts of dam construction on water-source areas and effective use of water.

From this standpoint, more efforts need to be made to encourage upper-lower stream exchanges.

A Paradigm Shift in Water Resource Development in Korea

Kyungtaek Yum
Manager Ph.D., KAWACO, Daejeon, Korea

Heekyung Park
Professor, Department of Civil and Environmental Engineering, KAIST, Daejeon, Korea

Wanho Lee
Director, KAWACO, Daejeon, Korea

Abstract

Until now, water resources were developed as the most essential public commodities for assuring the stable supply of power for industrial development as well as improving quality of life. However, in accordance with economic development, public awareness has evolved over time. In addition, societal views regarding water resources are demanding greater consideration of environment-friendly, resident-friendly and society-friendly aspects rather than simply looking at economic benefits. A move toward a new paradigm is being pushed for in water resource development. The demands of the times are judged to be fair and should be duly accepted. The period that the small was sacrificed for the large has already gone in water resource development. Political endeavor for clean and sufficient water resource and better environment is now realized all over the country and will go further. Because positive participation and productive plans from residents and NGOs is essential for it, lots of cooperation and rebuke are required.

1. INTRODUCTION

Water resource development has been promoted by central government as not only a stable guarantee of growth power energy for the national industry's development but also the most indispensable public goods for the improvement of life-quality. The multipurpose dam, one of the water resource developments, has been adapted as a most effective device to be aimed at the economic development in the relation with the output of large-scale construction industry, as a matter of course including its fundamental functions like water supply, flood control and hydraulic power supply. From that, we can have achieved the ability to supply enough industrial and domestic use in addition to supply national and non-pollutant electric power.

However, paradigm shift about water resource development is strongly claimed along with the change of people's consciousness nowadays. They need to focus the importance on the friendly aspect to environment, residents and society prior to the simple economic value. Achieving the active and positive reflection upon this kind of needs, current direction of the policy for the development and the management of water resource has been being rapidly changed. For example, there are some cases which promote the activation of local economy by using riverside place as the medium for amenity improvement especially of rural communities. It also make institutional devices to turn the material benefit from a dam construction to the people in the submerged districts and the local society.

Although the dam reservoir has clear water and green forests which are the potential for recreation and sightseeing, development for resident has been limited. Recent policies are to gradually open the reservoir site, redevelop the submerged area, and arrange them for the improvement of life-quality. Besides, lots of technical professionals of water-resources do their best efforts to resolve the goal, the construction of natural-balanced and beautiful appearance dams.

2. HIERACHICAL CHANGE OF THE SURROUNDINGS FOR WATER RESOURCE DEVELOPMENT

2.1 Unusual change in the weather and flood damage

2.1.1 Increase of torrential rain

Currently the global climate change including Korea is getting distinctive. The global warming which already caused the temperature increment of 0.6 degree for the past 140 years is still going on by now and this trend is also applied naturally to Korea after 80s.

The number of the torrential rain over than 100mm for a day is 222 from 1971 till 1980, but 325 days are recorded from 1992 to 2001. It means the frequency of the heavy rain occurrence increased by 1.5 times than before. Particularly, the amount of daily rainfall in Kangneung on August in 2002, was 870.5mm, the biggest precipitation ever from the first observation of 1911. This increase of the torrential rain under the situation that there is little change of the yearly precipitation and the number of typhoon occurrence is a raining pattern accompanied by the current climate change.

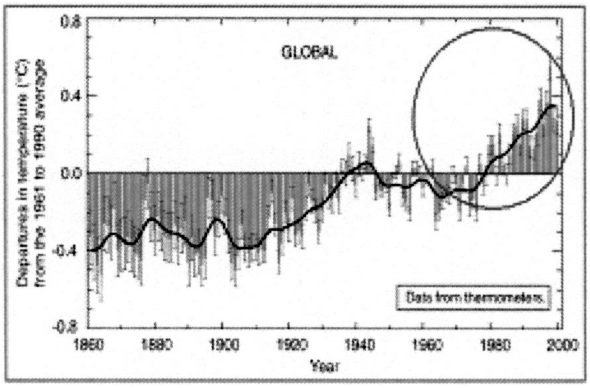


Fig. 1 Yearly variations of global temperature (from 1961 to 1990)

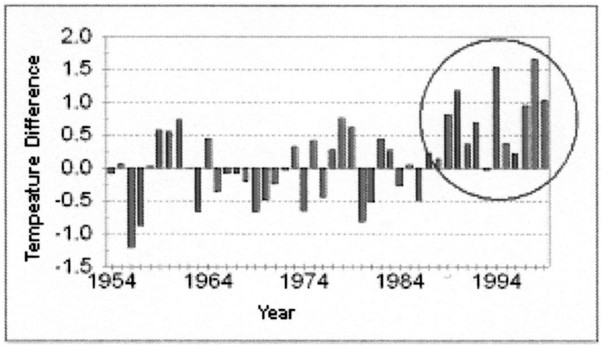


Fig. 2 Yearly variations of temperature in Korea (from 1954 to 1999)

2.1.2 Flood damage

In spite of the continuous investment of structural and non-structural countermeasure, flood damage is sharply increasing. The annual average damage cost from flood for the last 10 years is up to 3500 trillion won which is also including 6 trillion and 1153 hundred million won of 2002, the largest damage ever. The fortunate fact is that the decrease of human injury to less than half compared with the one during 70s and 80s. Another remarkable thing is that the damaged area by flood become decreasing but the cost over the area is now about 7 times to the cost of 70-80s.

2.1.3 Defense countermeasure against flood

The flood control projects are now actively promoted. They include the extension of bypass-waterway and the raise of dam height to improve the flood control ability of existing dams and finally to protect nation's welfare. Recently, the design criterion of dam construction became improved by applying PMF (Possible Maximum Flood) to the design flood amount and this criterion has already been applying to the new small-scale dam construction.

2.1.4 The Need for Increasing Flood Defense Capabilities in Small-scale Basins

Recently, flood defense methodology in Korea has seen the introduction of new and innovative concepts, rather than continued reliance on traditional concepts which focused mainly on large rivers such as the Han and Nakdong.

— *Concept change to the way of flood control area defense from river bank defense*
- Various flood defense facilities within a basin, such as flood detention, outlets, underground currents, underground dams etc., and the optimized cooperative operations of these facilities can better defend against flooding by dispersing the flood discharge throughout a basin (this avoids line concepts such as embankment expansion)

— *Introduction of flood discharge allocation*
- The allocation of flood discharge in advance allows utilization of embankment defenses of individual-river and alleviates the peak loads of embankments. As well, excess flood discharge will be mitigated by developing undercurrent sites within a basin and by developing comprehensive disaster prevention plans.

— *Introduction of selective defense*
- Water damage in small areas that are commonly inundated can be prevented by evacuating and relocating the regions residents. Providing affordable housing rather than building embankments yields a better economic return in many cases.

To implement these new flood defense measures, older measures focusing on large rivers must be combined with the newer methods thus ensuring an increase in flood defense capability within small-scale basins. These methods include the construction of

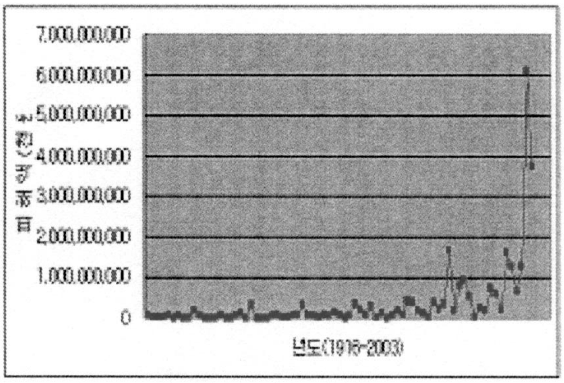


Fig. 3 Yearly damage cost by flood

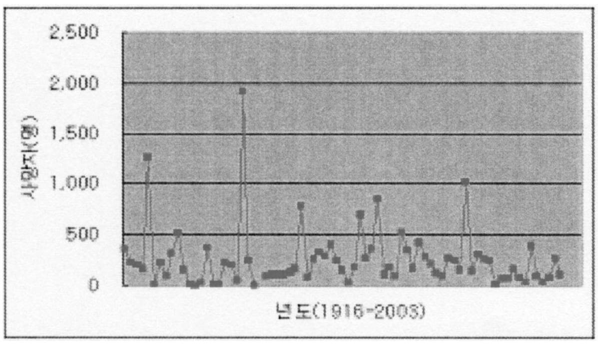


Fig. 4 Yearly loss of lives by flood in Korea

Table 1 Comparison of water coverage among regions

	Population	Service Population	Coverage (%)	
Nationwide	48,517	43,022	88.7	
City	38,896	38,068	97.8	
Small Town	9,621	4,954	51.5	52.6% compared to city

□ Source : Water service statistics ('03, Ministry of Environment, Korea)

debris barrier and small-medium scale multi-purpose, as well as, flood defense purpose dams. Finally, it must be possible to co-ordinate flood control management ability with optimized customer service.

2.2 Need for management about the water utilization and control in the small-scale watershed

Until now, multipurpose water resource development projects have been promoted in the large-scale and nationwide solution for the water utilization and control. In result, building large dams are difficult because the suitable lands for them have been reduced considerably, and geographical and social circumstances have been changed such as increase of the land price due to urbanization, largely submerged people, increase of the development cost by increase of the needed compensation cost and supporting business. Therefore, the recent trend in the water resource development is building the small- and medium-scale dams. It aims at raising self-defense capability against the flood in comparatively weak watershed basins and growing the water coverage at the secluded villages larger. Also, the other schemes are given carefully to the utilization of the valuable riverside place of reservoir water for the rural amenity, helping the regional economy. These schemes have been applied lately to the Hwabuk dam, the Seongdeok dam and Sinpung dam. These dams aiming at the irrigation originally are redeveloped into the middle- or small-sized multipurpose dams, and expected to solve some problems of the water utilization and control in the small-scale watershed basin and grow the rural amenity larger.

2.2.1 Need for flood control in the small-scale watershed

The proceeding countermeasures against the flood have the epochal concepts compared with the past cases, applying to large rivers like the Han and the Nakdong.

— *Application of the areal dispersion concept for the self-defense within the watershed*

- The volume of flood is defended dispersedly by using of the diversification and optimum connection of relevant facilities including undercurrent places, control areas, flood control channels, underground watercourses and dam within the watershed basin. (it means that the past 1-dimensional concept depending on the height of embankment are escaped from.)

— *Introduction of the quota system of the flood volume*

- Each river is allotted to the charged volume against the flood so that burdens of embankments are reduced and the remaining portion turns into the undercurrent places or regional schemes for prevention of disasters.

— *Introduction of selectively defense concept*

- In the small-scale and habitual flooded area, flood damage is withheld by the resident emigration into the rental dwellings instead of higher embankment construction.

To complete these countermeasures against the flood, policies should be interrelated with viewpoints of large river and demand of consumers such as constructing soil saving dam upstream in small river, small- and medium-scale multipurpose dam and dam only against flood for increasing flood defense capability.

2.2.2 Need for management about flood control in the small-scale watershed

In most cities, water supply is sufficient and safe through the multi-regional water supply system from large multi-purpose dams. In 2003, average water coverage of cities is reached up to 98%. However, coverage is just 51.5% in small towns and remote mountain area.

To supply safe and sufficient water in this undeveloped region, a national policy would include small dams construction for increasing self-sufficing water supply.

2.2.3 Prerequisite : Sustainable development

Since the World Commission on Environment and Development introduced the concept of "environmentally sound and sustainable development" (ESSD) in its 1987 report "Our Common Future," also known as the Brundtland Report, sustainable development has become a proposal that engineers can no longer refuse. Multi-purpose dams have been and are still playing a crucial role as a major driving force in social and economic development. Nonetheless, we cannot deny the fact that they have been regarded as the main culprit of environmental destruction. As part of efforts to cast off past disrepute, however, and be newly viewed in the light of ecologically sound and sustainable water resource development, the construction of environmentally friendly dams is highly valued. In the same vein, this concept is rigorously incorporated in newly planned dams as well as existing dams.

Recently, to prevent social conflicts between development and preservation, revision of "Environmental Policy Act" was announced for turning present "prior environmental review system" into new one which has a concept and priciple of "Strategic Environmental Assessment." It aims at evaluation of validity of the project through considering environmental aspect in all steps from planning to execution and collecting opinions among the stakeholders.

3. PARADIGM SHIFT AND ITS CASES

3.1 Countermeasure for supporting dam area

Government adds leisure and tourism to purpose of the dam to stimulate local economy. Ultimate object of this policy is to develop local community by promoting the amenities of rural areas. To achieve this object, some of laws for dams are revised. These revisions are evaluated as good opportunity to give not only residents but also neighborhood all direct and indirect profits from the development. Even though the residents hesitate to take part in policy because of their distrust against central government, an atmosphere has become gradually created to gain the confidence about it.

3. 2 Countermeasure for supporting local residents

The ultimate achievement of simulating local economy through water resource development lies in improving the quality of life of residents in areas to be submerged and their neighbors and providing them with a better economic environment.

The most important element in paradigm shift in water resource policy would be how to help these residents comfortably adapt to a new environment and find a new way of life through better means based on government support projects. After all, realistic answers to these questions would be found through tripartite cooperation and collaboration among local authorities, residents and central government.

For instance, a project that departs from the traditional practice of collectively relocating residents to newly created settlements and instead creates self-sufficient settlements or joint production bases using the concept of lifetime service (renting out villas or offering of comprehensive services including the co-production, management and sale if local specialties or organic agricultural products) may be a good example. A mechanism in which these facilities are operated and managed by an autonomous farming organization composed of residents with profits shared among its members may also be a recommended model. Residents will benefit directly from indirect support by the government.

In addition, water resource developers should proactively assist with and support in the locating of markets for these produce. Developers should fulfill their responsibilities in promoting the amenities of rural areas by jointly developing events with local residents.

Moreover, local economies should be further stimulated through the creation of a Tourism Belt by linking water sides of dams to local tourism resources.

Table 2 Overviews of supporting projects (unit: 100 million won)

Dams	Fund for Public Affairs	Fund for Resident Supporting	Fund for Dam Utilities
Hantangang	495	4 - 8	259
Hwabuk	340	4 - 8	undecided
Gamcheon	355	4 - 8	70
Songliwon	450	4 - 8	undecided
Seongdeok	320	4 - 8	79
Sinpung	320	4 - 8	undecided

Public Affairs (the law for dams Article 41, an Enforcement Ordinance Article 36)
○ The projects to improve regional economy and life environment for dams and its residents during the period of dam construction ○ Fund : about 30-50 billion won(KRW) ○ Allotment : the company of dam construction 90%, local government 10% ○ Area : less than 5km of dam from the flood line officially-authorized area ○ Main contents : project for productiveness, project for welfare and culture, project for public facilities

Dam Utilities (the law for dams Clause 2, Art.18)
○ The projects not only to build dam facilities for operation and management but also to stimulate local economy ○ Main contents : building parks and facilities for the physical exercise

Support projects for environs (the law for dams Article 43, an Enforcement Ordinance Article 40)
○ The projects to improve residents' income and welfare after dam construction ○ Fund Procurement: 6% from sale of electric power, 20% from sale of water ○ Fund : 500 million to 1 billion won per year (continued after dam construction) ○ Area : less than 5km of dam from the flood line officially - authorized area ○ Main contents : project for income raise, project for welfare promotion, project for education

Fig. 5 Contents of supporting projects

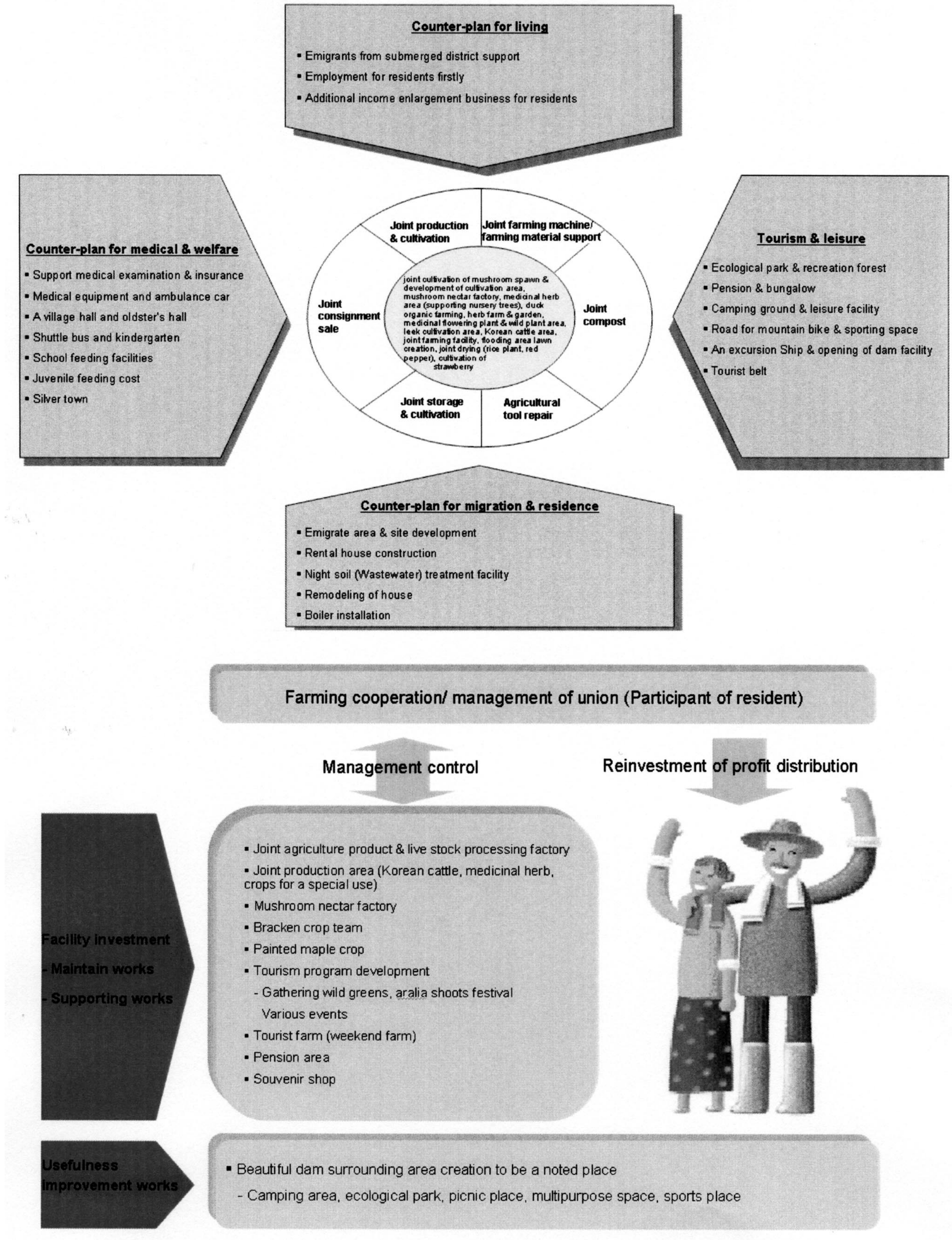


Fig. 6 Scheme of the program for resident income and welfare increment

Chungju dam is located in Jecheon, Chungchungbuk-do (province). It succeeded in Jecheon to turn submerged area into tourism resources. It is evaluated as successful project to make social problems caused by dam construction as local tourism resource through relocation and restoration of cultural assets in submerged area, building leisure facilities and running tour ship and contribute to develop local economy.

3. 3 Institutional devices for social consensus

When Government promote dam project as a large national project, "regional council" in which submerged residents, non-submerged residents, local government, Government and regional professionals partici-

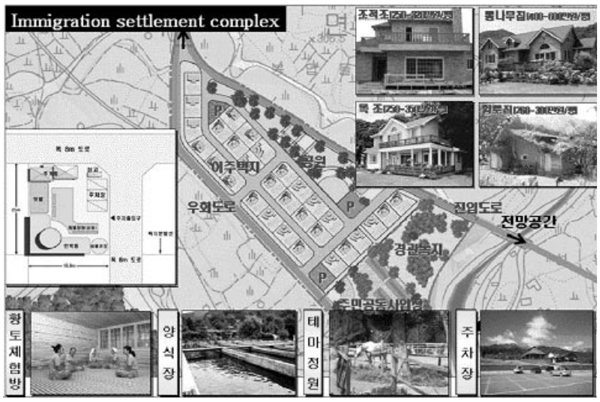


Fig 7. Example of immigration settlement complex

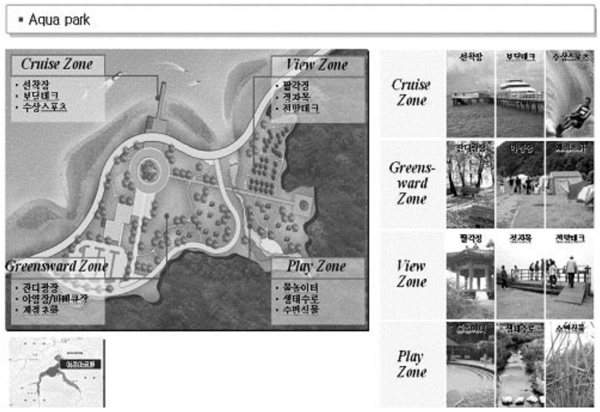


Fig 8. Example of project for dam utility increment

pate is running from the planning step for transparency and social agreement. The roles of "regional council" are shown below;

— Alleviation of social problems e.g. civil appeal and conflict between regions from planning step through institutionalization of opinion collection procedure.

— Enhancement of resident opinion collection procedure for transparency and objectivity to large nationwide project (briefing session for resident, public hearing, panel discussion)

"Dam surrounding area supporting project council" must be organized to support harmoniously dam surrounding area and the project enforce to give a part of revenue from water and electricity sale to the residents up to lifetime of the dam. Moreover, briefing session for the residents and public hearing with professionals must be opened when resident require them. Additionally, briefing for compensation and project meeting

Fig. 9 Observatory corridor and pond garden in the artificial wetland

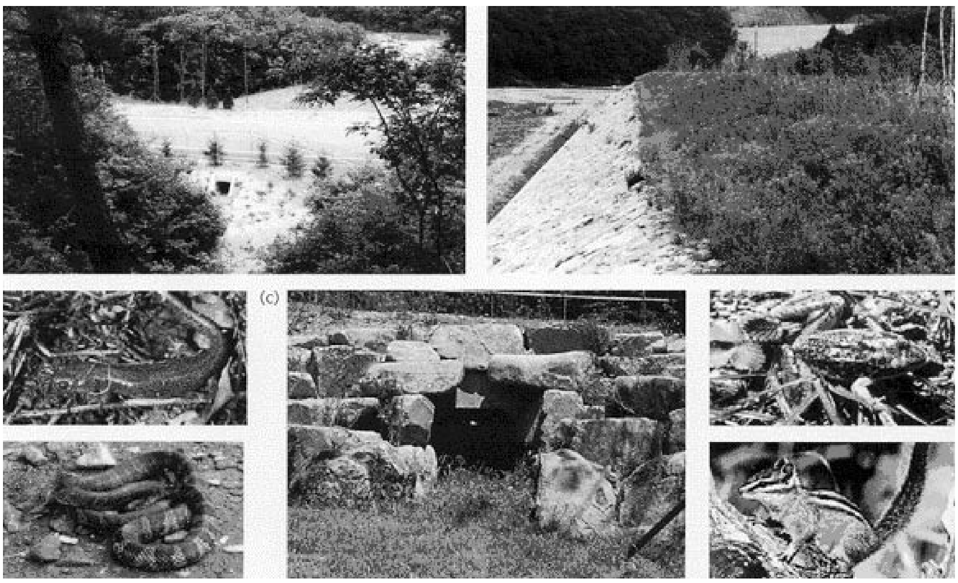


Fig. 10 Ecological passage under the access road

help people to comprehend matter of concern and interest for social consensus.

3. 4 Nature and ecologically friendly dam planning

In order to develop environment-friendly and sustainable dams, aggressive efforts are made to both analyze the impact on the environment and overcome the isolation of ecosystems. Utmost efforts are made to develop nature-friendly surroundings such as eco-corridors for animals and plants, habitats for animals and plants, biotopes, man-made floating islets, ecological nature parks, alternative environments and alternative habitats for otters. As these environment-friendly spaces are proactively created, they are naturally utilized as public rest areas as well as spaces for recreational activities available to local residents.

Recently built small- and medium-sized dams tend to be located at the upper stream. Due to the nature of topographical and geographical features, the downstream of rivers or valleys where these dams are situated are frequented by vacationers and campers. When an even water flow is maintained throughout the year, they may be enjoyed by a greater number of visitors. In consideration of these geographical characteristics, more recent small- and medium-scale dams are designed so that they do not directly collect household and industrial water within the dams. They collect water only after letting the water flow to a certain section in order to maintain the water level or to be used for recreational purposes.

This concept is an example of maximizing the utilization of water resources. Instead of using water once, it first uses water for the natural environment and then reuses it for industrial and domestic purposes. This is an environmentally friendly approach to operating a reservoir that was unimaginable in the past. It is expected to contribute significantly to the improvement of water quality and the ecological features of rivers.

4. CONCLUSION

The fact that survived species from the Genesis adapt to a change of environment is well-known. It is also a matter of common knowledge that only the positively and actively changed species can be dominant. Paradigm shift of the water resource development is not from a passive attitude but a positive trial along with the change of the times.

The period that the small was sacrificed for the large has already gone in water resource development. Political endeavor for clean and sufficient water resource and better environment is now realized all over the country and will go further. Because positive participation and productive plans from residents and NGOs is essential for it, lots of cooperation and rebuke are required.

From now on, water resources specialists will exert all possible efforts to develop water resource environment- friendly and welcomed to residents.

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Environmental Rehabilitation Plan of Hwabuk Multipurpose-Dam Area

Lee, Jiwon
Managing Director, Civil Engineering Team 2, Daewoo E&C, Seoul, Korea

Min, Kwonsik
General Manager, Civil Engineering Team 2, Daewoo E&C, Seoul, Korea

Kim, Changsi
Deputy General Manager, Civil Engineering Team 2, Daewoo E&C, Seoul, Korea

Abstract

The purpose of this thesis is to introduce the environmental rehabilitation plan of Hwabuk multipurpose-dam area which is under construction in Gunwi-gun, Gyeongsangbuk-do, Korea. The Hwabuk multi-purpose dam is a kind of Concrete Faced Rock-fill Dam (C.F.R.D.) and adopts the type of Concrete Faced Gravel Dam (C.F.G.D.) that mostly uses the aggregate and rocks on the upper and down stream of the dam site as the major embankment material.

In designing of dams, the concept was changed from the concept focused on irrigation and flood control functions in 70's to water-friendly and environment-friendly concept through 80's and 90's, and even the measures for local development are also considered now.

The environmental rehabilitation plan of Hwabuk multipurpose-dam contains an intention to make this area a noted place through environmental rehabilitation of the environs of this dam. The environmental rehabilitation plan largely consisted of three parts, i.e. environment-friendliness, scenery, and local economic activation, and the facility plan and programs of each part were suggested.

This plan was focused on minimization of environmental damage and recovery of ecological environment, and furthermore, on preparation of the foundation of local activation through maximum utilization of connectable potential resources with consideration of surrounding sceneries.

Environment-friendly rehabilitation plan was focused on optimization of preservation and recovery of ecological system by territory and positive utilization of environment-friendly materials and scenery plan consists of formation of water scenery belts, preparation of historical sceneries, and demonstration of night sceneries. To elevate life levels of near residents and to utilize tourism resources, territorial development was planned in connection with wide-area resources and the dam and surrounding facilities were to be utilized as tourism resources with development and application of the programs concerned.

Keywords: *Hwabuk multipurpose-dam, environmental rehabilitation, local development plan*

1. Introduction

Hwabuk multipurpose-dam is under construction in the area of Gunwi-gun, Gyeongsangbuk-do, Korea now. The purpose of construction of this dam is to stable supply with water to the middle area of Gyeongsangbuk-do, reduction of damage from floods, improvement of aged facilities surrounding the dam, and improvement of the life quality of local residents.

Construction order was placed by turn-key method by Korea Water Resources Corporation. The term of construction is about five years with starting from June 2004 and the Consortium of Daewoo E&C takes the charge of construction.

The major specifications of Hwabuk multipurpose dam are as follows:

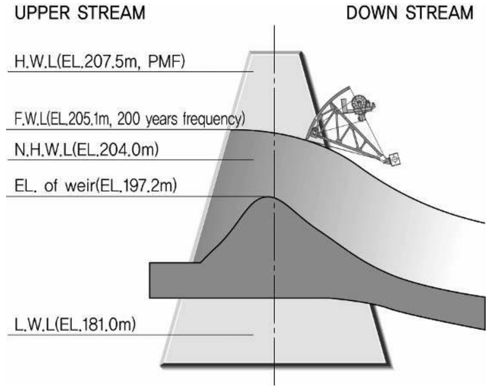
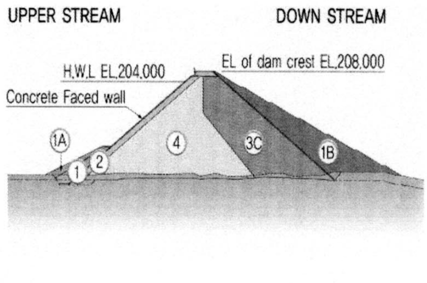


Fig. 1 Conditions of water levels of Hwabuk multipurpose-dam

Table 1 Major specifications of Hwabuk multipurpose-dam

Items	Specifications
Water supply	Municipal & industrial : 31.9 mil. ㎥/year
	Agricultural water : 2.6 mil. ㎥/year
	Water stream maintenance : 3.8 mil. ㎥/year
Capacity of reservoir	Total capacity of reservoir : 48.7 mil. ㎥/year
	Flood-control capacity : 3.1 mil. ㎥/year
	Available reservoir capacity : 40.1 mil. ㎥/year
Water levels of reservoir	Designed flood water level (F.W.L) : El.205.1m
	Normal high water level (N.H.W.L) : El.204.0m
	Low water level (L.W.L) : El.181.0m



Zone	Material
1	Impervious material
1A	Random material
2	Support material (Max. 75mm)
4	River aggregate (max. 500mm)
3C	River bed rock (Max. 800mm)
1B	Environment-friendly layer (top soil)

Fig. 2 Standard cross section and construction materials of Hwabuk multipurpose-dam

Hwabuk multipurpose-dam has app. 45 meters of height and app. 330 meters of length. This type is a kind of Concrete Faced Rock-fill Dam (C.F.R.D.) and the dame is designed to use the materials (aggregate and rocks) on the upper stream of the river as major construction materials. We defined this dam as Concrete Faced Gravel Dam (C.F.G.D.). The crossed section of the dam is shown in Fig. 2.

In the respect of dam design, dams were designed focused on the stability, economy, constructability, and maintainability of dams focused on the functions of irrigation and water control before 1970's but environmental concept was emphasized with providing with water-friendly spaces and with taking attention in natural scenery and ecological environment since 70's~80's. In recent concept of dam design, rehabilitation of dam area, promotion of life level of nearby residents, and activation of local economy were also considered for territorial development.

Such dam design concept was positively adopted in designing of Hwabuk multipurpose-dam and the materialization of 'a new noted place with the waves of nature and history' through activation of undeveloped local economy with preservation of natural scenery and historical culture surrounding the dam as well as elevation of the quality of dam design and construction technology, safety, and maintainability.

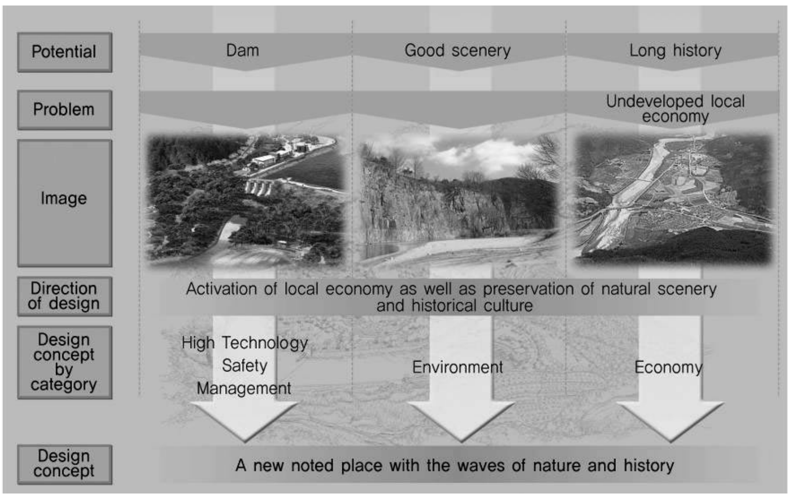


Fig. 3 Concept of design of Hwabuk multipurpose-dam

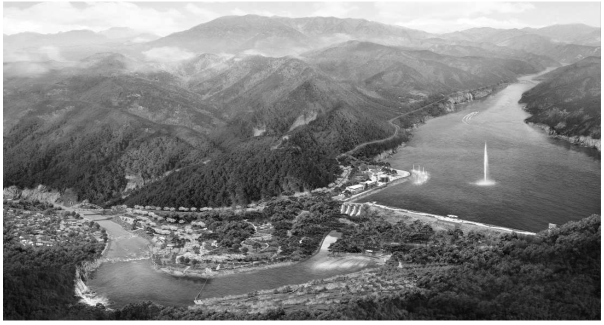


Fig. 4 View at downstream of dam

Fig. 5 View at upper stream of dam

Fig. 4 and Fig. 5 are the bird's-eye-views of the area surrounding Hwabuk multipurpose-dam that was designed with adoption of above-described concept.

2. Design concept of environmental rehabilitation

The design concept of environmental rehabilitation of Hwabuk multipurpose-dam was settled as 'Hwabuk, new noted scenery'. It was contained in the concept to make the area of Hwabuk dam new scenery through environmental rehabilitation.

The environmental rehabilitation plan largely consisted of three parts, i.e. environment-friendliness, scenery, and local economic improvement, and the design plan and programs of each part were suggested. Environment-friendliness was focused on the network of ecological environmental and the recovery of biological inhabitation environment; scenery part was focused on the demonstration of impressive noted scenery through water scenery and local historical scenery; in case of local economic improvement, it was intended to provide with local activation foundation through activation of green tourism based on the occupations of residents and through making dam area a tourism resource by holding various festivals and cultural events.

3. Space design

The environmental rehabilitation plan of Hwabuk dam area was settled so as to contain a wide region including dam facilities, upper stream of dam, and downstream of dam. In addition, it was intended to maximize the positive effect of dam construction on local societies by suggesting long-term and integrated master plan.

The conditions of development were reviewed in diverse points based on local characteristics, use of nearby land, analyses of cultural environment including distribution of nearby tourism resources, topography, plants, soil, hydrology, and biological haunt. The space design intended to utilize existing potential power in maximum and to minimize the adverse influence of development.

As shown in Fig. 6, it was intended to maintain fruitful plant scenery by preserving existing good forests in maximum and by connecting the unconnected forest. Observation decks were installed at major observation points and waterside park and ecological park were provided at downstream of the dam for possible activities of experiences. Existing fruit garden zones were used as agricultural experience place and Dungdungyi Village at downstream of the dam was revitalized to be used as the infrastructure of green tourism.

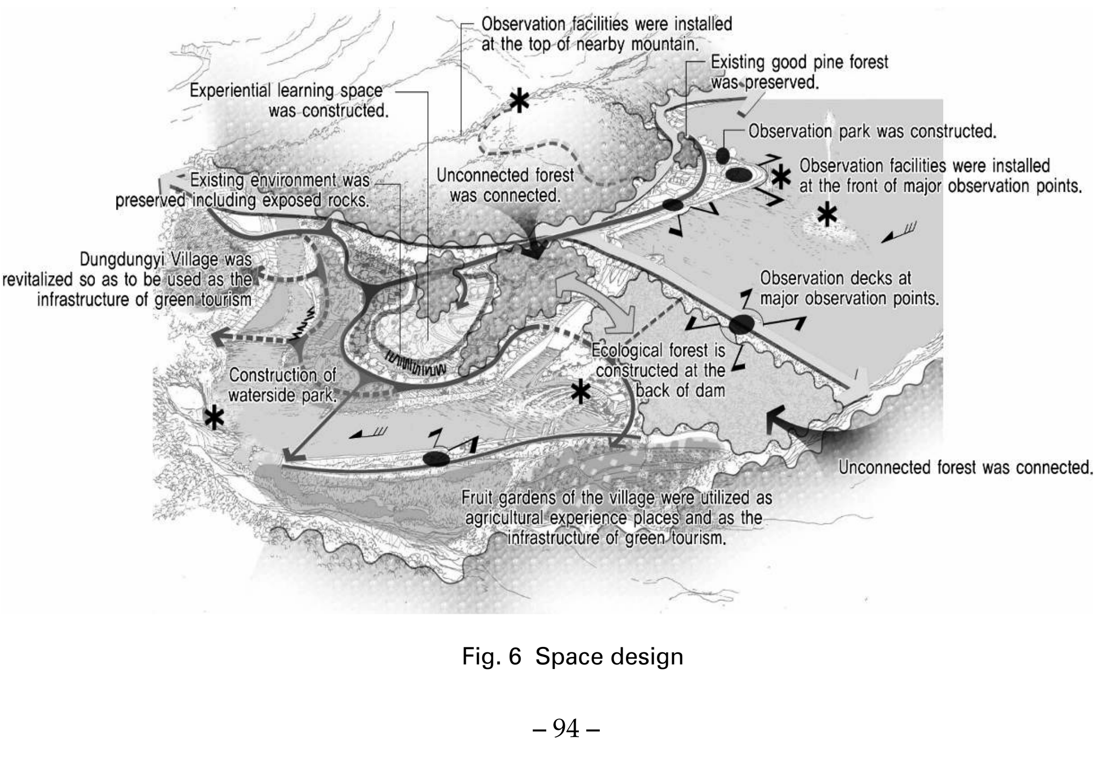


Fig. 6 Space design

4. Environment-friendly plan: preservation of natural environment for breathing of life

1) Target and direction of environment-friendly plan

Environment-friendly plan has the target to construct the durable environment of dam through recovery of ecology and environment-friendly technique and through application of the best management practices after construction of dam.

Ecological recovery technique consists of tree-planting at alternative haunt, construction aquatic biological haunt environment, and construction of biological network and preservation method with the best management practices consists of tree-planting at the slope with changing water level and the management of downstream plants.

2) Preservation of ecological system and planning of the best management practices (BMP's)

Water-cleaning plants were planted on waterside of damp ground of the upper stream of dam as shown in Fig. 7, and rock stacks and clothes racks were installed to provide various organisms with inhabitation environment. Appropriate level of water quality is to be maintained by preventing inflow of soils and sands and by precipitating of floating matters on the bottom and fishway was constructed with installation of fishway-type artificial dam. In case of downstream of dam, damage on plants is expected at flood due to contracted cross section of waterway by dense plants with changing water level. Therefore, a management of the structure of river is needed and recovery of biological haunt was planned through the recovery of plants and improvement of river environment (Fig. 8).

In case of drawdown zone with changing water level where corrosion is continued, corrosion was prevented with recovery of plants at edges and extreme cut of connection with nearby edges of forests was relieved.

As the kind of plants to be recovered at the slope with changing water lever, perennial herbs were used so as to survive for a while during soaking and to stand against exposure to soaking. In case of gentle slopes, water-attractant and drought-resistant trees (alder trees, willows, poplars, and Taxodium distichum) were mostly planted and, in case of steep slopes, perennial wild grasses and shrubs with well-developed rootstocks were planted to prevent loss of soils and sands (Fig. 9 and Fig. 10).

3) Application of ecology system recovery technique

The foundation of inhabitation for various organisms is supplied by providing alternative haunt against loss of haunt for organisms in socked places and sudden change in ecological environment is relieved in case of dryness of lake-edges during dry season.

Bird haunt was planned in planning of haunt environment in consideration of not only the organisms near to the lake but also seasonal birds.

Major alternative haunt facilities are as shown in Table 2 and the concept drawing of major haunts is as shown in Fig. 11, 12, and 13.

Green network was formed to connect left and right forests by constructing ecology corridor at surrounding area of main dam and ecological pipes were constructed in consideration the moving of organisms (Fig. 14).

Elk pools and natural mountain streams are prepared with utilization of existing hydrosphere and

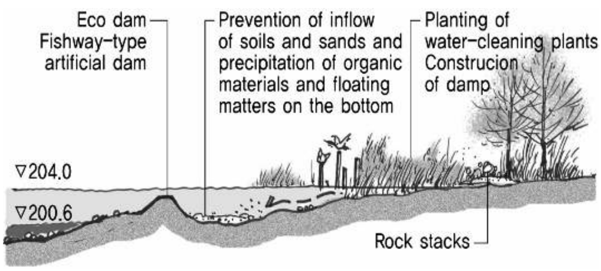


Fig. 7 Preparation of upper stream damp

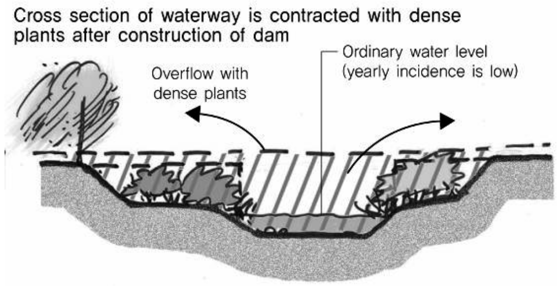


Fig. 8 Cross section of downstream after construction of dam

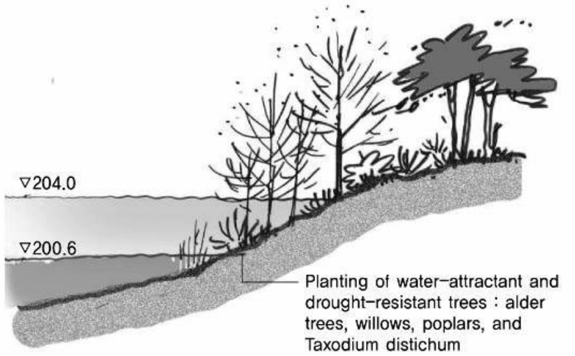


Fig. 9 Cross section of gentle slope

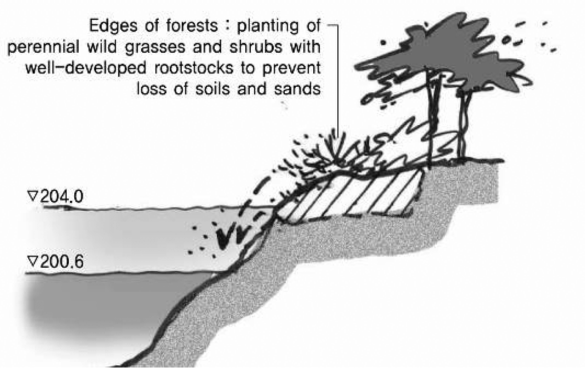


Fig. 10 Cross section of steep section

Table 2 Alternative haunt facilities

Kinds	Major facilities
Alternative waterfowl haunt #1	Eco dam, fishway, and clothes racks.
Alternative waterfowl haunt #2	Fishway-type artificial dam and gravelly field
Artificial damp	Bird observation platform, stay wall, and clothes racks.
Alternative otter haunt	Eco dam, log stacks, and rock stacks.

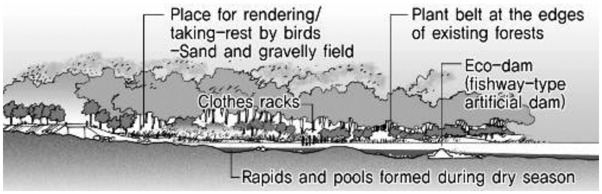


Fig. 11 Cross section of alternative waterfowl haunt

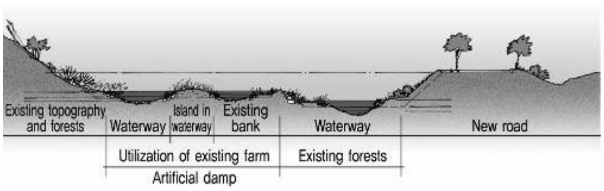


Fig. 12 Cross section of artificial damp

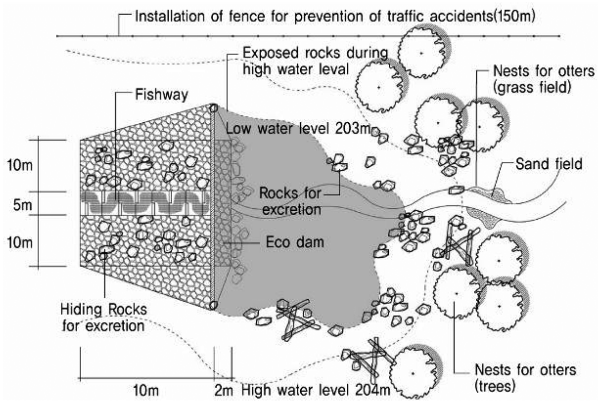


Fig. 13 Alternative otter haunt

damp haunt is prepared so as to be used as the haunts for birds, amphibians, and fishes.

In recovery of plants at back of dam, the concept of forest design was adopted for continuance of plant scenery at the left/right shores and the back of dam body. Available soil depth was prepared with use of top soil and various biotopes were formed at edges with plants, grasses, pine forests, and oriental oaks (Fig. 15).

4) Formation of ecological network

The influence on ecology system could be minimized through the connection with land ecology system, which was unconnected with existing roads and construction of dam, by constructing ecological bridges. Bushes were planted to provide with foods and the place for hiding places and fence was installed to for temptation and protection of wild animals (Fig. 16). It was planned to install reinforced spun concrete pipes and boxes for moving of small organisms such as amphibians and reptiles at low part of the road constructed.

Places for spawning and hiding by fishes are provided at downstream of dam and rapids-type fishway was constructed with utilization of existing reservoir dam (Fig. 17).

5) Utilization of environment-friendly materials

Harmonization with existing environment was made and the value of natural resources was elevated through application of environment-friendly materials and utilization of by-products produced during construction. Major measures for utilization of environ-

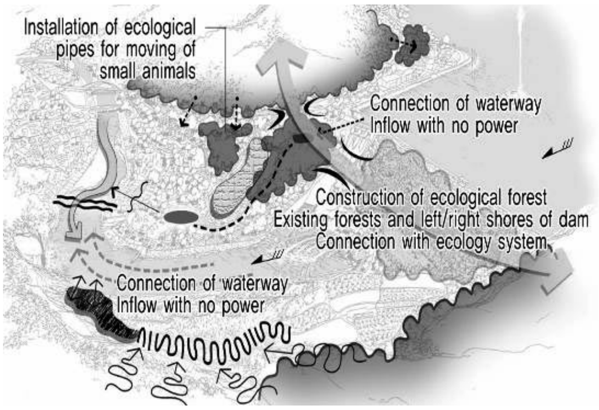


Fig. 14 Ecology recovery plan of dam area

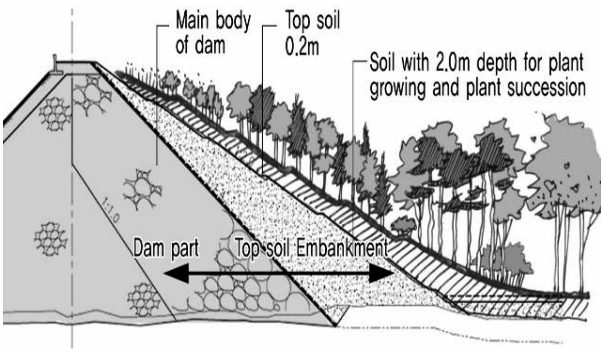


Fig. 15 Cross section of recovered plants at theback of dam

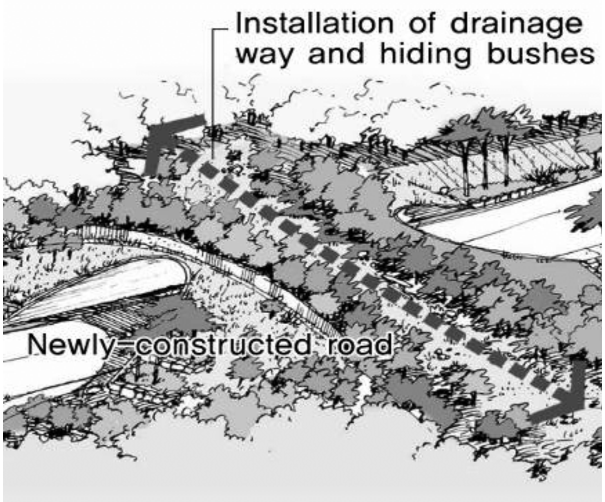


Fig. 16 Plane section and cross section of ecological bridge

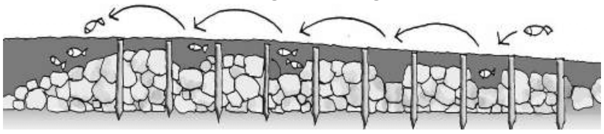


Fig. 17 Cross section of rapids-type fishway

ment-friendly materials are as shown in Table 3.

5. Plan of scenery: Making a beautiful place for visiting

1) Target and concept of scenery plan

The scenery plan of Hwabuk multipurpose-dam largely consists of three categories: formation of waterside scenery belt, revival of historic scenery, and demonstration of noted scenery. Waterside scenery belt is formed along to the waterway with use of the various shapes of 'Water' that is the major element of scenery at the site. Display of historical scenery is made by making the spaces to reflect the characteristics of history and culture of the territory with preparation of memorial spaces for the residents on submerged places and by adoption of the facilities containing such motives and construction of noted

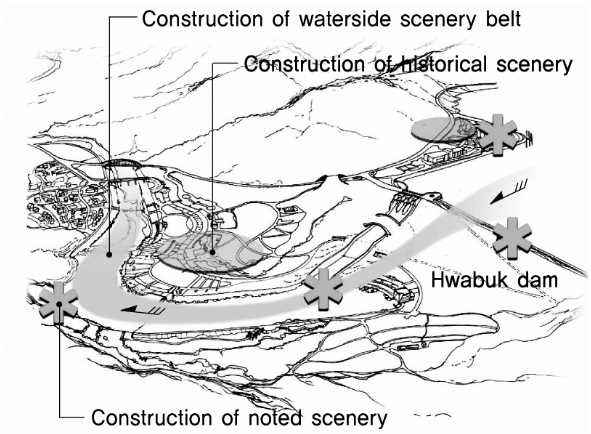


Fig. 18 Basic concept of scenery plan

scenery is providing with impressive places containing the characteristics of the site.

2) Formation of waterside scenery belt

Waterside scenery belt means the water scenery that is continuously demonstrated according to the change in space.

To provide with sights at major observation points, the elements of water scenery adopted at visual focus part are demonstrated in various shapes according to the characteristics of each space so as to provide with dynamic and changing scenery experience.

Natural harmonization is made with surrounding scenery by adopting environment-friendly water scenery (formation of ecological damp with utilization of naturally inflowing rain water) with utilization of surround topographical characteristics, and no-power technique was adopted with use of water level difference so as to elevate the efficiency of maintenance. The elements of demonstration of various water scenery adopted in this plan are as follows:

3) Demonstration of historical scenery

Demonstration of historical scenery is made through modern interpretation and revival of historical

Table 3 Utilization of environment-friendly materials

Materials	Measures for utilization
Pavement material	Use of water-permeable pavement material
Pavement drainage	Pavement drainage utilizing grass blocks
Utilization of top soil	Providing with available soil depth with use of top soil and preparation of foundation for healthy growth of plants
Utilization of collected stones at site	Preparation of inhabitation/hiding by small mammals, amphibians, and Reptiles - Stone stacks and heaps of stones Utilization of parapet wall at dam crest
Utilization of cut logs	Preparation of inhabitation/hiding by huge mammals, amphibians, and reptiles - Wood grave and placing of logs Utilization as the material for various guide boards (district and direction)
Utilization of existing	Utilization of good plants and stumps in submerged place for stable and early settlement of trees at the back of dam

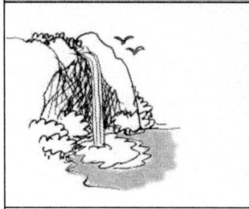
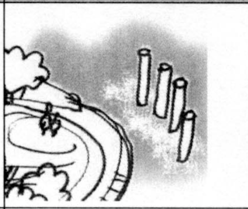
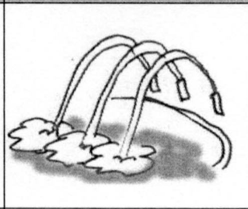
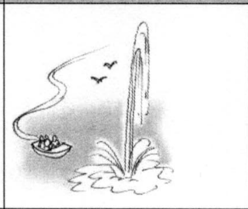
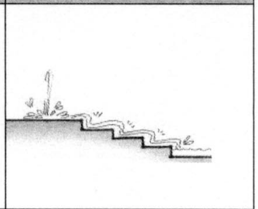
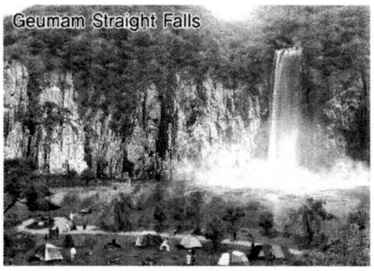
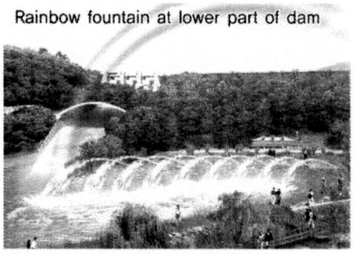
Pouring water	Ascending water	Dropping water	Springing water	Flowing water
				
Waterside park	Observation park	Lower part of dam	Hwabuk Lake	Water environment amusement place

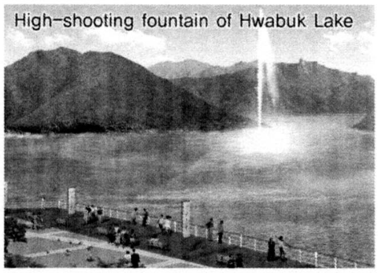
Fig. 19 Elements for demonstration of water scenery



60m-height falls using existing bluffs: utilization as the landmark of waterside park



No-power fountain using water level difference : improvement of the scenery of projected parts with use of spillway



The high-shooting fountain at the center of the lake : formation of visual focus at various observation points

Fig. 20 Examples of water scenery facilities

remains of the area.

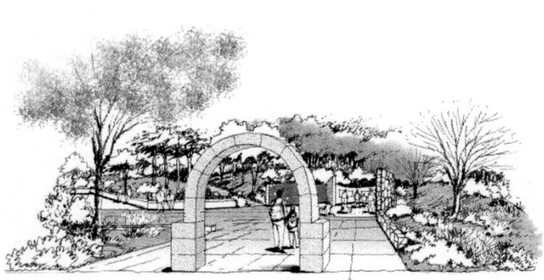
It was intended to improve the understanding of the area by providing learning spaces with the theme of local history and to provide with pride and a space for excitement of nostalgia was provided with use of trees and stones that contain the memories of submerged places.

Friendly scenery was demonstrated with local scenery by applying C.I. plan and facilities with the motives of local characteristics such as Wicheon and Hwasansanseong.

4) Demonstration of night sceneries

The theme of demonstration of night sceneries in Hwabuk dam is 'Moonlight Park of Hwabuk, a new noted place' and was focused on the demonstration of lightings that coexists with nature.

Strange lighting demonstration elements, which are harmonized with the characteristics of each space, were adopted for the experience in various night sceneries and so as to give vitality to the night sceneries.



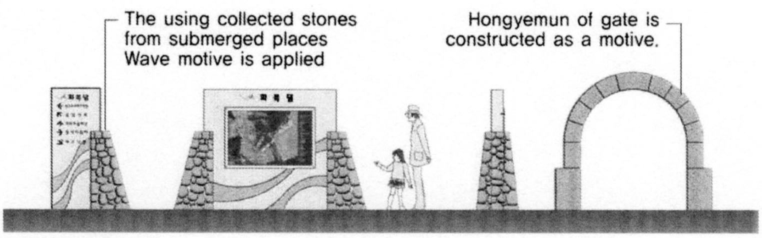


Fig. 21 Facility design with reflection of local characteristics

6. Local economy improvement plan: Making a vital village with residents together

1) Target and direction

This plan has the target to utilize water resources and waterside scenery as the resources for local economy improvement with the motive of construction of dam, to provide an environment in which nature and facilities are harmonized through development of environment ecology, and to motivate local economy by making noted scenery for tourism with fruitful sights and amusement subjects.

For these, it was intended to provide with more concrete and practical local development measures by providing with resources with use of potential beautiful sceneries and native cultural elements on dam area and by suggesting the programs, business plan, and marketing strategies for connection of wide-area tourism resources.

2) Territorial development plan

Centered on the site of this plan, the south-north axis forms Confucianism and Buddhism and territory and the east-west axis forms tourism territory between inland and sea; therefore, a local characteristic is formed for possible construction of differentiated theme tourism route with connection with the axes of the large city.

Differentiated directions of plan was made for possible mutual connection and supplementation based on territorial potential, and practical development plan is suggested with examples of facilities, program plan, and subject of business (Table 4 and Fig. 22).

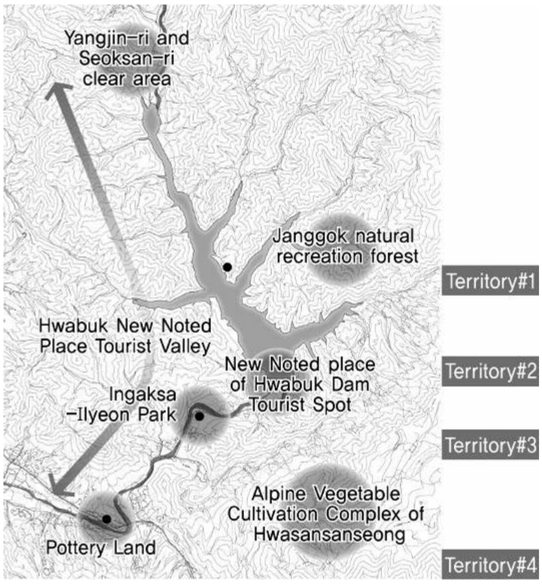


Fig. 22 Plan of territorial development

Table 4 Territorial development plan

Classification		Potential	Direction of construction	Tourism resource facilities	Subjects of business
Upper Stream	Territory #1	·beautiful natural scenery · image of clear territory · recognition level of recreation Forest	·environment-friendly agricultural complex · natural recreation place ·waterside recreation territory	·experience program facilities in recreation forest and marine leisure facilities	Korea Water Resource Corporation and local council
Downstream	Territory #2	·a noted place with harmonization of natural scenery and lake scenery ·agricultural village near to dam facilities	·tourist complex harmonized with the sceneries of noted places · pension for experience in agricultural culture	· theme facilities of water scenery hall · facilities for experience in natural ecology learn · facilities for experience in, exchange of culture	Korea Water Resource Corporation and local resident council
	Territory #3	· beautiful historical scenery resource such as the place of writing of Samgukyusa, Ingaksa, and etc.	·Buddhist culture and national history theme park	·Samguk History Hall ·Temple Experience Hall ·Ilyeon Academic and Cultural Festival ·light festival around the Buddha's Birthday	Korea Water Resource Corporation, Ingaksa (Jogyejong), and local resident council
	Territory #4	· formation of new housing culture place with new settlers and the place of cultivation of local characteristic	·tourism an lodging complex for experience in local culture, history ·stronghold of green tourism	·pension complex ·Pottery Land ·Experience in Alpine Vegetable Cultivation Land	Korea Water Resource Corporation, local residence council, and farmers of products

Fig. 23 Branding of tourism resources

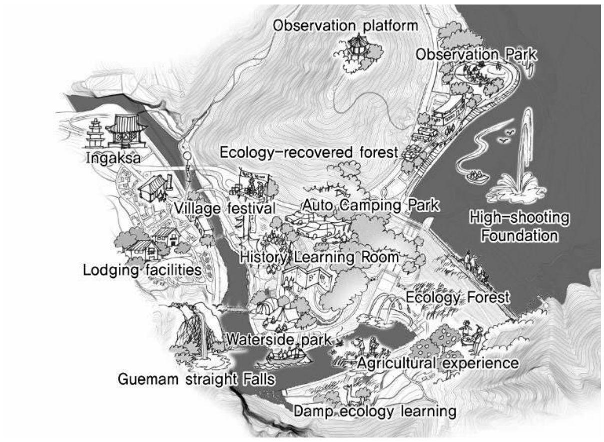


Fig. 24 Hwabuk Green Map

3) Plan for improvement of life environment and utilization of agricultural production infrastructure

Settler resettlement plan has the target to provide housing environment for and rapid life stabilization of residents that are the subjects of soaking due to construction of dam. It was intended to induce positive expected effect of development business and to provide with stable life environment infrastructure through induction of township offices, police stations, and post offices and through providing with fresh and environment-friendly agricultural communities.

The purposes of preparation of environment-friendly agricultural complex art to motivate the elevation of price rise of local agricultural products through execution and brand of environment-friendly agriculture, to induce direct trade between village residents and urban residents with utilization of direct sales shop of agricultural special products, to expand urban-rural exchange, and to provide with stable sales routes of agricultural special products of the village.

Therefore, it was intended to induce smooth technique exchange among residents and to meet the purchase willingness of visitors through adoption of corresponding facilities with selection of differentiated special produces by each agricultural territory.

Table 5 Urban-rural exchange program

Items	Programs
Sales of environment-friendly agricultural products	· Quality improvement of agricultural products with adoption of organic agricultural method · Construction of agricultural product processing and development of processed special products · Resident training, PR, and sales
Operation of direct sales shop of agricultural/special products	· Development of designs such as trademark and vessels of village · Operation of direct sales shop of agricultural products and sales to visitors · Reservation and sales of agricultural products through homepage
Development and sales of Mementoes and native foods	· Development and sales of products and mementoes of the village · Development of specific native foods of the village · Location for agricultural product processing and selling facilities
Operation of private lodging, pension businesses by residents	· Management/Distribution method by joint reservation system · Maintenance of village image and unification and challenge for differentiation of village · Lecture by invited professionals and training on resident services

Table 6 Plan for development and marketing of tourism products

Development of attractive tourism products with utilization of territorial resources	Development of participation-oriented products to meet the patterns of tourists	Development of characters and tourism mementoes that meet the identity of each territory
· Connection of new facilities and surrounding waterside/mountain resources · Development of tourism products to meet one-day tour and stay-tour	· Development of experience and participation tour products → induction of visiting by urban residents · Utilization of waterside resources → Direct participation in nature experience including exploration and inquiry, festivals, and events by tourists	· Development of products with harmonization with modern design senses so as to be used as tour mementoes with utilization of territorial resources. · Participation of marketing/design professionals

Table 7 Plan for stepwise driving of tourism product business

Name of business	Driving step			Subjects driving the plan		
	1	2	3	Administrative authorities	Private	Local residents
Resident training	●			□		○
Development of the qualities of agricultural products		●		○		□
Operation of direct sales shop		●			○	□
Development and sales of mementoes			●		□	○
Development of native foods		●			○	□
Support of private business			●	□		○

Note: 1. Driving subjects - □ Management, ○ Support & cooperation

4) Tourism resource plan

The programs for interesting and efficient induction of experience were planned with classification of tourism resources dispersed in the area and with branding of them by theme (Fig. 23).

Tour products focused in experience were organized so as to extend staying time of tourists and to induce revisiting through development of tour products linked to tourism resources in planned sites and Gunwi-gun and tour courses were designed in consideration of tour patterns sought for by sex and age group with use of the subjects of near urban residents that are the largest background market.

Facilities for adoption were selected focused in participation/experience type facilities with large temptation effect of tourists in adopting themes and selecting facilities with utilization of scenery resources in the dam construction area.

Major theme courses are 'noted tourism facility course', 'natural ecology experience and learning course', and 'local culture experience course'.

In addition, Hwabuk Green Map, as shown in Fig. 24, was prepared with containing of major particulars such as courses, nature, history, and environment so as to be used as PR material.

5) Local community development program plan and marketing plan

The contents shown in Table 5 are the suggested programs to create occupational opportunities with participation of residents in management and operation of parks in dam area facilities and to elevate the efficiency of operation through assigned management of special produce sales facilities and theme facilities to local residents.

Plan for development and marketing of tourism products in dam area is as shown in Table 6.

It was planned to hold events and social meetings for PR of the village, to drive PR activities so as to utilize as agricultural experience places with invitation of student users, and to provide with marketing methods through homepage production and PR materials, as the invitation strategies of visitors.

7. Conclusion and suggestions

Dam design, which took attention on the functions of 'irrigation and water-control', treats environmental characters of 'water-friendliness and environment-friendliness' as important points of arguments after entering into 1980's and 1990's. Such concept of dam design formed a sympathy belt on the important task named 'local development' caused by development of dams with start of consideration of various aspects after entering into 2000's.

Such change in the paradigm in dam design becomes an important motive for improvement of the image of dam environment as the facilities that are coexisting with the area and are durable through offsetting of negative recognition of dams that is unavoidably constructed and through improvement of expected effect with development.

The plan of Hwabuk multipurpose-dam, which is planned and prepared in the time of such change, was started with various aspects of problems to be solved in relation to functional characters, environmental characters, and local economy improvement.

The area of Goro-myeon, Gunwi-gun, Gyeongbuk, which was the candidate site, was a traditional agricultural village that could not escape from undeveloped situation due to wrong approach and weak infrastructure in spite of beautiful surrounding sceneries and fruitful potential resources that could be connected.

Therefore, this plan was progressed focused on preparation of foundation for local activation through maximum seeking for resource elements in dam area and through making them programs with minimization of environmental damage caused by development.

The land receiving damage from development was to be recovered to stable environmental infrastructure through stepwise development and alternative techniques and the scenic and cultural potential contained in the candidate land was supplemented and reorganized in the aspect of value elevation. In addition, for the space design for local activation, various operation and management programs were suggested to improve practical sense and made the business and continuous maintenance easy.

We want that Hwabuk multipurpose-dam construction, which is being executed with target completion in 2009, is continued as a long-term development plan to meet the requirements by residents as well as rehabilitation of surrounding environment and wants that it is recorded as a model example in the time of change in dam design and construction.

Effective Utilization of Existing Dams in Japan

Tsuneo Uesaka
Vice-President,
Japan Dam Engineering Center, Japan

Fumio Yonezaki
Director, Second Engineering Dept.,
Japan Dam Engineering Center, Japan

Takashi Sasaki
Senior Researcher, Hydraulic Engineering Research Group,
Public Works Research Institute, Japan

Abstract

To satisfy future demand for flood control and water supply, effective utilization of existing dams as important social capital stocks is sought for in Japan as the nation's investment capability has reduced with the advent of the aging society and the social demand for the preservation of natural environment has been escalating. The Japan Commission on Large Dams (JCOLD) established the Committee on Effective Utilization of Existing Dams and carried out various activities between 2003 and 2004. With the aim of contributing to the forward planning and implementing effective dam utilization, the committee summarized the subjects to the achievement of its goals by investigating the effective operation and improvement measures for dams and putting together precedents for existing facilities to enhance their functions. This paper outlines the committee's research results.

Keywords: *Existing dams, Effective utilization, Questionnaire survey*

1. INTRODUCTION

In order to satisfy future demand for flood control and water supply in Japan,, there has been an escalation in demand for the effective utilization of existing dams as vital social capital stocks as the nation's investment capability has reduced with the advent of the aging society and the social demand for the preservation of natural environment has been escalating. Thus, in addition to the improvement of dam management and the cooperative operation of multiple reservoirs, the purposes and methods of utilization of storage capacity have changed and, in more and more cases, dams are reorganized into a new system.

From the perspectives of physical technology, the technical subjects necessary for raising the height of dams, building additional structures and making modifications to intake and outlet facilities have risen in importance. And the same time, there have been advances in the field of research, design and construction that have greatly enhanced the techniques of dam heightening, remodeling and enlarging of outlet structures. To satisfy the future demand for flood control and water supply, requirements for non-physical innovations concerning the modification of operating methods and the reorganization of dam groups seem to be imposed more and more.

In this light, the Japan Commission on Large Dams (JCOLD) established the Committee on Effective Utilization of Existing Dams and carried out its activities for two years. With the aim of achieving its goals and thereby contributing to the forward planning and implementation of effective dam utilization, the committee summarized subjects by investigating effective operation and improvement measures for dams and putting together precedents for existing facilities to enhance their functions.

This paper introduces the ways in which existing dams have been utilized effectively in Japan, demonstrating 227 cases collected and analyzed by the committee and their trends according to time period and purpose. This paper represents only a part of total research results obtained by the committee and its final report will be issued later this year[1].

2. TRENDS OF RESEARCH CASES

2.1. Measure used in the case study

Besides such general procedures as the enlargement of capacity by means of dam heightening and the reallocation of capacity, an attempt was made in this research to gather a number of informative cases by broadly defining effective dam use, including those projects for increasing downstream maintenance flow, building fish ladders and adding sediment management functions as environmental protection measures. The dams investigated in this research include the following: those under the jurisdiction of the Ministry of Land, Infrastructure and Transport and the Ministry of Agriculture, Forestry and Fisheries; those administered by private electric power companies; and those managed by public power districts.

A questionnaire was sent out to each organization to elicit the following information:

1) Examples of the effective utilization of existing dams through operational improvement and rehabilitation including redevelopment
2) Technical descriptions regarding the remodeling and reinforcement of relevant dam facilities
3) Adjustments of cost sharing and property rights among the owners of existing dams and the organizations undertaking their improvement and redevelopment

As a result, information on a total number of 227 cases was collected.

2.2. Case Types

The cases of the effective utilization of existing dams were classified with two codes: a case code (project category for effective utilization) and a technical code (technical method to realize effective utilization).

Both the case code and technical code are summarized below.

2.2.1. Case code

The case code is a classification that focuses on the category of projects for the effective utilization of existing dams. Table 1 presents a list of the case codes used. Because, in some examples, one case may correspond to several case codes, the total number of cases given in the summary table for case codes (Table 3) is not necessarily equal to the total number of the cases examined.

2.2.2. Technical code

The technical code is a classification that focuses on the type of engineering methods for the effective utilization of existing dams. Table 2 is a list of the technical codes used. Because there are some cases in which physical techniques are not employed, the total

Table 1 Case codes

Case code	Description of effective dam utilization
1-(1)	A case in which a dam operation scheme is modified in response to a change in a water supply plan to meet social demand (single dam)
1-(1)-1	A case in which storage capacity is expanded by means of raising the height of an existing dam, constructing a new dam in a downstream site which submerges an upstream existing facility, or dredging/excavating the sediment in a reservoir.
1-(1)-2	A case in which a dam operation scheme is changed by cutting back generation hours while at the same time increasing the maximum output for a hydropower plant to produce on-peak energy.
1-(2)	A case in which a new dam is constructed and the efficiency of both the old and new dams in the same river system is enhanced by modifying the operating plan of the existing dam.
1-(3)	A case in which integral operation is practiced on a group of dams to improve the efficiency of individual dam operation (a group of dams in the same river system).
1-(3)-1	A case in which upstream dam groups are orderly supplemented so that dead outflow can be reduced and storage efficiency can be enhanced in the total dam system.
1-(3)-2	A case in which hydropower generation is efficiently operated or a power plant is newly constructed in a river system after reviewing a generation plan on the occasion of the construction of a new dam-type power plant.
1-(4)	A case in which water service to benefited areas is reallocated among several dams.
1-(5)	A case in which an integrated dam operation is carried out with another river basin beyond a given river system.
1-(6)	A case in which existing dam facilities are effectively utilized as in an increase in the capability of outlet structures, eliminating gates, modifying flood control rules and implementing sedimentation measures.
1-(6)-1	A case in which water supply operation is efficiently and simply conducted in upstream dams through the effective use of mid-to-downstream regulating reservoirs.
1-(6)-2	A case in which an efficient dam operation is realized by adding a new control channel (integrated operation in a dam group)
1-(6)-3	A case in which flood control and water supply functions are reallocated and made efficient within a river system or conjoined with another river system (reorganization in a dam group)
1-(6)-4	A case in which an existing dam is utilized effectively by adding a power plant that includes low power generation using maintenance flow or by increasing output.
1-(6)-5	A case in which an existing dam is effectively utilized by enhancing the maintenance flow to improve the river environment, by building a surface/selective intake facility or by taking various sediment control measures such as transporting sediment down a river.
1-(6)-6	A case in which an outlet structure and a bulb are constructed and/or added to supply water.
1-(6)-7	A case in which an outlet structure to control flooding is added, in which gates are eliminated or in which an emergency flood spillway is remodeled so that the dam management situation is improved.
1-(6)-8	A case in which flood control rules are modified to respond to the improvement status of downstream river channels or in which water service operations are revamped and the purposes of the water supply is converted to align with the changes in social situations.
1-(6)-9	A case in which the construction of a check dam and a bypass as well as dredging works are carried out to prevent sedimentation in a reservoir.
1-(6)-10	Other than above.

Table 2 Technical codes

Technical code	Description of effective dam utilization
2-(1)	A method in which dam capacity is increased by raising the height of a dam or constructing a new dam which submerges an existing dam and its reservoir.
2-(2)	A method in which both the intake and outlet capacities are increased to comply with changes in an operating plan.
2-(3)	A method in which outlet capability (reliability and minute adjustability) is increased for compliance with changes in an operating plan.
2-(4)	A method in which selective intake capability is increased, including such measures for cold or turbid water.
2-(5)	A method in which a reservoir is equipped with a sediment removal function (which includes a downstream sediment supply as an environmental strategy).
2-(6)	A method in which a fish ladder is added.
2-(7)	Drilling and cutting of a dam body
2-(8)	Construction of connecting tunnels between river channels or dam reservoirs
2-(9)	Addition of power generators and the like
2-(10)	Other than above.

Table 3 Breakdown of effective dam utilization (case code)

Dam type / Case code	Multipurpose dam	Agricultural dam	Hydroelectric dam	Total
1-(1)-1	38	8	3	49
1-(1)-2	0	0	4	4
1-(2)	13	2	7	22
1-(3)-1	1	5	0	6
1-(3)-2	0	0	3	3
1-(4)	3	0	0	3
1-(5)	5	0	0	5
1-(6)-1	1	0	0	1
1-(6)-2	3	1	0	4
1-(6)-3	2	0	0	2
1-(6)-4	4	3	20	27
1-(6)-5	26	3	29	58
1-(6)-6	6	1	0	7
1-(6)-7	9	4	7	20
1-(6)-8	3	5	0	8
1-(6)-9	11	2	6	19
1-(6)-10	0	2	0	2
Total	125	36	79	240

Table 4 Breakdown of effective dam utilization (technical code)

Dam type / Technical code	Multipurpose dam	Agricultural dam	Hydroelectric dam	Total
2-(1)	28	8	2	38
2-(2)	24	2	18	44
2-(3)	4	2	7	13
2-(4)	15	2	6	23
2-(5)	15	1	1	17
2-(6)	0	0	3	3
2-(7)	10	0	5	15
2-(8)	7	0	1	8
2-(9)	5	3	22	30
2-(10)	2	1	0	3
Total	110	19	65	194

number of cases given in the summary table (Table 4) does not necessarily correspond to the number of cases examined or to the summary table for case codes.

2.3. Result of Case Classification

All cases of effective dam utilization classified by case code and technical code are illustrated in Tables 3 and 4 as well as Figures 1 and 2, in which dam functions are further divided into three purposes: multipurpose (flood control), agricultural and hydroelectric.

Regarding the case code, of all 240 cases, as many as 49 cases were classified into the 1-(1)-1 category and 58 cases into the 1-(6)-5 category, for which further information is provided in Figure 3. Among the 49 cases in the 1-(1)-1 category, capacity was enhanced by raising the dam height on the same dam axis in 23 cases while dam heightening was executed on the downstream dam axis in 14 cases. In six cases, the existing capacity was reallocated and dead storage

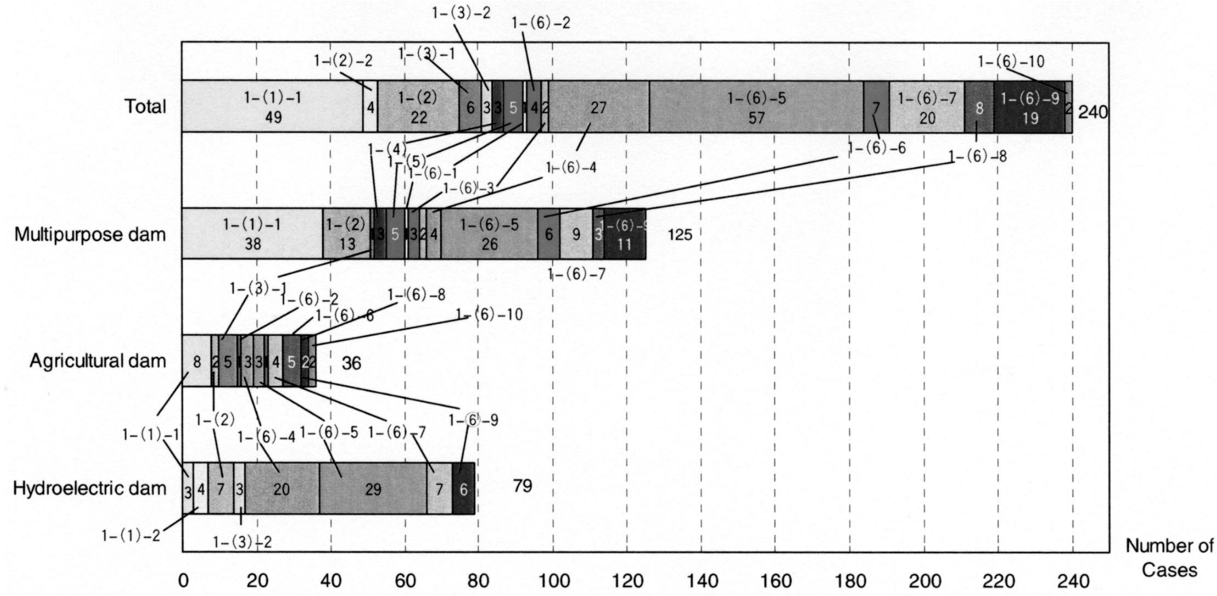


Figure 1 Breakdown of effective dam utilization (case code)

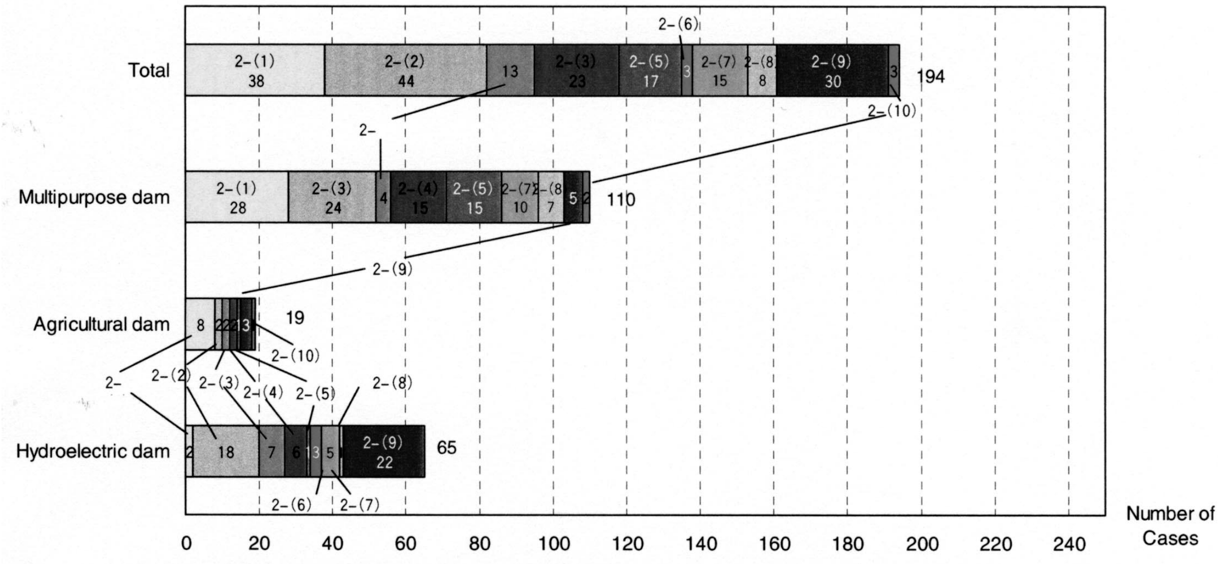


Figure 2 Breakdown of effective dam utilization (technical code)

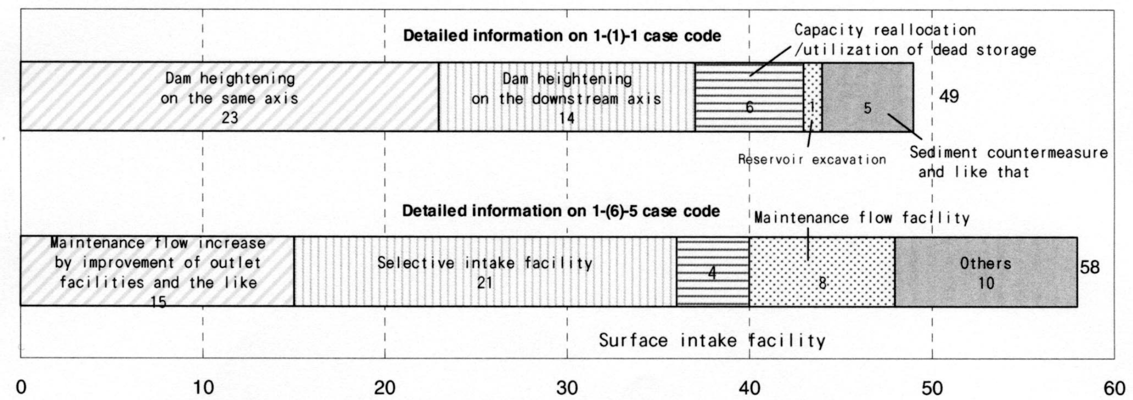


Figure 3 Detailed information on 1-(1)-1 and 1-(6)-5 case codes

was effectively utilized. Excavation within a reservoir was carried out in one case and new capacity was secured by controlling sedimentation etc. in five cases. Among the 58 cases in the 1-(6)-5 category, river maintenance flow was increased by improvement of outlet facilities and the like in 15 cases: a selective intake facility was constructed in 21 cases: a surface intake facility was installed in four cases: a maintenance flow facility was built in eight cases: and other effective measures were employed in ten cases. Multipurpose dams show a similar tendency to the whole. Within the total of 125 cases, 38 cases and 26 cases were grouped into the 1-(1)-1 and 1-(6)-5 categories, respectively. Within a total of 36 agricultural dams, eight cases in the 1-(1)-1 category and five cases in the 1-(3)-1 and 1-(6)-8 categories each outnumber the others. For hydroelectric dams, as many as 29 cases and 20 cases were grouped into the 1-(6)-5 and 1-(6)-4 categories, respectively.

Regarding technical code, of the total number of 194 cases, 38 cases in the 2-(1) category and 44 cases in 2-(2) category suggest their dominance. A similar tendency to the rest is shown in the cases of 111 multipurpose dams, with 28 cases in the 2-(1) category and 24 cases in the 2-(2) category. In 19 agricultural dams, category 2-(1) is often seen. As for hydroelectric dams, the categories of 2-(3) and 2-(4) in addition to 2-(2) and 2-(9) are characteristically often seen. This includes the expansion of intake/outlet facilities and the installation of selective intake structures, reflecting the movement to protect water quality and the river environment.

2.4. Transition in Effective Utilization of Existing Dams

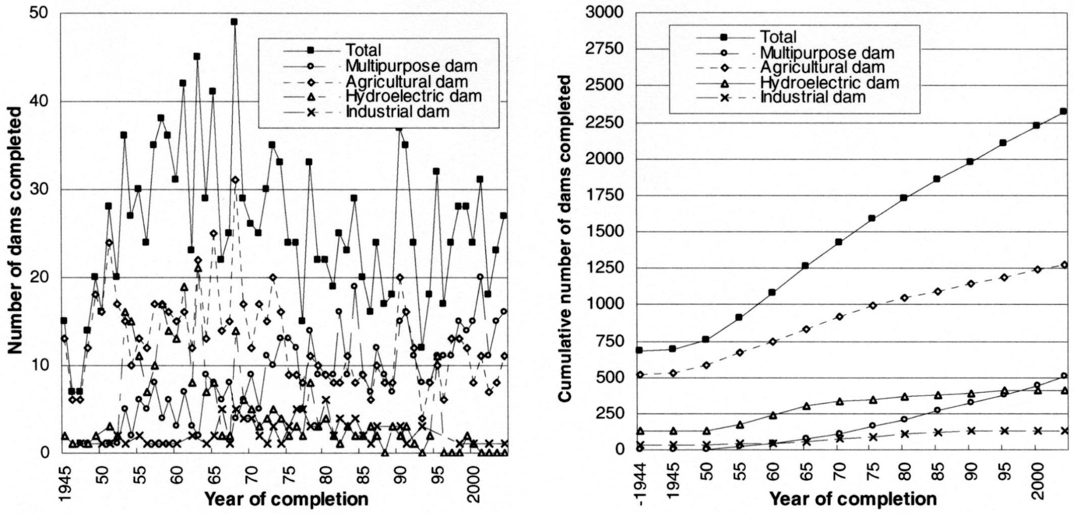
2.4.1. Changes in the number of completed dams

To examine the trends in effective dam utilization, changes in the number of completed dams are reviewed. Based on the database[2] provided by the Japan Dam Foundation, the changes in the total number of completed dams from 1945 to 2004 are itemized under several categories: all dams, multipurpose dams including flood control dams, agricultural dams, hydroelectric dams and industrial dams. Figures 4 (1) and (2) depict their yearly and accumulative data, respectively.

A dozen to a score of hydroelectric dams were built annually between 1953 and 1963. The number of dams constructed remained almost as stable at about five dams per year until 1996 with the exception of three years, 1968, 1978 and 1995 when 13 dams, 8 dams and 11 dams were constructed, respectively. After 1996, no additional new dams were constructed although three dams were built in 1999 and one in 2000. On the whole, there was a decline in the number of hydroelectric dams constructed after 1965. Agricultural dams, on the other hand, have been completed relatively constantly. From 1945 to 1974, more than ten dams were completed every year. Particularly, in 1963, 1965 and 1968, as many as 22 dams, 25 dams and 31 dams were increasingly added. Although there is a slight decline in the tendency since 1975, nearly ten dams have been completed annually. Regarding multipurpose dams including flood control dams, one dam a year was built from 1945 to 1952 and several dams were built yearly between 1953 and 1971. After 1972, the numbers fluctuated but approximately ten to twenty dams were completed in a year, showing no conspicuous tendency of increase or decrease. The number of industrial dams completed was five a year at most. When the overall trend from 1945 to 1975 is examined, the number reached a peak in early 1960s and started decreasing slightly from 1970.

2.4.2. Comprehensive tendency of effective dam utilization

Figures 5 (1) and (2) show yearly and cumulative information on operational commencement of effective dam utilization. As for data for the years following 2004, projected years are used in the questionnaire,



(1)Yearly data (2)Cumulative data

Figure 4 Change in the number of completed dams

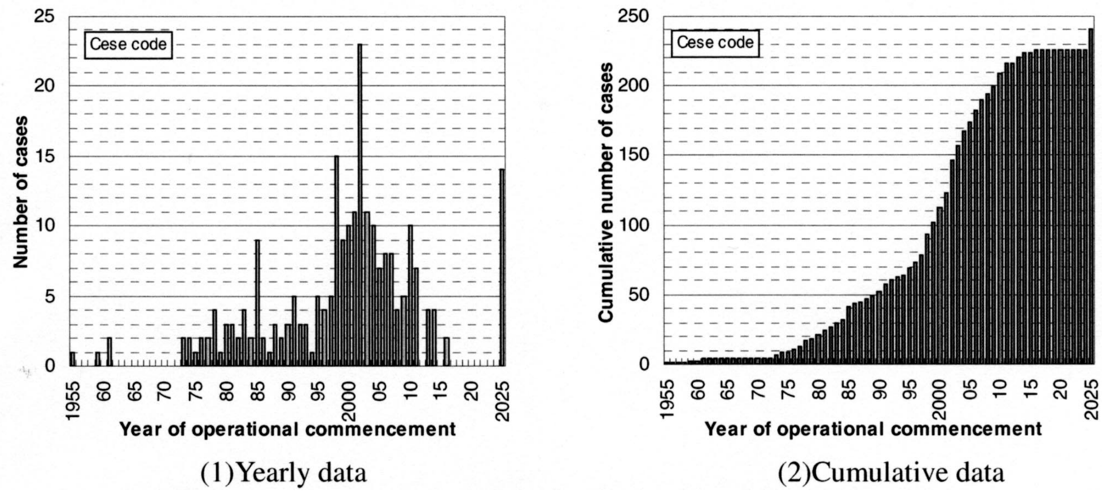


Figure 5 Change in the number of effective dam utilization cases

while those projects that do not have a clear completion date are arbitrarily fixed at 2005.

In this research, 1955 was the year when the effective utilization of an existing dam first began. Several cases of effective dam utilization are identified during the time period between 1973 and 1995 with a peak in 1985. However, the number has increased sharply since then, jumping up to 22 cases in 2003. As far as the cumulative data are concerned, the trend started to progress gradually from the mid-1970s and accelerated greatly after the second half of the 1990s. To date, 74 projects (including those which do not have a clear completion date) are under examination or construction.

2.4.3. Tendencies of effective dam utilization by case code and technical code

Figures 6 (1) through (14) and Figures 7 (1) through (12) depict information on operational commencement for codes 1-(1)-1, 1-(2), 1-(6)-4, 1-(6)-5, 1-(6)-7, 1-(6)-8, 1-(6)-9, 2-(1), 2-(2), 2-(5), 2-(7) and 2-(9), into which a relatively large number of cases were grouped.

From 1959 until 2016, one to five cases a year started or will start operations under the 1-(1)-1 category and this number has constantly increased since the second half of the 1990s. The total number of such cases including those still in the planning process is 49. The increase in the number of capacity reallocation cases, a typical method of effective dam utilization, is a manifestation of the movement developed in the late 1990s in which flood control capability can be increased at a lower cost by making using an existing dam with greater efficiency. Accordingly, the concentration of many cases is higher after the 1990s, reflecting the social demand for a more effective use of existing facilities.

As for the 1-(6)-4 case prior to 1990, the completion of large-sized uppermost reservoirs in river systems introduced the possibility of year-round plant operation, which, as a result, expanded available discharge and realized large peak-discharge operation, enabling the addition of downstream power plants. After 1990, electric power shortage due to economic development came to the surface. To support the peak-load, many measures were taken, in which the operation hours of existing pumped storage power plants were shortened while discharge was increased to boost output.

For the 1-(6)-5 case, the operations began in the early 1970s. The number of cases started to increase after the early 1990s and this tendency became much conspicuous in the late 1990s. The total number of such cases including those still in the planning process is 58. The enhanced social awareness of environmental preservation seems to be reflected behind the scene.

The 1-(6)-7 case appeared in 1979, and its number increased noticeably from the late 1990s. The total number of such cases including those still in the planning process is 20. Through the rationalization of future maintenance and management, it is expected that there will be an increase in the number of elimination of gates.

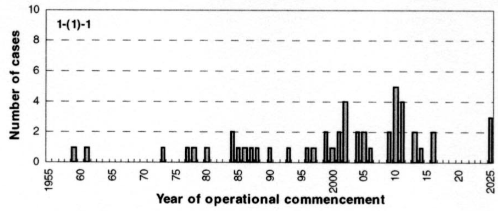
For the 1-(6)-8 case, one example appeared in 1982 and one or two examples after 1995. It is assumed that a review of flood control rules and the modifications in water use will appear more often in future for reasons stated below.

The 1-(6)-9 case appeared as early as in 1959, revived after the first half of the 1970s and rapidly increased from the 1990s.

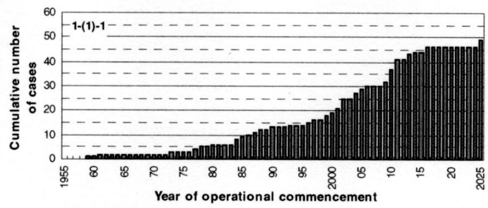
The tendency in the technical code is discussed in this section.

One example of the 2-(1) method was found in 1959. From the late 1970s until 2000, the number remained as constant at approximately one case per year, and this figure has increased in recent years. The total number of such cases including those still in the planning process is 38.

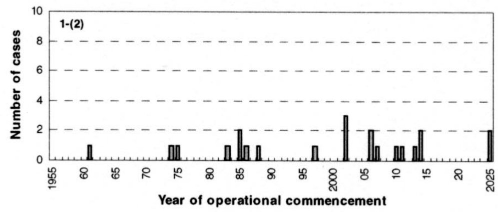
The 2-(2) method appeared in the mid-1970s, and the figure tended to increase from around 1991, suggesting a growing demand for flood control and water



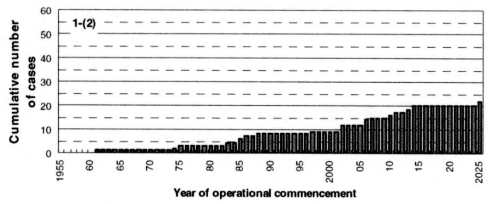
(1) Yearly data for 1-(1)-1



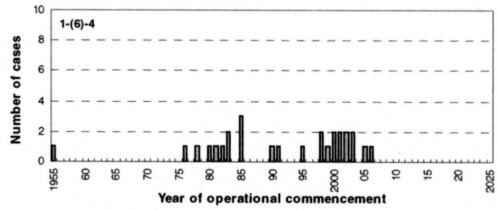
(2) Cumulative data for 1-(1)-1



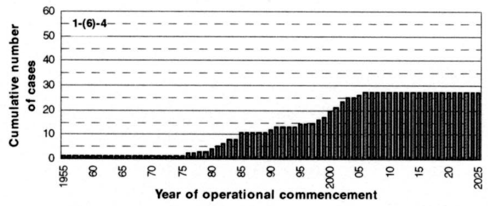
(3) Yearly data for 1-(2)



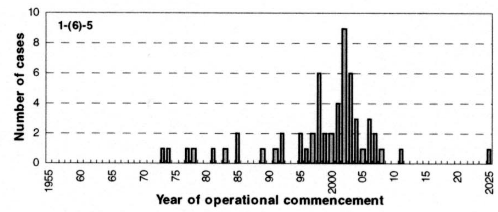
(4) Cumulative data for 1-(2)



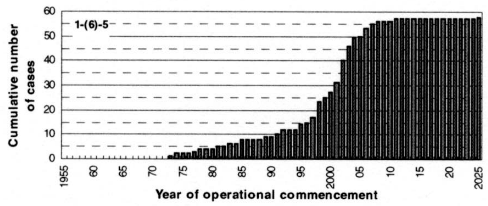
(5) Yearly data for 1-(6)-4



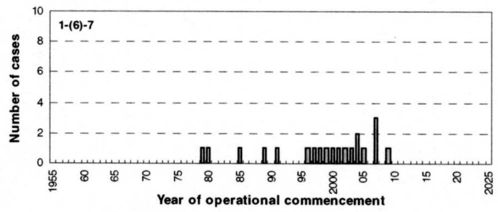
(6) Cumulative data for 1-(6)-4



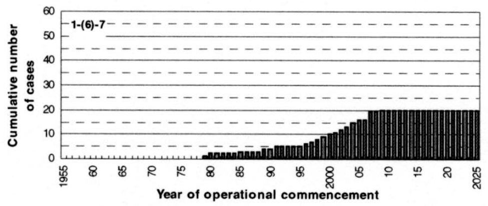
(7) Yearly data for 1-(6)-5



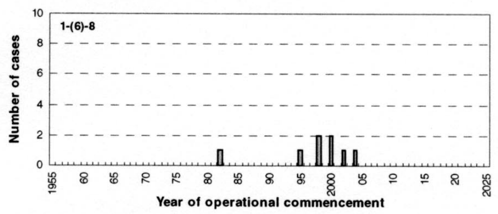
(8) Cumulative data for 1-(6)-5



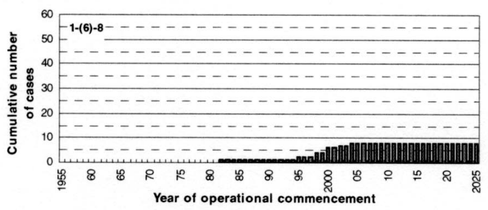
(9) Yearly data for 1-(6)-7



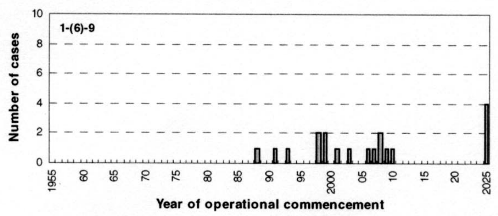
(10) Cumulative data for 1-(6)-7



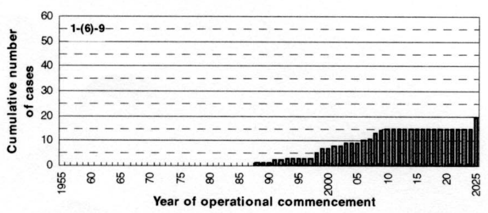
(11) Yearly data for 1-(6)-8



(12) Cumulative data for 1-(6)-8

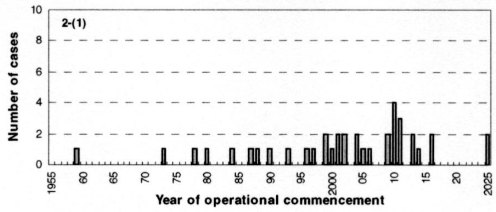


(13) Yearly data for 1-(6)-9

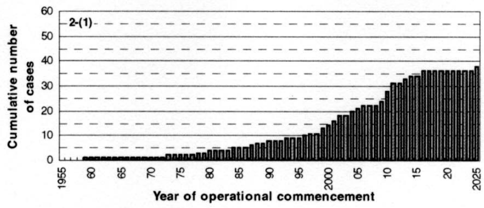


(14) Cumulative data for 1-(6)-9

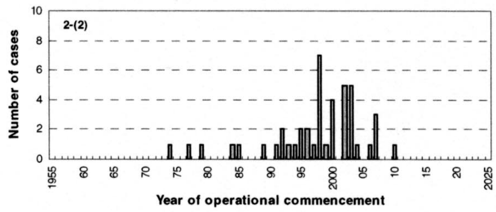
Figure 6 Change in the number of effective dam utilization cases (case code)



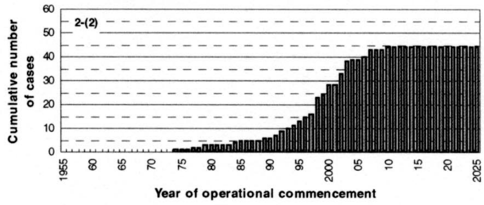
(1) Yearly data for 2-(1)



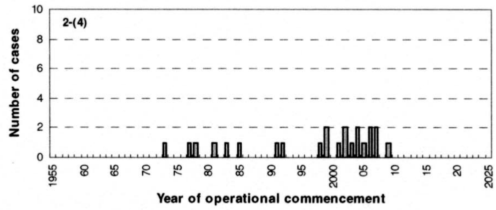
(2) Cumulative data for 2-(1)



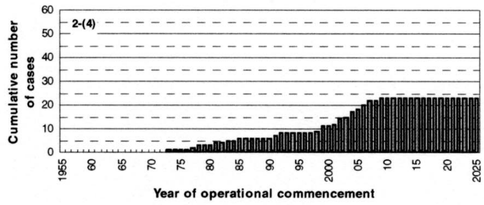
(3) Yearly data for 2-(2)



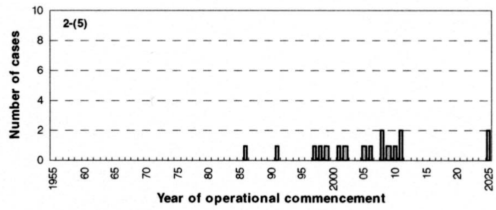
(4) Cumulative data for 2-(2)



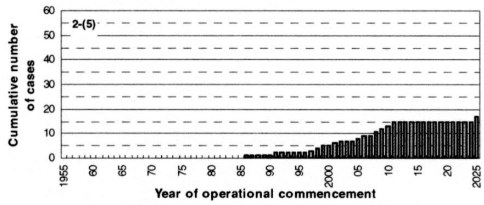
(5) Yearly data for 2-(4)



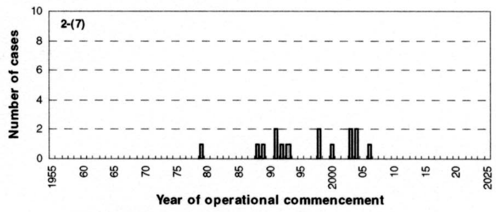
(6) Cumulative data for 2-(4)



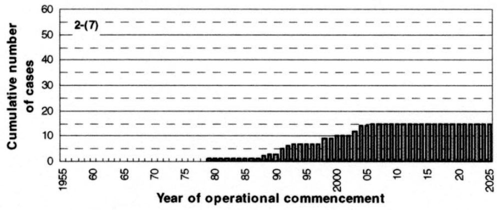
(7) Yearly data for 2-(5)



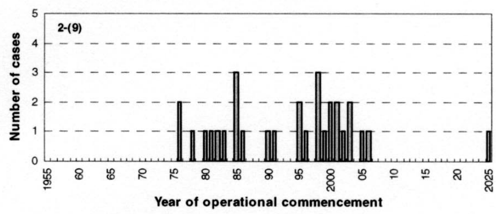
(8) Cumulative data for 2-(5)



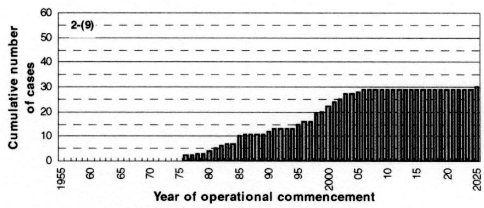
(9) Yearly data for 2-(7)



(10) Cumulative data for 2-(7)



(11) Yearly data for 2-(9)



(12) Cumulative data for 2-(9)

Figure 7 Change in the number of effective dam utilization cases (technical code)

supply. The total number of such cases including those still in the planning process is 44.

The 2-(4) method is a technical measure for the 1-(6)-5 case, and therefore these data show a similar tendency. This method started to appear in the mid-1970s as a water preservation measure against cold/turbid water in reservoirs and increased slightly in the late 1990s. Although it is suspected that the social background of a deterioration in forest management has led to the increase in the number of sources of turbid water, the promoted social awareness of environmental preservation and the advancement of relevant engineering techniques encouraged implementation of the measure in the late 1990s and after. The total number of such cases including those still in the planning process is 23.

Effective dam utilization employing the 2-(5) method was first conducted in 1986. One or two projects have been carried out annually since 1991. The total number of such cases including those still in the planning process is 17.

The figure of the 2-(7) methods tended to increase from around the late 1980s, and the current total number is 15.

The figure of the 2-(9) methods tended to increase from around the late 1970s, and the current total number is 30.

2.5. Background of Overall Tendency

In Japan, the Comprehensive National Development Plan has been revised every ten years, and a part of which the Water Resources Plan has been compiled. In the 1989 Water Resources Plan, for example, a new concept of water environment was characteristically introduced in addition to the goal of quantitatively securing enough water. The plan also mentions an environmental effect brought about by the construction of water resource development facilities such as dams, and discusses the importance of maintenance works to prevent their superannuated malfunction. On that point, however, a prospect to positively promote effective utilization of existing stocks has yet not been drawn up. Nevertheless, the plan underlines the necessity to actively preserve and improve the quality of stored water in accordance with local situations and the importance of an integrated management system to bring about greater efficiency in the function of existing facilities.

However, in the 1999 Water Resources Plan includes a perspective of securing water quality depending on water supply purposes to establish a sustainable water supply system. Its concrete measure, "effective utilization of existing water resource development facilities" was proposed to take into consideration the yearly cost increase in water use, and the report stresses the necessity of positively promoting effective measures which were already in existence by that time. In relation to the Environmental Impact Assessment Law enacted in 1997, there is a greater emphasis on the concern for environmental preservation while executing any project.

As stated above, the emphasis of Japan's water resource planning has shifted over time from the construction of new water resource development facilities to the effective utilization of existing facilities. Effective dam utilization measures are called for because of the "cost increase in water use," and "environmental awareness, including the preservation of water quality." It goes without saying that the increase in the cost of water use is an important element affecting the feasibility of a water development project in the light of stringent finance of both national and local governments. From the standpoint of cost-benefit efficiency, methods which are capable of securing new water resources by improving existing facilities such as dam heightening are attractive options. Underlying this is the complexity of water resource development and the shortage of appropriate dam construction sites. Another important viewpoint is the fact that there is the impact on the society by caused by engineering activities such as site transference is very small. Awareness of the dam-related river environment has greatly triggered an increase in the promotion of improving the functionality of existing facilities. Also noteworthy in the tendency of effective utilization is the addition of selective intake facilities since the 1990s so that proper water quality can be secured according to the purpose of water use.

Changes in the pattern of water demand over time are also a powerful driving force in promoting the effective utilization of existing facilities. There are a number of cases in which effective utilization measures have been taken with the aim of converting from one water supply purpose to another or for changing water supply capacity into flood control capacity. In these cases, remodeling or construction of an outlet structure may be required but the cost is much more reasonable than executing a large-scale project, including dam heightening. To utilize the existing stocks with as great efficiency as possible, it may be necessary to rethink the original purposes of a dam and to apply effective utilization measures to several dams together. Some examples of implementing and planning the cooperative operation of multiple dams can be seen in recent years. In addition to the transference of capacity mentioned above, it is expected that the focus of effective dam utilization will shift from on physical technology to on non-physical factors.

Characteristic tendencies of the effective utilization of existing dams can be summarized as follows.

1) A full-scale movement of effective dam utilization started in the mid-1970s and the number of cases rapidly increased from the late 1990s.
2) An increase of maintenance flow came into practice from the early 1970s, and the number of cases grew from the early 1990s, showing a noticeable increase since the late 1990s.
3) An improved selective intake function as a water quality preservation measure was introduced in the mid-1970s, and the number of cases increased slightly during the late 1990s. Works to improve selective intake functions have been applied only to those dams that were completed by the early 1980s.
4) The implementation of a sediment-related function has been mostly came into practice after 2000.

Among all the dam operators, the tendency of those conducting effective utilization of "irrigation-oriented dams" can be described as follows.

1) Measures for the effective utilization of existing dams began to be applied in relatively recent years.
2) The proportion of effective dam utilization cases for

river environment improvement is lower while that for increasing capacity for single agricultural use is higher.
3) Capacity transference came into practice while at the same time reinforcing dam functions (restoration and improvement of downstream disaster prevention efficiency, functional longevity and smooth management) and changing the purposes of water supply, as a result of the reduction in the area to be served by irrigation.
4) The introduction of small-scale hydroelectric power generation has come into practice to reduce maintenance and management costs.

"Dams for development of electric power resources" has been effectively utilized in the following manners.
1) As some dams were constructed prior to World War II, their equipments have become aged, and for these, some gates have been renewed or a gateless system has been introduced.
2) To respond to the demand for qualified water, selective intake facilities have been installed and the maintenance flow has been secured.
3) To more effectively utilize pumped-storage power plants, generators have been added in some power stations.

"Flood control oriented multipurpose dams" are effectively utilized in the following manners.
1) In addition to the integrated operations of multiple existing dams, collaborative operations between new dams and existing dams have been conducted.
2) Flexible management of dam storage has being promoted.
3) Preliminary discharge has come into practice for the purpose of flood control by making effective use of the capacity saved for other purposes.

3. CHALLENGES IN THE SMOOTH IMPLEMENTATION OF EFFECTIVE DAM UTILIZATION PROJECTS

Under restricted topographical and geological conditions, the achievement of an effective development of water resources has become more difficult. While financial conditions for improving infrastructure are stringent, the social awareness of the natural environments has grown. Under these circumstances, a tendency to make effective use of existing dams will developed and promoted with the aim of satisfying water demands (including requirements in the environmental and sediment management) in the current and future society at a low cost although the priorities and execution periods differ among the various project fields.

In order to more smoothly perform the effective utilization of existing dams, there are various challenges to be overcome. JCOLD intends to continue to carry out close examinations to make effective use of existing dams. On the basis of the investigations carried up to this time, it is proposed that the following need to be dealt with:

* Establishment of diagnostic procedures to verify the functionality and soundness of existing structures
* And examination of the countermeasures for sediment accumulation according to dam conditions
* Data-based safety execution using Information-Technology (e.g. dam drilling)
* Development of a low vibration method for dam drilling
* Invention of the optimum cofferdam to carry out redevelopment works while maintaining reservoir operations
* A rational design for a diversion work at the time of dam heightening
* Adoption of a large-scale cavity excavation technique to construct a large connecting water channel
* Provision to meet opportunities for stakeholders to build a consensus regarding the integrated operation of dams

To facilitate effective dam utilization, dam-related legal and institutional aspects (e.g. property rights and water rights) would also have to be examined. Supportive measures including fund-raising aids also need to be investigated.

ACKNOWLEDGEMENTS

We would like to thank the regional development bureaus of the Ministry of Land, Infrastructure and Transport, regional agricultural administration offices of the Ministry of Agriculture of Forestry and Fisheries, Japan Water Agency, prefectural governments, electric power companies and staff members in these organs. They accepted and responded to a questionnaire survey form to provide data. Thanks to their support, this report has been issued.

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Rehabilitation of a Large Dam by Increasing the Capacity of Spillway (Im Ha Multipurpose Dam in Korea)

Baek, Un Il
Vice-president of Daelim Ind. Co., Korea

Park, Hyung Seop
PPM of the construction of an emergency spillway in Im Ha dam, Korea

1. Introduction

It is well-known to the world that unexpected severe rainfall has frequently come recently in the worldwide due to the climate change. As a consequence there were a lot of happenings take place in our country by the increased flood discharge in the multipurpose dams at present. In order to tackle these phenomena, in general, 3 ways how to resolve the increased flood discharge can be classified as follows.

At first, increasing the capacity of flood control by heightening the dam height. Secondly, increasing the capacity of the rate of flood discharge into the downstream river by making another spillway. Thirdly, connecting into the adjacent dam either by the open channel or by the diversion tunnel to absorb the unexpected concentrated rainfall into the watershed of the dam. However, first 2 methods have been adopted in our country as a useful solution like as a common use in the other countries.

In the article construction of a new spillway in the multipurpose dam at present will be introduced in order for increasing the flood discharge.

To enlarge the capacity of flood control, it can be categorize into 2 conceptual designs for building a new spillway either for supplementary use or for emergency use according to the purpose of usage. From the technical points of view, although there is a little difficulty to build a tunnel spillway, open tunnel spillway, as an emergency spillway conceptually, has been applied in order to fulfill environment-friendly construction which is a hot issue in 21 century in the world on these days.

Planning and concept of the basic design of the emergency spillway in Im Ha multi-purpose dam in Korea will be introduced briefly at the below.

2. Background

2.1 Increased frequency of the concentrated rainfall due to climate change

Frequency of the locally concentrated rainfall has been enormously increased in our country due to the climate change. Occurrence of the concentrated rainfall in Korea is shown in the Table 1 below.

2.2 Increased design flood by the increased intensity of the rainfall

Basic planning and construction of Im Ha multi-purpose dam (Im Ha dam from below) have been established in the year of 1985 and 1992 respectively. In the year 2003, assessment for hydrologic stability of Im Ha dam was carried out for realizing what kind of an impact due to the climate change. An increased precipitation and peak discharge have been found as follows in the Table 2.

Table 1 Occurrence of the concentrated rainfall

Year	1981	1987	1998	2002
Region	Jang Heung	Bu Yo	Po Hang	Gang Rung
Rainfall(mm)	655	605	610	880
Duration(hrs)	47	48	45	23

Table2 Precipitation and flood discharge at each frequency

	Precipitation (24hrs, mm)		Peak inflow (㎥/sec)		Peak discharge (㎥/sec)
	200 yr.	PMF	200 yr.	PMF	PMF
Basic plan of Im Ha Dam (1985)	213	424	4600	7550	5200
Assessment for hydrologic stability (2003)	234	561	5200	14800	13500

Through the assessment in the year 2003, it has been found that PMP and PMF have been increased 32% and 96% respectively.

2.3 Demand for increasing of flood discharge for the safety of the dam

According to the design criteria reinforced recently, large dams in Korea have to adopt PMF as a design discharge. Therefore, an emergency spillway, especially in the large rockfill dams, has to be built in order to reinforce existing spillway up to probable maximum discharge through the spillway. To decide the additional discharge capacity by the emergency spillway which will be built newly, Height-Discharge curve of an existing spillway has been investigated and is shown in Fig. 1. For references, maximum flood level of Im Ha reservoir is at EL. 165.8m and the crest of dam is at EL. 172.0m. As it is seen in the figure, existing spillway can discharge 5,300cms at the maximum flood level of EL. 165.8m. In order to satisfy increased maximum discharge of 13,500cms newly built emergency spillway should have a release capacity of 8,200cms at the peak time.

3. Selection of type of an emergency spillway

In general, type of the spillway of a large scale discharge is classified into 2, which is either tunnel type or open channel type. For the case of Im Ha dam, best locations for these 2 types from the geomorphologic and geological point of view are as shown in Fig. 2 respectively. Longitudinal and plan views of each type are shown in the Fig. 3. Compared results from the hydraulics, environmental and geological points of view are as shown in the Table 3. Moreover, there is a decisive handicap to keep up with the construction period of 3 years after award the project proposal in the open channel type of spillway.

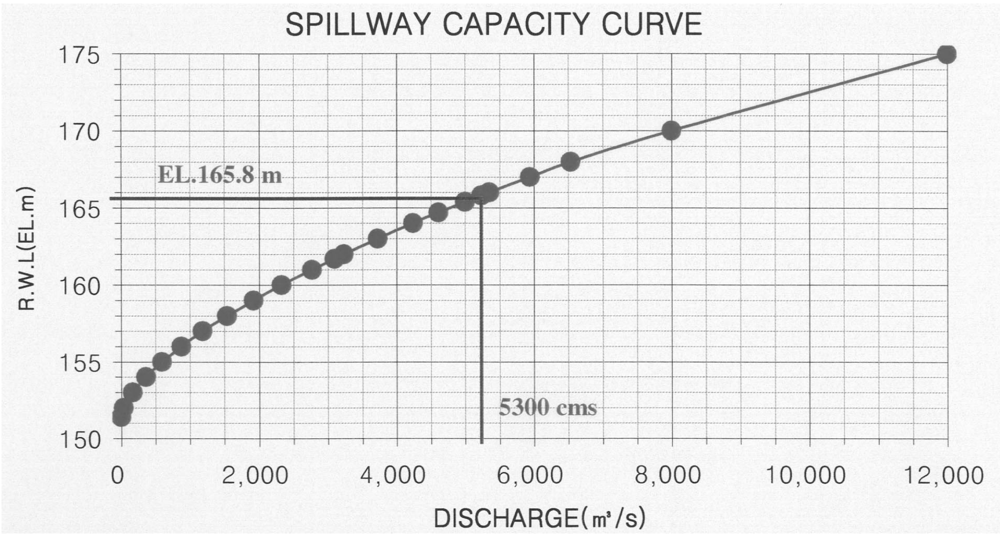


Fig 1 H-Q curve of an existing spillway

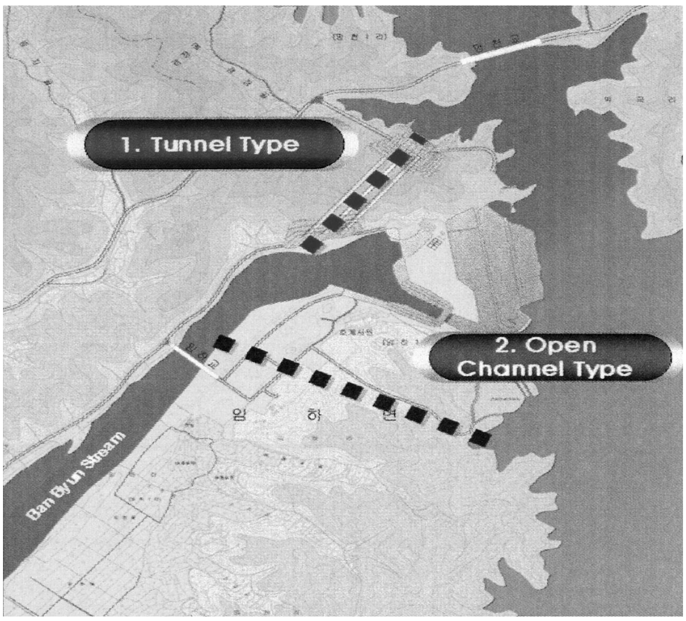
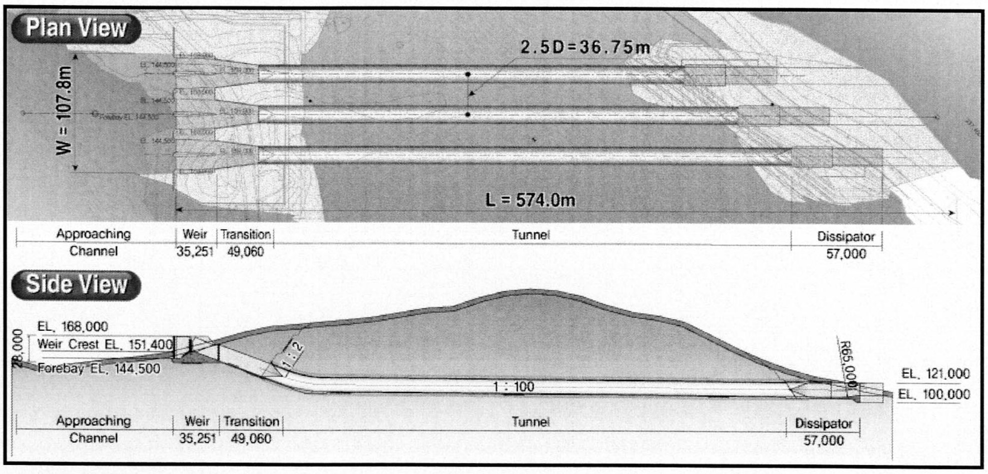
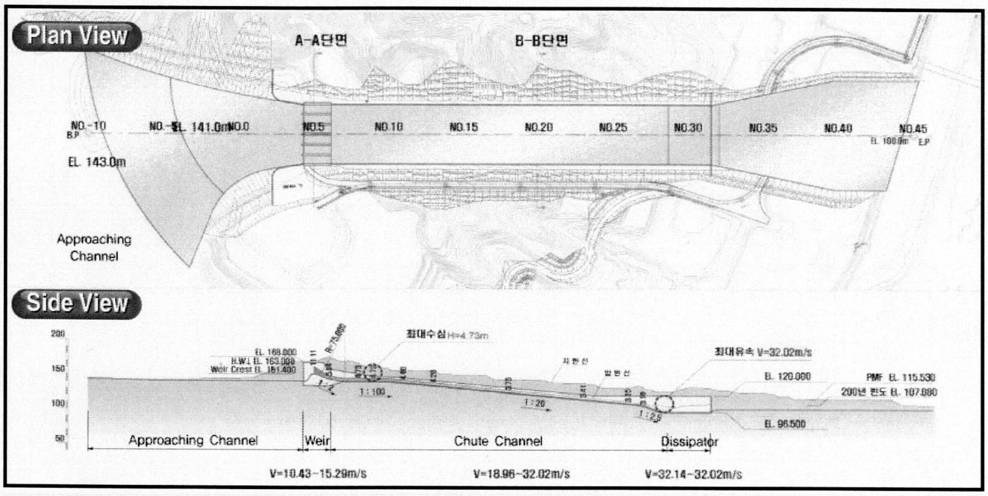


Fig 2 Substantial location of each type of spillway



(A) Tunnel type



(B) Open channel type

Fig 3 Longitudinal and plan view of each type of spillway

Table3 Comparison table of each type of spillway

View Point	Tunnel Type	Open Channel Type
Hydraulic Aspect	Geographical revision of Approaching Zone is required.	
	Advantage to the flow stability near junction of downstream(parallel discharge)	Disadvantage to the flow stability near junction of downstream (perpendicular discharge)
	Complicated flow configuration in transition zone	Simple flow configuration in transition zone
	High flow velocity ⇒ Aerator is required	High flow velocity ⇒ Aerator is required
Structural & Geological Aspect	Not sensitive to weathering	Sensitive to weathering
	Not generate a large scale of the cutting ground zone	Generate a large scale of the cutting ground zone
	No need of spillway-cross bridge	Need of spillway-cross bridge
	No need of countermove for safe in the entrance road	Need of countermove for safe in the entrance road
Environmental Aspect	Less environmental destruction due to the excavation	More environmental destruction due to the excavation
	No need of Eco-Bridge	Need of Eco-Bridge
	Less generating of noise, vibration, and dust	More generating of noise, vibration, and dust
Final Decision	⊙	

Table 4 Main features of the basic planning

Tunnel Type	Weir crest	Incident angle of Chute	Length of transition	Section Changing Length
D14.7×3Diversion	EL. 151.4m	10°	35.7m	13.4m

4. Technical difficulties of the tunnel spillway

Basic planning of an emergency spillway of Im Ha dam has been performed in the year of 2004 immediately after the assessment of hydrological stability has been carried out in the year of 2003. Tunnel type spillway with an open channel style has been recommended in the planning stage. Competition of basic design, as a Turnkey basis, was issued right after the planning has completed. Main features of the basic planning are as shown in the Table 4.

Technical difficulties found out after that numerical model and hydraulic scale model have performed for the basic planning were as follows:

(1) Unstable flow at the access channel
(2) Negative pressure occurred at the slowly varying chute
(3) Water level rise at the end of the transition zone
(4) Harsh flow pattern inside the tunnel
(5) Uncertain energy dissipation at the end sill

5. Optimum design of tunnel spillway

In order to optimize the basic design of tunnel spillway and rectify the problems outcome from the basic planning stage the following procedure (Fig. 4) has been established.

Analysis and solution of the problems occurred at the basic design stage have carried out through the procedure written above and the final results are shown in the Table 5.

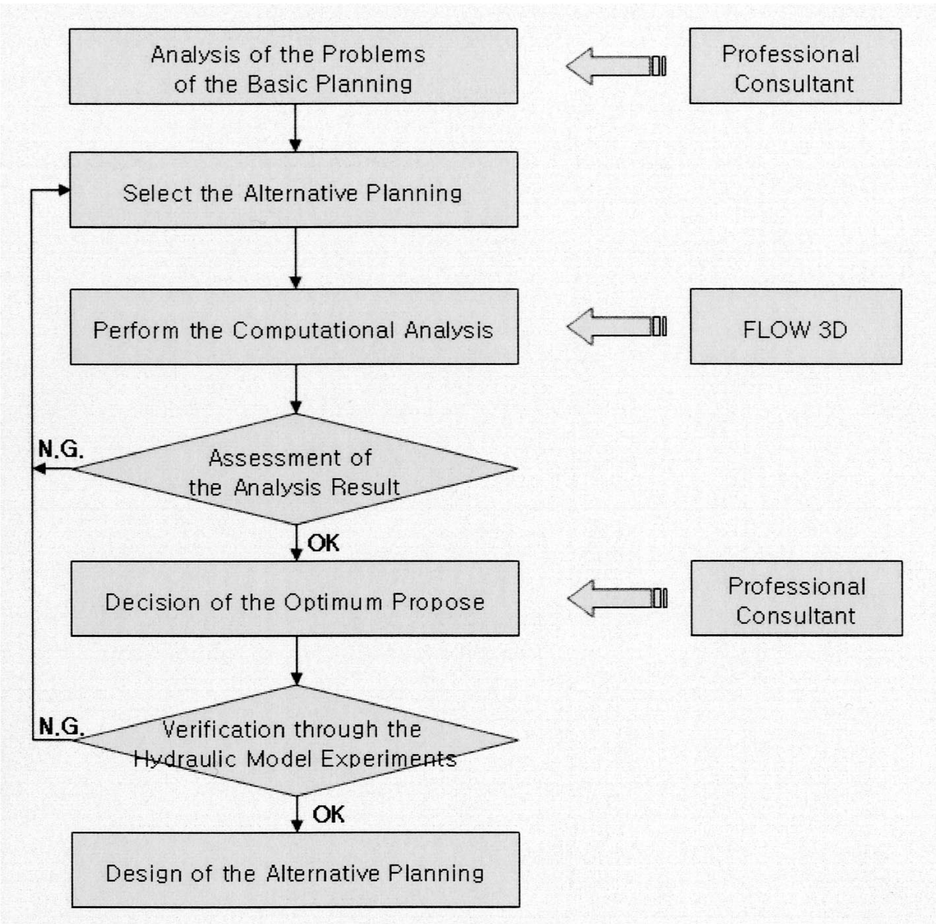
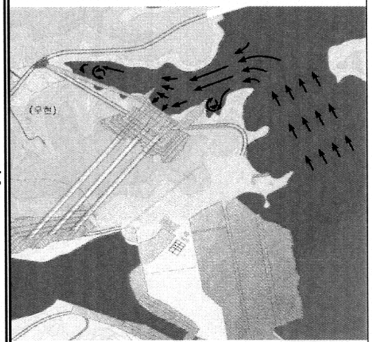
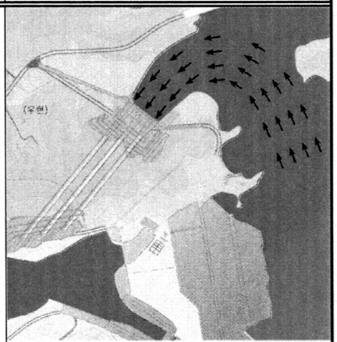
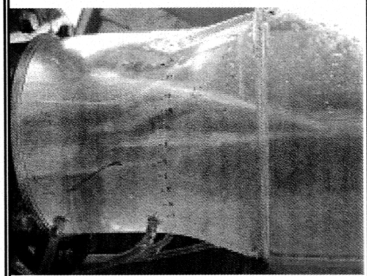
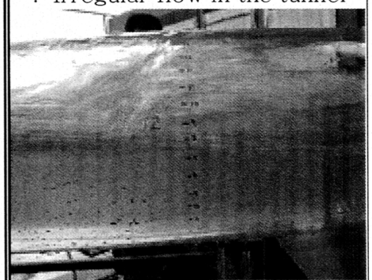
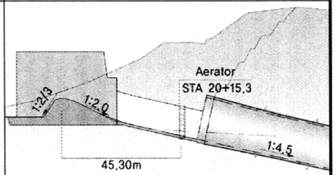
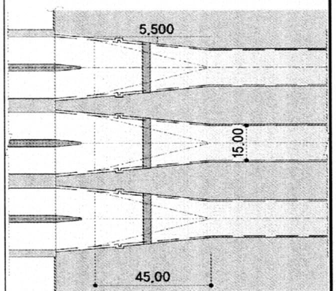


Fig. 4 Analysis of the basic plan

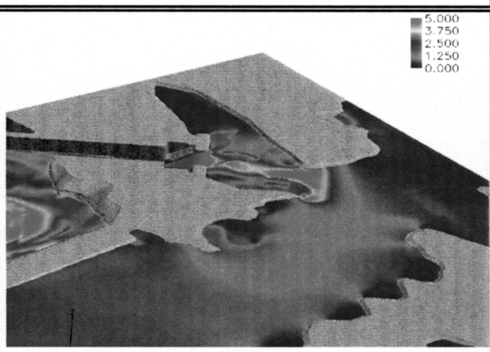
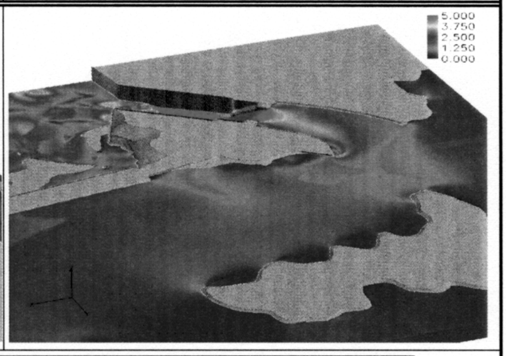
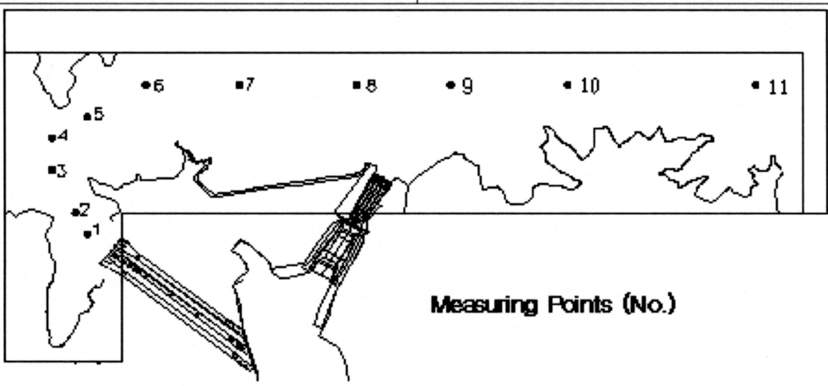
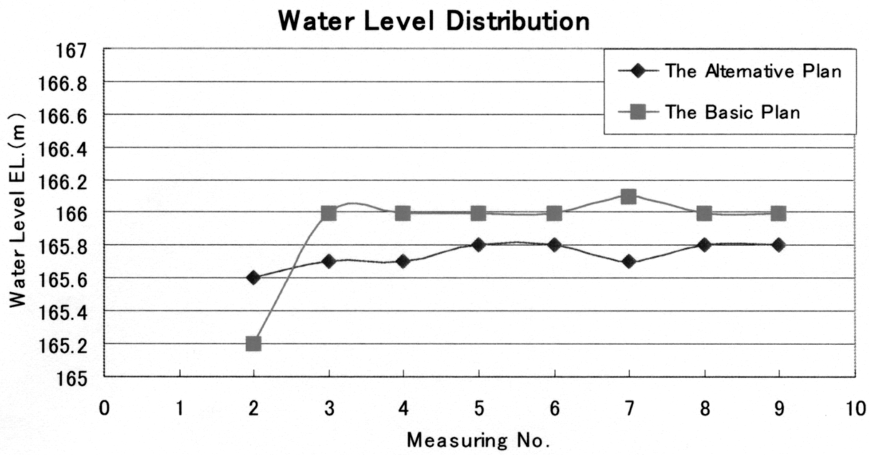
Table 5 Analysis and solutions for optimum design

Zone	Problems	Problem source	Solution & Application
Approaching channel	 . Unstable flow	. Flow stagnation near Approaching Zone . Complicated geo-logical features	 . Modifying the shape of Approaching channel . Forebay EL.144.5m
Weir zone /Transition Zone /Tunnel chute	 . Rising water level at the end of Transition Zone . Irregular flow in the tunnel  . Generating negative pressure in the flow section changing area (Av. negative pressure: -5.5m)	. Rough flow near the Weir Zone . Steep slope of the downstream of the weir . Excessive transition angle . Rapid flow section changing . Generating rooster tail	 . Modifying the slope of the downstream of the weir  . Revision of the approach wall , pier width, and nose shape . Decreasing the angle of Transition Zone . Increasing the length of Transition Zone . Gradually changing the flow section
Dissipator	. Unstable dissipation of the energy	. Flip bucket tip EL. is lower than T.W.L . The bucket shooting angle is too small	. Increasing the tip EL. . Increasing the bucket shooting angle

6. Verification of the optimum design

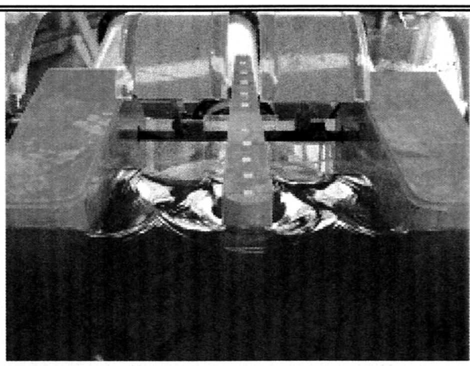
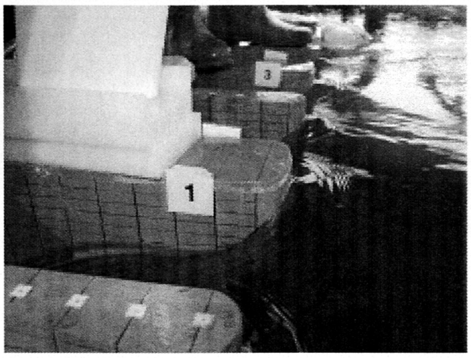
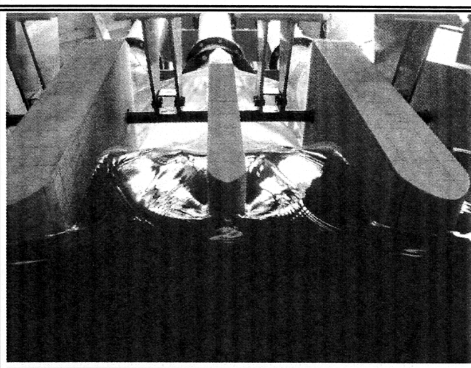
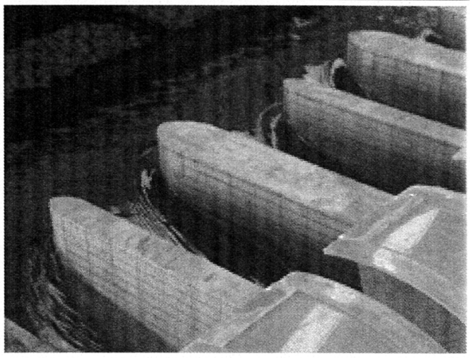
The problems found after hydraulic scale model test of basic planning have been rectified by the numerical model test through Flow 3D model. Basic design has been verified through the hydraulic model test as shown in the figures below.

6.1 Approaching channel

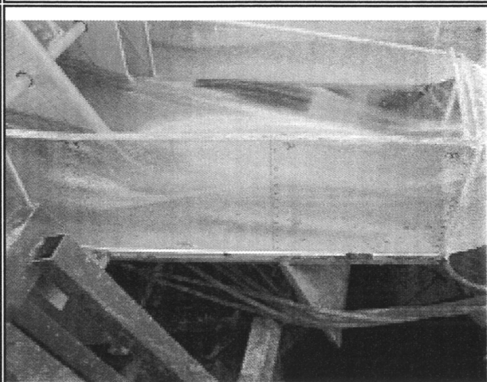
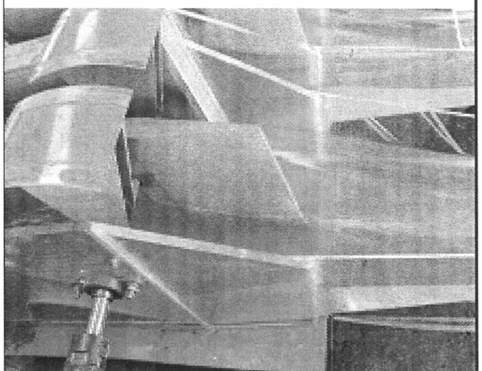
<table>
<tr><th>Specification</th><th>The Basic Plan</th><th>The Alternative Design</th></tr>
<tr><td>Flow velocity</td><td>
</td><td>
</td></tr>
<tr><td rowspan="2">Water Level</td><td colspan="2">
</td></tr>
<tr><td colspan="2">
</td></tr>
<tr><td>Discharge capacity</td><td>◦ 7,900 cms at H.W.L EL. 165.8m</td><td>◦ 8,000 cms at H.W.L EL. 165.8m</td></tr>
</table>

6.2 Weir Zone / Transition Zone / Tunnel Chute

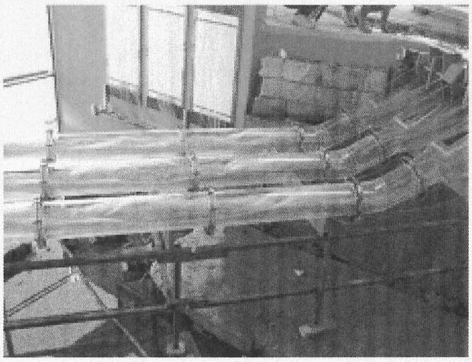
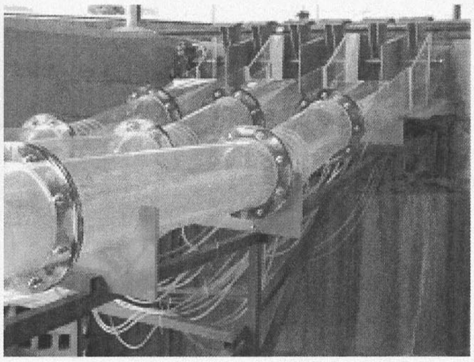
1) Weir Zone

Classification	The Basic Plan	The Alternative Design
Hydraulic Model Experiment	 	 
Test results	◦ Stable flow though the weir	

2) Transition Zone

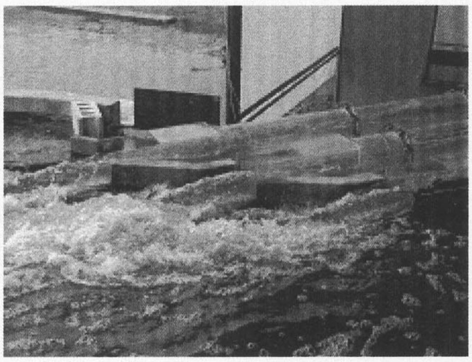
Classification	The Basic Plan	The Alternative Design
Hydraulic Model Experiment		
Test results	◦ No rooster tail behind the pier ◦ Uniform flow level in the Transition Zone ◦ Reducing the phenomenon of the water-level increasing	

3) Tunnel Chute

Classification	The Basic Plan	The Alternative Design
Hydraulic Model Experiment		
Test results	◦ Stable flow in the Tunnel Zone : Av. water depth 8.5m : Av. flow velocity 28m/sec	

6.3 Dissipator

Table 9

Classification	The Basic Plan	The Alternative Design
Hydraulic Model Experiment		
Test results	◦ Inflow angle to the downstream 40°, flying length 40m ◦ Stable dissipation of the energy	

7. Conclusion

It is found, through the 8 month preparation for Turnkey proposal of an enlargement of emergency spillway of Im Ha dam, that the open tunnel spillway is better than the open channel spillway not only from the environmental points of view but also from the geomorphologic points of view at the downstream river. In addition to these findings it is required to consider several hydraulic features written below because that tunnel spillway is a difficult hydraulic structure than the open channel spillway.

Approach zone: shape of approach channel, elevation of Forebay, shape of approaching wall, shape and width of pier
Transition zone: contraction angle, length and inclination of Chute
End shape of Pier: to prevent severe Rooster tail effect
Dissipator: angle and tip EL. of Flip bucket

Introduction of tacking of those features, which is undergoing at the moment, will be presented in the Seminar on October this year.

Treatments to A Large Scale Horizontal Crack Along Placing RCC Layer of Linhekou RCC Arch Dam

Chen Guanfu Zhou Jianping Zhao Quansheng
China Hydropower Engineering Consulting Group Co., Beijing, China

Abstract

Linhekou hydroelectric project was put in operation on May 1 in 2004, its RCC arch dam is 96.5m high and a large scale horizontal crack along placing RCC layer appeared and then was successfully treated during the construction. This article firstly introduced the appearance of the crack, roughly analyzed the cracking cause; specially introduced the treatment measures, those are needed to meet the requirements of the safe operation, to resume the integrity and the seepage resistance of the arch dam. The measures mainly include chemical grouting into crack surface and sealing the crack boundary on the dam upstream and downstream face, also include anchor steel piles through the crack surface. Through the checking of water pressure test, temporary water retaining during flood season before dam completion, initial filling of dam and the generation operation for more than one year, it indicates that the crack treatments are successful and ensure the safe operation of the arch dam. The article also provides lessons for other RCC dam construction and valuable suggestions for similar crack treatment.

Key words: *hydroelectric project, RCC dam, crack treatment, chemical grouting, anchor steel piles, SR plastic filler, water pressure test, boring concrete cores*

1. Introduction

Linhekou hydroelectric project is located in Shanxi province, its total installed capacity is 72Mw, its water-retaining structure is RCC arch dam with maximal height of 96.5m, the bottom thickness of the crown cantilever section is 27.2m, the elevation at the dam crest is 515m, the arc length of the dam crest is 311m. The dam RCC volume is about 224600 m^3, which is about 70 percents of the total dam concrete volume. Contractor began to place dam RCC on December 23 in 2001, compacted thickness of the placing RCC layer is 30 cm. Dam RCC placement was completed on June 25 in 2003, dam began to retain water on October 27 in 2003, the project started to generate waterpower on May 1 in 2004.

In August 2002, a horizontal crack at EL.4551.m was found on the upstream face of right-hand dam, as shown in Figure 3. Water pressure test and bored dam concrete cores indicated that, the crack was along the surface between the eighth and the ninth placing layer of the 11th placing lift, which is from El.452.7m to El.455.7m, the crack developed from the dam upstream face to the downstream face, the total cracking area is nearly 1000m^2, the crack width is less than 0.3mm. After careful observation, indoor test and field experiments, measures including chemical grouting into the crack surface, sealing the crack boundary and installing anchor steel piles through the crack were carried out twice for the arch dam.

After the crack treatment, the results of water pressure test and bored dam concrete cores, temporary water retaining during flood period before dam completion, initial filling of dam and generation operation for more than one year all verify that the crack treatment is successful and then ensures the safe operation of the dam. At the same time, it provides lessons for other RCC dam construction and valuable suggestions for similar crack treatment.

2. The expanding process and the formation cause of the horizontal crack

2.1 Crack expanding

A horizontal crack was discovered on the upstream face of the right-hand dam on August 8 in 2002 (the crack may appeared previously, but was not discovered in time), after careful observation, another horizontal crack was also discovered nearly at the same elevation on the dam downstream face, and there was $Ca(OH)_2$ resolved out of the cracks. The elevation of the upstream crack is 455.1m, the downstream crack is at El.455.26m owing to the placing RCC layer inclining towards upstream slightly. Simple water pressure test was carried out on August 16 in 2002, water leaked out of both the upstream crack and the downstream crack, and water splashed from several locations of the cracks; the length of the upstream crack is about 40m, the downstream length is about 20m at that time.

After the chemical grouting for the first time, we found that there still was water leaking out of the

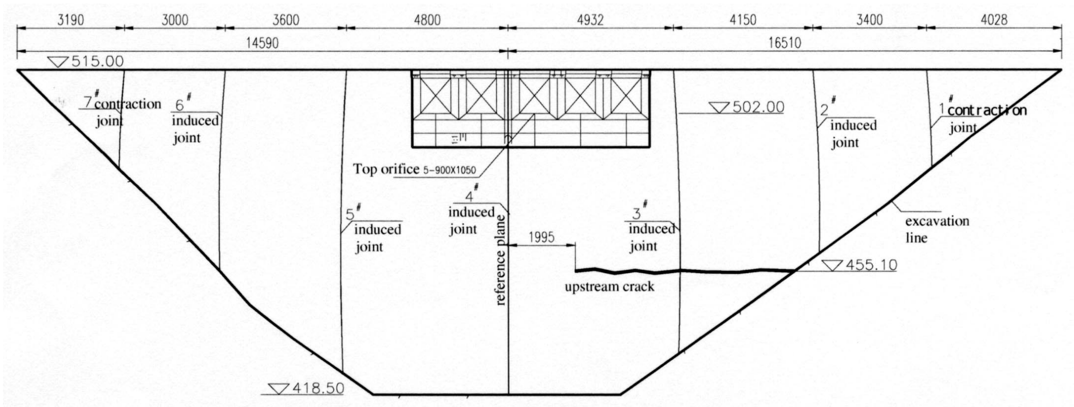


Figure 1. The Upstream view of Linhekou RCC arch dam (unit: mm for length)

cracks on October 21 in 2002, and the cracks were extending toward the crown cantilever. It indicated that the first chemical grouting was not successful.

The crack treatment for the second time began on November 20 in 2002. During drilling chemical grouting holes, especially when drove away the water existed in the grouting pipe system and the crack surface by high pressure wind, cracks further extended toward the crown cantilever by judging from the wet cracking trace. By surveying, the upstream crack at El.455.1m extended from the right abutment to the location that is 19.95m right from the reference plane as shown in Figure 1, the downstream crack at El.455.26m extended from the right abutment to the location that is 28.6m right from the reference plane, the crack runs through the upstream face to the downstream face, the total crack area is about 1000m^2, it is infrequent in RCC dams, and the treatment is very difficult.

2.2 Preliminary analysis on the cause of the horizontal cracks

According to the test results of the contractor's and the owner's laboratory and the monitoring recordings of the engineer, the raw material quality, concrete construction quality, RCC physical and mechanical parameters and the results of bored dam concrete cores show that, the RCC quality of the 11th placing lift has no difference from other lifts and can meet the design requirements, at the same time, the time interval between the eighth and the ninth placing RCC layer also satisfies the requirements of *construction specification of Roller Compacted Concrete DL/T 5112-2000*.

According to the recordings of concrete construction, when placing the 11th RCC lift from May 6 to May 10 in 2002, water leaked from several tie-ins of the buried cooling PVC pipes, and then the cooling pipes under El. 461.0m were blocked up with sand grout on August 30 in 2002. During the period from May 6 to August 30 in 2002, the placed RCC was cooled by cooling water (about 6~9˚C), sometimes cooled by natural river water (from May 10 to May 25, the river water temperature is about 15~24˚C). The cooling water pool is located at El. 530.0m. The water head from the water pool to the crack surface is about 75m. Thus, the water pressure at the leaked tie-ins of the cooling pipes is very high, and the water splitting action is powerful. In addition, there is a temperature drop between the cooling water and the just placed RCC mix, for example, in June 2002, the cooling water temperature is 6˚C~9˚C and the water temperature at the exits of cooling pipes is 10˚C~18˚C, the temperature drop is about 4˚C~9˚C at that time. The strength of the just placed concrete is very low disadvantageously at that time, when cooling the RCC mix just after spreading them as Linhekou project, the water leaking out of the cooling pipes will dilute and sweep away the cementious materials along the placing RCC layer with the help of the high water head, then deteriorate the bond quality of the interface between placing RCC layers, then form the weak interface, even form initial crack by the tensile stress caused by the temperature drop.

As for the crack development after the crack treatment for the first time, one side is the tensile stress caused by the internal concrete temperature drop. For example, the thermometer T341 is close to the crack position, located in the middle of the 3# and the 4# induced joint at El. 460.0m and is 3.4m away from the dam upstream face, the temperature drop is about 13˚C in October 2002; one the other hand, high pressure water and wind used for cleaning the crack surface during the treatment for the first time, produced stress concentration at the crack tip, it leads to the further extending toward the crown cantilever.

Thus, water leaking out of the cooling pipes, water splitting action induced by cooling water head and temperature drop by cooling RCC too early are possibly the original cause of forming the weak plane and the horizontal crack along placing RCC layers, then the tensile stress caused by the internal temperature drop and the external excitations of high pressure water and wind used for crack treatment may speed the development of the crack.

Due to the limitation of preliminary discussion, the above deductions on the cause of the crack formation are just based on the qualitative analysis, and the final conclusions should be made through further researches.

3. Comprehensive treatments to the crack along placing RCC layer

3.1 Crack treatment for the first time

According to the requirements of *design specification for concrete arch dams SL282-2003* on the tensile strength of dam concrete and the characteristics of arch dam stress distribution, the treatment measures for the first time included 184 drilled holes for anchor steel piles through the cracking surface, 84 drilled holes for chemical grouting and sealing the external boundary of the cracks.

According to the seepage resistance requirements of the arch dam, a long narrow slot cut along the upstream cracking trace, then a kind of waterproof material named as SR and the epoxy sand grout were filled into the slot, finally, a layer of epoxy vitreous texture was glued on the crack face.

The treated region is about 890m^2 for the first time, i.e., from the right abutment to the location which is 26.75m at the upstream face and 10.0m at the downstream face left away from the 3# induced joint. The chemical material is a kind of epoxy resin named as LPL, the viscosity of LPL is relatively large.

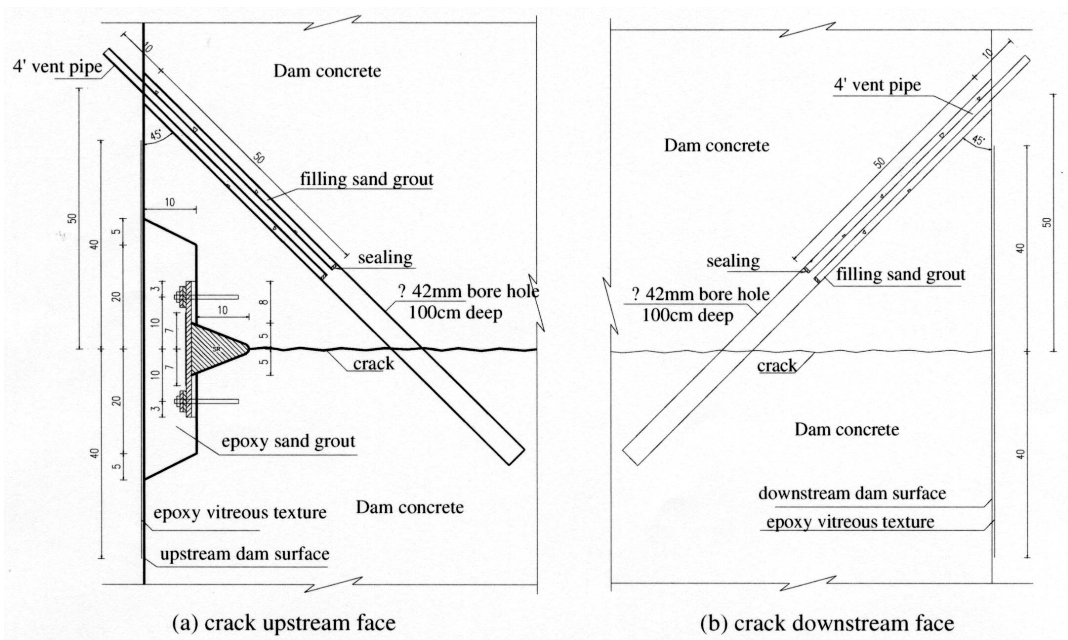
3.2 Crack treatment for the second time

Because the crack treatment for the first time did not gain the anticipative effects, and the crack further extended toward the crown cantilever, in order to ensure the seepage resistance and improve the tensile strength of the cracked RCC layer, and to control the crack development, the crack treatment for the second time was carried out. For the second time, the treatment measures included 150 drilled holes for chemical grouting, 19 drilled holes for installing anchor steel piles and 43 spare drilled holes for chemical grouting placed in the intact region between the crack front line and the reference plane.

The treatment working surface was at the dam top face of elevation 470.7m. Among those measures for the second time, there are 6 rows of drilled chemical grouting holes, the first row is 2.0m away from the dam upstream face, the second row is 2.0m away from the first row, other row distance is 3.0m, the distance between neighboring grouting holes is 3.0m, the grouting hole is 16m deep and reach the plane that is 40cm under the crack surface, the diameter of the grouting holes is 56mm; the anchor steel piles are 18.6m deep and used to restrict the cracking development; the grouting pipe system of the spare grouting holes were extended and collected in the observation chamber at El.478m gallery, and whether the chemical grouting through these spare holes are carried out or not lies in the cracking development status; vent pipes for grouting were placed on the crack face as shown in Figure 2; the chemical grouting type is non-circulation for the second time.

To meet the seepage resistance requirements of the arch dam, after beating away the improper filler for the first time, a longer and wider slot cut along the upstream cracking trace, then the SR plastic filler and the epoxy sand grout were carefully filled into the slot, a layer of epoxy vitreous texture was finally glued on the crack surface, as shown in Figure 2.

The treated region for the second time is about 1000m^2, i.e., from the right abutment to the location which is 19.55m at upstream face and 28.6m at downstream face right away from the reference plane. The chemical grouting material is a kind of epoxy resin named as ZH798, which was successfully applied in the foundation treatment of Longyangxia hydroelectric project, and is specially suitable for the crack or joint treatment with width less then 0.3mm. Epoxy san grout should be filled into the slot after driving away the water existed in the chemical grouting pipe sys-



(a) crack upstream face (b) crack downstream face

Figure 2. Treatment of the crack upstream and downstream face (unit: cm)

tems and in the cracking surface; chemical grouting can be carried out only after the strength of sand grout filled into drilled holes for anchor steel piles and the strength of the epoxy sand grout filled into the slot are more than 50 percents of the design strength.

4. Brief introduction of the crack treatment procedures for the second time

The ordinal construction procedure of the crack treatment for the second time is: tidy up the dam top face (El.470.7m) for drilling holes → drill holes for chemical grouting and installing anchor steel piles, at the same time, dig the slot along the upstream cracking trace→ tidy up the treatment working surface after drilling holes→ clean the drilled holes for installing anchor steel piles with high pressure water→ drive away the water existed in the drilled holes with high pressure wind → install the anchor steel piles, then fill and compact sand grout into the interspace of the holes →wait for 3 days after the sand grout was set, clean the drilled holes for chemical grouting with high pressure water and wind mixture →install grouting pipe system, including an injection pipe and a vent pipe for each grouting hole, then fill and compact sand grout into the interspace of the holes; seal the crack upstream and downstream boundary and installed vent pipes along the cracking trace→ wait for 3 days after the sand grout was set in grouting holes, clean the cracking surface with high pressure water →carry out water pressure test and determine the maximal grouting pressure → drive away the water in grouting pipe system and cracking surface by high pressure wind after sealing the crack boundary and installing 4" vent pipes on the crack upstream and downstream face as shown in figure 7→ drive away the water existed in the grouting pipe system and in the crack surface by pressing the acetone → drive away and reclaim the acetone by the high pressure wind → carry out the chemical grouting through grouting pipes on the dam top face→ carry out the chemical grouting through vent pipes on the crack upstream and downstream face → wait for several days after the slurry of epoxy resin ZH798 was set→ carry out water pressure test for checking the treatment effects and continue to place dam RCC above the treatment working face.

According to the test results, the initial viscosity is about 16mPa.s under the environment with temperature of 20°C, the viscosity is about 33 mPa.s ~55 mPa.s for the ordinary mix proportion after 12 hours. Thus, as long as guarantee the grouting pressure and the net grouting time, the viscosity of ZH798 is suitable for the horizontal crack along the placing RCC layer.

As we know, the grouting effects are not only related to crack width, viscosity of grout slurry, also related to grouting pressure, net grouting time and grouting equipments. For the crack with specified width, selecting chemical grouting material, grouting equipments, grouting pressure, net grouting time and criterion of ending grout in accordance with crack width, crack area and crack orientation, is very important to crack treatment. Moreover, grouting pressure, net grouting time and criterion of ending grout should be decided on basis of field tests and adjusted along with the actual site construction.

Flow control pump special for chemical grouting is adopt for the second time, the outage and the grouting pressure can be adjusted in accordance with the actual requirements, and it is especially fit for the fine large scale horizontal crack. An injection pipe and a vent pipe are placed for each grouting hole, and the injection pipe is connected to the grout pump with high pressure grouting pipes.

Field tests were carried out before grouting on a large scale to check the acetone pressure test and each procedure of the chemical grouting. Maximal grouting pressure during the chemical grouting is 1.2Mpa. Water, a mixture of water and acetone, acetone, a mixture of acetone and grouting slurry, till pure grouting material slurry outflow in turn from the downstream crack and the downstream checking hole for consolidation grouting in the right abutment, it verified that the effects of using acetone for driving away water and using grouting slurry for driving away acetone were very good.

5. Effects of crack treatments

5.1 Grout consumption of the crack

In order to ensure the workability of the grouting slurry, the viscosity of the slurry is checked for each 2~3hours. According to original recordings, most of the initial viscosity is varying from 8m.Pas to 12m.Pas, the final viscosity for each chemical grouting hole is less than 20~25m.Pas. During the whole process of chemical grouting, the grouting pressure is steady, the pressure value and the net grouting time met the design requirements.

As mentioned above, the treated region for the second time is about 1000m^2. The grout consumption of the grouting holes on the dam top face is 5176.2 liter, in which, the consumption of the grouting pipe system is 325.3 liter, the wasted consumption is 176.4 liter, consumption under grouting pressure (including leakage out of the crack) is 3674.5 liter, the net consumption forced into the crack by pressure is about 830.5 liter. Grout consumption through the vent pipes on the upstream crack is 32.7 liter, through the downstream vent pipes is 72 liter. The total net grout consumption is 935.2 liter, it is obvious that a lot of chemical grout material slurry was filled into the crack.

Concrete cores are bored from the dam top face at elevation 515m in August 2003. The longest concrete core is 10.57m. Some cores located in the treated region were used to check the effects of the crack treatment, the crack section was fully filled with grout material.

5.2 Bond performance between grout material ZH798 and RCC

In order to check the bond strength between ZH798 and RCC, in December 2002, RCC specimens with age of 90 days were glued with ZH798 slurry after the splitting tensile test. After curing the glued specimens for another age of 28 days or 90 days, bond strength of the glued specimens were tested. The splitting tensile strength of the glued specimens with age of 28 days are bout 2.29Mpa~2.56Mpa, that with age of 90 days is about 2.26Mpa~3.23Mpa. Judging from the splitting sections of glued RCC specimens, most of them ruptured from the intact RCC, but not from the glued splitting section, it indicates that concrete surfaces can be bonded well by ZH798.

Mention specially, indoor bond test is different from the actual chemical grouting, when boring the dam RCC cores located in the crack region, many cores broke at the cracked section, it implies that the bond strength of the actual chemical grouting is less than the tensile strength of intact dam RCC.

5.3 Results of water pressure test

Four holes were bored in the treated region for water pressure test in August 2003, the test results show that the water permeability of the test segment from EL.456.0m to EL.454.0m (the crack is located at EL. 455.1m) is 0.00312 Lu, 0.00284 Lu, 0.0053 Lu and 0.0366 Lu respectively, it meets the design requirements.

Before the first filling of the dam, water pressure tests were carried out for the spare grouting pipes in the observation chamber, and the results indicate that there is no further crack development. In addition, the treatment effects were verified by temporary retaining water during flood season in July 2003, first filling of dam since October 2003 and generating waterpower operation since May 2004, there is no leakage in the treated section, and the observation data indicate that the dam operates safely after initial filling.

6. Experiences and lessons

(1) It is inevitable to place RCC in high temperature season for most of the large scale RCC dam, buried cooling plastic pipe is one of effective temperature control measures, but the tie-ins of cooling pipes should be processed carefully to avoid the damage from the compact rollers, and then avoid the leakage out of the pipes. For example, tie-ins had better be connected by vulcanization.

(2) Select the right time to begin the RCC cooling in accordance with the setting time of RCC mix and the temperature variation of RCC. Thus, even if there is some damage in the cooling pipes, the induced leakage can not lead to large scale damage to the placing RCC layer as long as the strength of placed RCC is enough high at that time.

(3) There are many kinds of chemical grout materials for concrete crack treatment at present. Chemical grout material should be selected in accordance with the different characteristics of the actual crack. For example, the epoxy resin LPL was successfully applied in the vertical crack treatment of Three Gorges project, but failed in the horizontal crack treatment of Linhekou arch dam for the first time, because the high viscosity of the slurry, the little outage and the low grout pressure of the grouting pump are not appropriate to the fine large scale horizontal crack in Linhekou arch dam. However, ZH798 is successfully applied in the crack treatment for the second time, its viscosity can be adjusted and the outage of the grout pump is large, the grout pressure can be very high, all these are need for fine large scale horizontal crack. So, selecting grout material, grout equipment and grout parameters in accordance with the crack width, the cracking area, the crack orientation and other characteristics of the crack is vital to the crack treatment.

(4) Selecting appropriate grouting chance in accordance with the air temperature and the RCC temperature variation. Grouting in low temperature season with steady temperature stress in the dam and without crack development, can get more successful opportunity. However, it is usually difficult to make that decision when waiting for crack treatment chance prolongs the construction time and delays the initial generation time.

(5) Making the crack surface clean before chemical grouting by avoiding the pollution from waste water or other powder of drilling holes, it can improve the bond quality between chemical grout material and concrete. For example, chemical grouting may should be carried out before installing anchor steel piles. Thus, can reduce the pollution to the crack surface and improve the bond strength.

(6) Necessary instrumentation placed on the cracks will make for the observation in the operation period of the project, will be propitious to master the crack status in time, especially for the large scale crack in the key section as Linhekou arch dam.

Crack treatment is basically successful on basis of the current judgement. However, the final conclusion can be made only after long period operation. We paid out great costs for the crack treatment, but got a lot of experiences and lessons at the same time. We expect that it can provide experiences for other RCC dam construction and valuable suggestions for similar crack treatment.

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Applicability of Cement- Stabilized Mud Soil as Embankment Material

Shigeru Tani
Program Coordinator, National Institute for Rural Engineering, Japan

Shinji Fukushima
Engineer, Fujita Co. LTD, Japan

Akira Kitajima
Engineer, Fujita Technical Center, Japan

Koji Nishimoto
Engineer, Fujita Co. LTD, Kanagawa, Japan

Safety improvements

1. Outline

Along with the progress of measuring technologies in various fields and on the basis of the huge volume of accumulated data, design criteria have been improved to suit the situation in Japan where floods and earthquakes occur frequently. The current dam design criteria were formulated in accordance with the Government Ordinance for Structural Standard for River Administration Facilities, which was enacted in 1976. The number of dams with a height of over 15m completed in Japan more than 50 years ago exceeds 1,000. Most of these do not satisfy the current design criteria. Although no disastrous accidents such as dam failure have resulted from this, the safety of dams and the design flood discharge have been investigated and the necessary improvement works have been conducted in accordance with the latest design criteria.

2. Repair of earth dams

There are a number of older irrigation dams that were constructed more than 50 years ago. Among these, nearly 200,000 dams are small-scale earth dams (hereafter called "farm ponds") with a height of less than 15 m. In some urban areas, which have developed around farm ponds, an increasing volume of leakage threatens safety, demanding the immediate repair of nearly 20,000 farm ponds. In such areas, the following problems are likely to occur:

(1) It is difficult to obtain the embankment materials necessary to repair and reinforce the dam embankment , and the impervious material in particular, in the area around a farm pond,

(2) Although mud soil has accumulated in farm ponds and thus decreased the water storage capacity and caused water pollution, it is difficult to remove and dispose of the mud soil because of environmental concerns.

In order to solve these problems, a new improvement technique has been developed that utilizes mud soil in a farm pond as the embankment material. This method is called the "crushed and compacted embankment method," the mud soil is improved with a cement type additive to form stabilized soil, which is used as embankment soil to repair the embankment. This embankment soil made of stabilized soil is made by crushing cement-improved soil to a pre-determined maximum grain size, and then compacting it.

With the crushed and compacted embankment method, the stabilized soil which originally has the property of strain softening, behaves like a strain hardening material as a result of the crushing and compacting process and is resistant to cracking. So far, although cement-improved soil has a high strength, it breaks easily under small strain and so has not been used for constructing embankments as it would cause a difference of stiffness between the new embankment and old embankment.

Thus, as part of a government-industry joint project of the Ministry of Agriculture, Forestry, and Fisheries for research and development of new technologies entitled "Efficient improvement of earth dams", a method was developed to stabilized sediments in the bottom of the reservoir and use the product for repairing and strengthening earth dams (Figure 1). As shown in the flow chart in Figure.2, this method involves adding and mixing stabilization agents to sediments, curing the mixture for a certain period (ts*), breaking the hardened soil into grains of specified sizes using a breaking machine (Photograph 1), and immediately using the soil for embankment by roll compaction as ordinary banking materials. The method improves the deformation capacity of stabilized soil, which has low failure strain, by breaking and roll compacting the soil, and enables new embankment sections that have the same rigidity as the existing sections to be constructed.

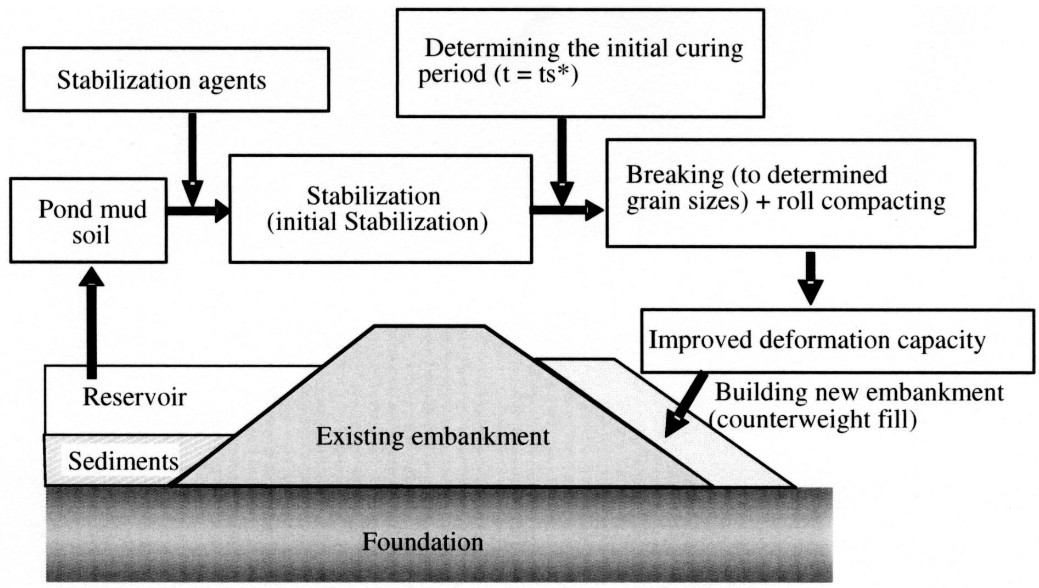


Figure 1 Use of stabilized sediments as banking materials

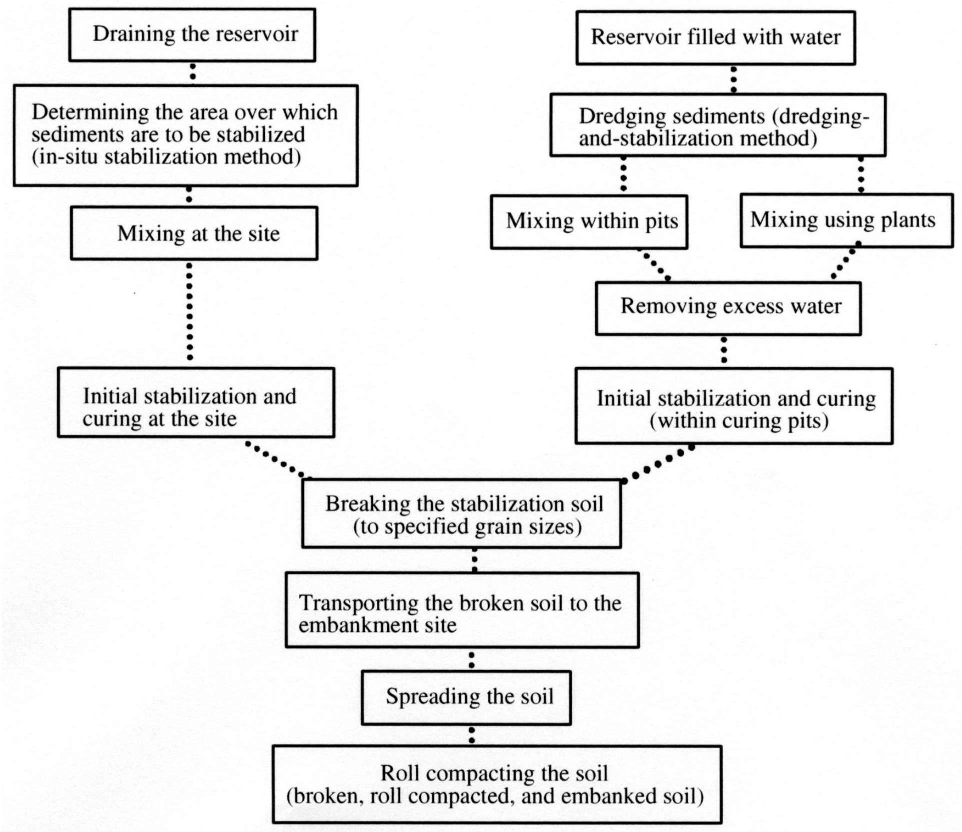


Figure 2 Process of the breaking and roll compacting banking method

The manifested strength depended on the days of curing (ts) and the grain sizes after breaking (Figure 4). Thus, banking materials having the required strength and deformation capacity equivalent to that of ordinary soil can be prepared by controlling the days of curing (ts) and grain size during breaking.Soil that has been stabilized, broken, and roll compacted using this method can be used for repairing and strengthening earth dams as 1) banking materials to widen the dams, 2) core materials, and 3) materials to raise the height of the existing earth dam to increase water storage capacity (Figure 5). The soil that has been prepared by mixing stabilization agents, curing for ts days to stabilized, breaking, roll compacting, and banking, showed larger failure strain than the initially stabilized soil and improved deformation capacity as shown by the stress-strain relationship derived by a triaxial compression test (Figure 3).

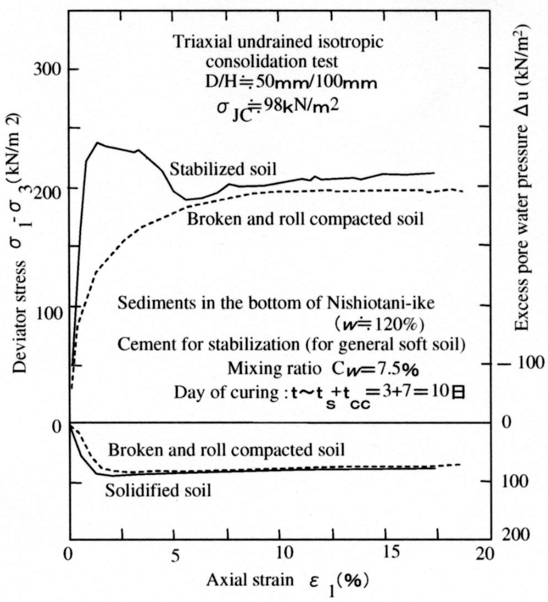


Fig. 3 Stress-strain curve

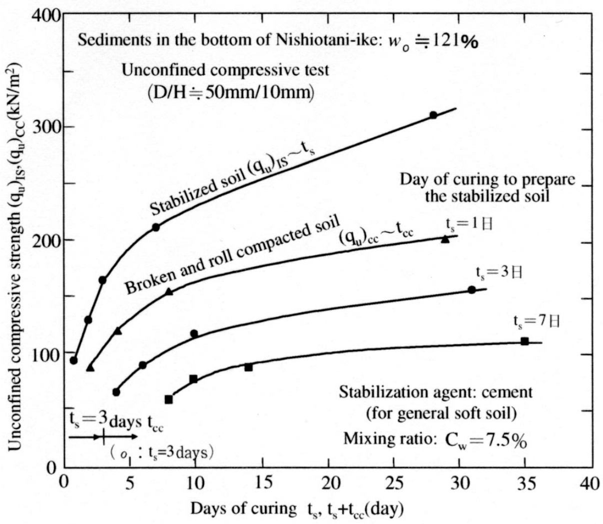


Fig. 4 Relationship between days of curing and unconfined compressive strength

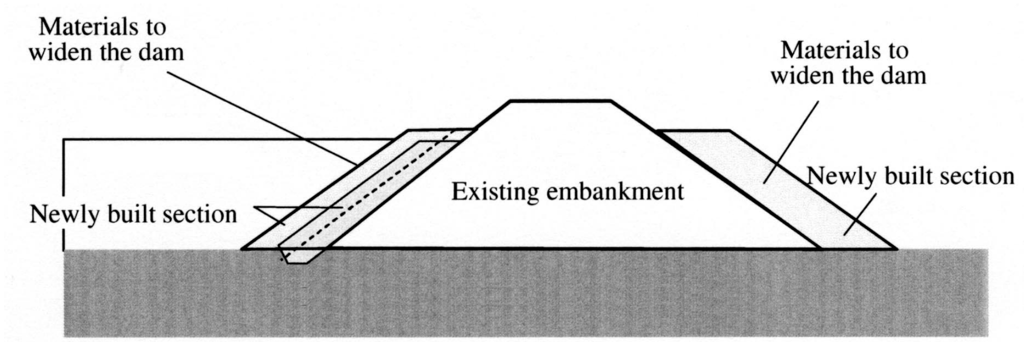


Fig. 5a Application of the breaking and roll compacting banking method (to widen a dam)

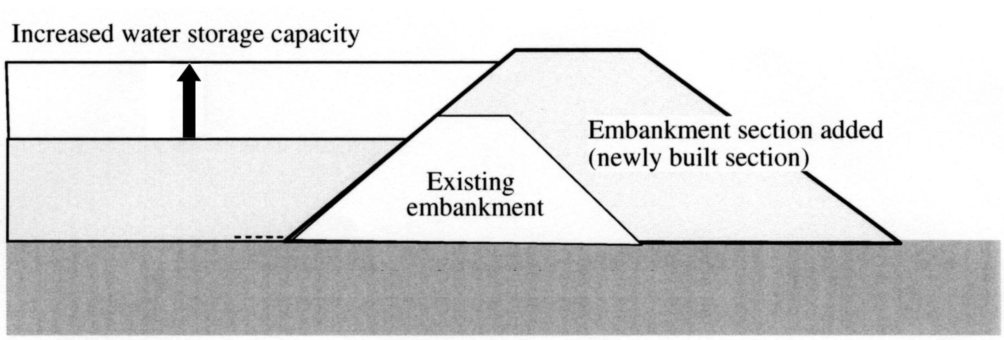


Fig. 5b Application of the breaking and roll compacting banking method (to raise a earth dam)

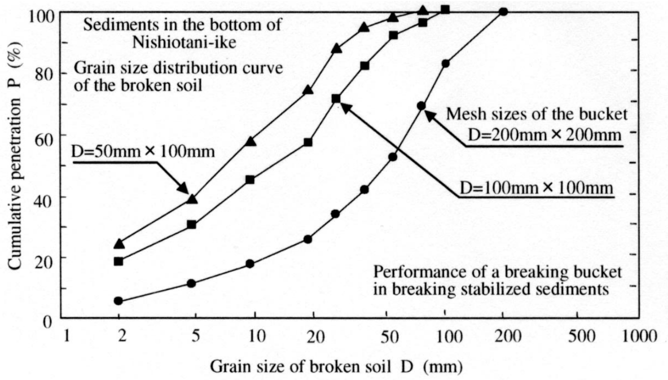


Fig. 6 Grain size of broken soil and grain size distribution


Photo 1 Breaking bucket

Some homogeneous earth dams that were designed and constructed at a time when the design criteria were not fully established cannot maintain stability against structural subsidence and liquefaction under earthquake conditions. Such structures need to be reinforced in accordance with current dam design criteria.

A typical method of reinforcing a dam embankment is to use counterweight fill. In an area where there has been an expansion of urbanization in the area around a dam and it is difficult to transport or utilize the required large volume of materials, the earthquake resistance of a dam can be enhanced with material extracted from a dam used as counterweight fill and the foundation soil reinforced. In this section, the situation of the Kitatani dam is introduced.

3. Repair work for the embankment of the Kitatani dam

The Kitatani dam in Mie Prefecture is formed by an old irrigation farmer's dam and its age is not known Figure 7 shows its location. Because of leakage from the slope toe and bottom trough as well as cross-sectional erosion of the embankment by aging, the Kitatani dam was evaluated as being unable to maintain stability during an earthquake. As a result, it was deemed necessary to construct a sloping core zone and completely repair the conduit and spillway in order to reinforce the dam embankment and prevent leakage. The specifications of the dam embankment before and after repairing are shown in Table 1.

The height of the Kitatani dam was 12 m and the slope was very steep. To ensure embankment stability and watertightness, the slope needed to be gentler

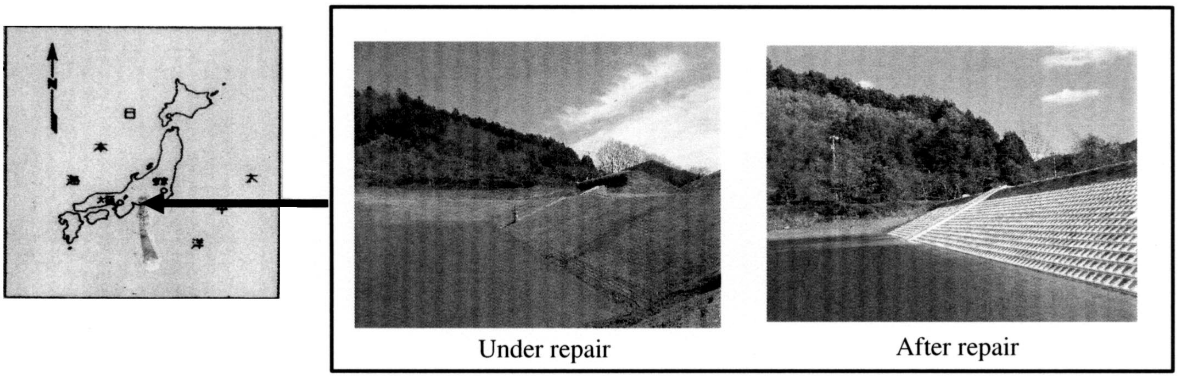


Figure 7 Location map

Table 1 Dimensions of the dam before and after renovation

Dimensions of the dam	Before repair	After repair
Dam type	Homogeneous type	Sloping impervious zone type
Dam height (m)	12.0	14.0
Crest length (m)	116.0	120.0
Dam volume (m^3)	25,500	36,000
Storage volume (m^3)	150,000	150,000
Volume of crushed and compacted soil (m^3)	—	16,000
Slope	1:1.6 (upstream average) 1:1.5 (downstream average)	1:2.1 (upstream average) 1:1.8 (downstream average)

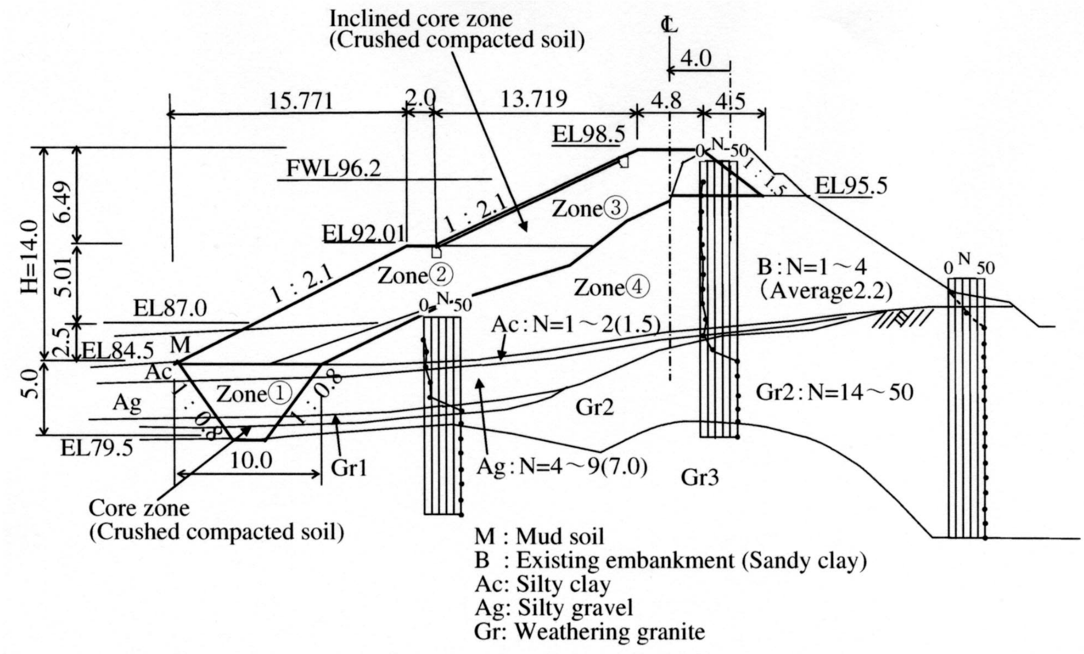


Figure 8 Typical cross-section of the dam

than about 1 in 3. However, an embankment with such a gentle slope would require a huge volume of embankment soil, which would likely severely reduce the storage capacity.

Consequently, the crushed and compacted embankment method was adopted in the Kitatani dam project. In repairing earth dams, the embankment soil requires strength to ensure the stability of the embankment and watertightness to store water. The merit of the crushed and compacted embankment method is that the embankment soil can be made artificially by controlling the cement content and the crushed grain size.

By adopting this embankment method, the following effects were observed in the Kitatani dam:

1) Because it was possible to prepare embankment soil having the pre-determined strength it was possible to repair the slope with a stepper gradient than would have been possible with ordinary materials as shown in the typical cross-section in Figure 8 thus reducing the volume of embankment materials needed and generating economic savings (the reduction of the slope from 1:3.0 to 1:2.1 reduced the quantity of material by over 40%).
2) Because all the embankment materials consisted of mud from the dam, there was no need to transport embankment soil in and out of the site and there was no reduction in water storage capacity, thereby avoiding damage to the environment, including the noise and exhaust emissions caused by dump trucks.

4. Conclusion remarks

Generally, in a farm pond with a dam height of 10 m or lower, the target strength of the crushed and compacted soil depends on the trafficability of construction machinery such as compaction rollers. However, in the Kitatani dam, the target strength was determined by that required for embankment stability since the embankment height was as high as 12m and the slope was as steep as 1 in 2.1. In most cases, there may be concern about the disparity in strength between the existing embankment and the inclined core zone newly constructed by the crushed and compacted soil, and this might lead to an undesirable consequence in the close contact of the two embankment zones. Accordingly, in the Kitatani dam, in order to reduce the effect of the disparity in strength between the existing and new embankments, the specifications of the required strength were altered in the top and bottom layers of the sloping core zone.

In order to reduce this disparity, the strength was determined to ensure the trafficability of construction machinery at the upper layer above the berm that is subjected to a larger seismic deformation, and at the layer below the berm where both the seismic deformation and the effect of disparity in strength are insignificant, and the strength was determined by the embankment stability. The quality of the crushed and compacted materials was controlled to investigate directly the strength rather than density in the ordinary soil embankment method, and tests were carried out on the embankment soil at volume intervals of about 1,500 m^3.

Rehabilitation of Hydropower Plants by Dam Tunneling

M. Kashiwayanagi
Senior manager, Electric Power Development Co. Ltd., Japan

Y. Mizuhashi
Manager, Electric Power Development Co. Ltd., Japan

1. Introduction

Development of high dams in Japan started as construction of gravity dams in 1890s with the spread of Portland cement. Now about 2600 dams of which height is more than 15m are under operation. These dams have supported the economic basis of Japan by utilization of stored water and have contributed to protecting human life and social property by flood control. From the above viewpoints, it seems that the Japanese public accepts benefit of dams.

Meanwhile, few promising dam sites are left since almost all dam sites, which are economically feasible and environmentally friendly, have been developed already in not so large national land of Japan. Therefore, it got to be more difficult to promote a development plan of a dam than before even though the plan is approved as the optimum development method after many-faced evaluation.

In comparison with totally new development plans, rehabilitation plans that make use of existing dams have advantages such as those cited below, and therefore it is expected that the demand for their positive implementation will increase in Japan:

1) mitigating the impact on the environment,
2) meeting immediate developmental requirements due to a shorter execution period,
3) realizing a more economical exploitation, and
4) making the best use of unused energy.

In addition to re-development plan by improvement of a dam and a supplement of facilities, a wide area re-development plan which produces a new function by coordinating plural dams and reservoirs has been put into practice in recent years.

From the design aspects of dams, Dr. Mononobe devised seismic coefficient method in 1925, which started a dam design system and the Japan Commission on Large Dams practically completed it by establishing the Design Standard of Dams in 1957. Accordingly, some of the existing dams which were constructed before the said standards were established have less resistance to earthquake and insufficient spillway capacity compared with recent ones. Safety of these dams shall be improved since most of them have worked efficiently to date and are expected to maintain the function in the years ahead. Therefore, some of those dams are under reinforcement works in proportion to the degree of importance considering characteristics of each dam site.

These major re-development plans are summarized as shown in Table-1. In some cases, however, innovative engineering methods need to be introduced to maximize these effects, and this is the urgent issue to be addressed.

Among all redevelopment plans, some examples of hydropower plants that have been rehabilitated by dam tunneling are introduced in this report.

Because only a few cases are known in Japan in which the concrete of an existing gravity dam has been excavated or modified after completion as of the beginning of 1990's, the safety and construction methodology of a dam during and after execution has become a hot subject. Meanwhile, as the character of Japan's consumption of electricity is to peak at certain time periods, there was an urgent demand to provide peak power. To address this problem, the plan for the extension to the Okutadami hydropower station were carried out to produce peak power using the existing dam facilities, as stated below. And the plan for the Akiha No. 3 hydropower station was carried out to make good use of reservoir water spilt through a spillway. Figure-1 and Table-2 show the locations and outlines of both generation plans. The design and application of dam tunneling will also be discussed in the next section.

2. Outline of dam tunneling method

There are two important points to keep in mind in designing a dam tunneling methodology for an existing concrete dam; 1) the stability of the dam and, 2) the stress around a tunnel.

As for dam stability, it needs to be confirmed whether the monolith to be tunneled satisfies the design requirements (e.g. dam stress, safety factor for sliding and foundation stress) to withstand various loads such as the hydrostatic pressure of the reservoir and seismic loads.

Besides, the stress around a tunnel is influenced by the configuration of excavation and its local load. Although this phenomenon can be modeled simply as stress concentration in an infinite perforated panel, its detailed evaluation of the stress is possible with a highly accurate FEM analysis. On the basis of these

Table-1 Rehabilitation projects of dams in Japan – Recent applications

Projects	Type	Description	Features
Sannokai dam	A	Supplemental water supply for the agricultural use by increasing the dam height	Dam type: Rock fill Previous dam height: 37m Completed in 1952 Modified dam height: 61.5m Completed in 2001
Mitaka dam	A	Supplemental water supply for the agricultural use by increasing the dam height	Dam type: Concrete gravity dam Previous dam height: 32.6m Completed in 1944 Modified dam height: 44m Completed in 2002
Taishakugawa dam	A	Supplemental hydropower by developing the unutilized energy with the enhancement of the capacity of flood discharge and seismic stability	Dam type: Concrete gravity dam Previous capacity: 4.4MW, 720m^3/sec(Flood) Modified capacity: 13.4MW, 1610m^3/sec
Okinawa integrated water supply project	A	Comprehensive operation of existing dams for the stable water supply in the main island in Okinawa	Five (5) dams and connection waterway systems are involved in this Project.
Comprehensive operation project in Awaji island	A,B	Enhancement of water supply and flood mitigation by cooperation of dams	Five (5) dams and connection waterway systems are involved in this Project.
Nagasaki flood mitigation project	B	Enhancement for the flood discharge by the cooperation of existing dams for water supply	Three (3) exiting dams and two (2) new dams are involved in the Project. Completed in 2005
Yamagichi reservoir	C	Enhancement of seismic stability by embankment on the existing dam body	Dam type: Earthfill dam Previous dam height and volume: 35m, 1.4MCM Completed in 1934 Modified dam volume: 2.37 MCM Completed in 2002
Ohmatazawa dam	C	Enhancement of seismic stability	Dam type: Concrete gravity dams Previous dam height: 18.7m Downstream slope: 1 to 0.6 Completed in 1917 Modified dam slope: 1 to 0.94 Completed in 2002
Yamakura dam	C	Enhancement of seismic stability to reduce the possibility of liquefaction during earthquakes by arranging the Soil Mixing Wall in the dam	Dam type: Earthfill dams Previous dam height: 22.5m Completed in 1964 Modified dam slope: 1 to 0.94 Completed in 2002

Type description
A: Enhancement of original function of dams
B: Utilization to new purpose
C: Enhancement of safety of dams

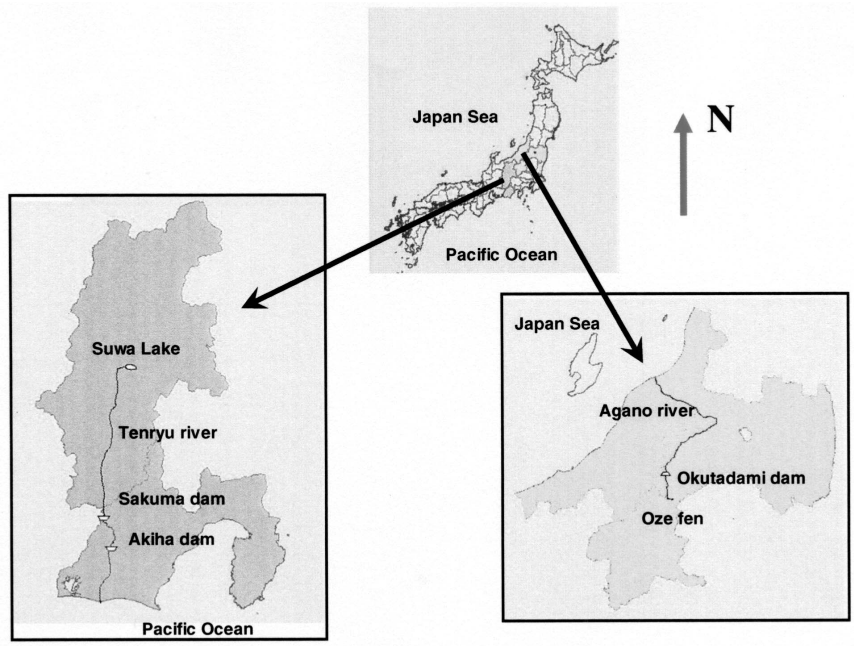


Figure-1 Location map

Table-2 Features of the rehabilitation projects

Name of Power plant	Akiha hydropower plant No.3	Okutadami extension hydropower plant
Year of completion	1991	2003
River	Tenryu	Tadami
Drainage area	4490 km^2	595 km^2
Reservoir	Akiha reservoir (Existing)	Okutadami reservoir (Existing)
	Effective capacity: 7.75 MCM	Effective capacity: 458 MCM
Maximum discharge	116 m^3/sec	138 m^3/sec
Maximum head	47.1 m	164.2 m
Maximum capacity	46.9 MW	200 MW
Annual output	Increase : 348 x 10^6kwh Decrease : 252 x 10^6kwh	For Peak electricity
Dam	Akiha dam (Existing)	Okutadami dam (Existing)
	Height : 89 m	Height : 157 m
	Crest length : 273.4 m	Crest length : 480 m
Dam tunneling		
Section	Circle with a flat invert	Square with fillets
Length	21 m	32 m
Diameter	6.5 m	6.2 m
Volume	720 m^3 (including the shaft for the intake gate)	1200 m^3
Construction machine		
Drilling (Hydraulic Drifter)	60 mm bit in diameter	102 mm bit in diameter
Tunneling (Breaker)	2300 kg class	2000 kg class
Construction rate	Approx. 1m/day	Approx. 0.6m/harf day

methods, it has to be confirmed that the stress around a tunnel does not exceed the tensile strength of the existing dam concrete but maintains an appropriate safety factor.

The following two points should be taken into account in executing a dam tunneling method:

1) prevention of cracking in concrete beyond the excavated area,
2) reduction of vibration and impact on the dam body kept to a minimum during execution.

A useful reference for achieving these results is the non-blasting tunneling method, which has been developed for the construction of tunnels under the ground with thin overburden in urban area. This method includes two types of techniques; the single-step approach based on machinery excavation, and the two-phase approach in which the outer edge of the excavated section is first drilled and then the internal section crushed. The latter approach is further divided into three subsets according to the machines used to drill the circumference. Table-3 shows these methods one by one.

In the selection of the construction methodology, the higher consideration should be provided in the hardness of the aggregate of the dam concrete, which could affect much on the efficiency of the construction, but less in the strength of the dam concrete itself.

The records of the dam tunneling in Japan are summed up in Table-4, showing in which the smaller scale of tunneling for the installation of discharge conduits has been predominant. In this report, the case histories of larger scales of tunneling for the extension of the hydropower plants are introduced hereinafter.

3. Akiha No. 3 hydropower station

3-1. Outline of the plan

The existing Akiha reservoir (35 MCM in total volume) on the Akiha dam (PG, completed in 1958, dam height: 89 m, and dam volume: 515,000 m^3), which is located in the middle reaches of the Tenryu River in central Japan, is used for power generation by the Akiha No. 1 hydropower station (completed in 1958, output: 45.3 MW, maximum discharge: 110 m^3/sec) and the Akiha No. 2 hydropower station (completed in 1958, output: 34.9 MW, maximum discharge: 110 m3/sec). Due to the affluent flow of the Tenryu River, this area is usually stricken by the discharge through the spillways nearly 100 days a year even though much water usage is carried out through power generation. Thus, there has been a demand for greater efficiency in the use of the reservoir. The Akiha No. 3 hydropower station plan has been coordinated to produce the maximum energy by reviewing the operation efficiency of the existing No. 1 and No. 2 hydropower stations and by taking advantage of unused energy. Figure-2 shows the discharge allocation of the reservoir at original state and after completion.

The Akiha No. 3 hydropower station utilizes the intake facility of the existing No. 1 hydropower station situated on the right bank of the Akiha dam[1)-4)]. From an intake that was built by excavation and modifying the dam body, water is consumed at a maximum volume of 116 m^3/sec and is conducted to the No. 3 hydropower station that has been newly constructed immediately below the right bank of the dam by way of a 70 m-long penstock. Power is generated by means of an effective head of 47.1 m, with a maximum output of 46.9 MW and an annual power generation of 96x10^6 kWh. The water is then discharged to the existing Funagira reservoir from the tailrace tunnel, nearly 3.6 km in length and 6.5 m in inner diameter.

Some sections of the intake and the penstock have been installed by tunneling into the existing concrete of the dam. The tunnel consists of a tubular hole 6.5 m in diameter (area of cut: 34 m^2) and 21 m in length that facilitates the installation of the penstock, and a rectan-

Table-3 Summary of methodologies of dam tunneling

Construction methodology	Outline	Description
Single-step approach	Machinery excavation	TBM, Boring machine, excavation machine are applicable
Two-phase approach	Drilling and breaking	Making consecutive slots by single drilling or multi-slot drilling at the peripheral of the tunnel, followed by machinery breaking
	Cutting and breaking	Cutting by water jet or wire saw followed by the machinery breaking
	High density drilling with small diameter, Drilling with large diameter	High density drilling at the internal of tunnel followed by the peripheral drilling with large diameter boring machine

Table-4 Summary of dam tunneling for the rehabilitation of dams
(Other cases more than 10 are found as of 2005 in Japan.)

Name of dams	Owner	Type of dams	Height	Dam tunneling		Purpose	Construction	
				Diameter	Length		Completion	Method
			m	m	m			
Fujiigawa	Ibaragi Pref.	PG	37.5	8.6 (Enlarge)	20.0	A	1977	Drilling(Φ34mm), Machinery excavation
Yoroibata	MLIT*1)	PG	58.5	4.4	29.0	A	1989	Machinery excavation
Akiha	J-Power	PG	89.0	6.5	21.0	B	1991	Slot Drilling Method (φ 60mm, Milti-bits of 5), Machinery excavation
Kakkomi	J-Power	PG	34.0	1.8	4.5	A	1992	High pressure water jet, Static blasting
Naiba	Kagawa Pref.	PG	50.0	2.5	30.0	A	1993	Machinery excavation
Kuki	J-Power	PG	28.0	1.0	3.0	A	1995	Wire saw
Futagawa	Wakayama Pref.	PG	67.4	1.5	23.3	A	1998	Machinery excavation
Nanairo	J-Power	VA	61.0	1.7	5.7	A	2000	Wire saw
Haginari	Akita Pref.	PG	61.0	2.3	25.0	A	2000	Machinery excavation
Ikari	MLIT*1)	PG	112.0	5.0	101.7 (49.1,48.6)	A	2003	Machinery excavation
Okutadami	J-Power	PG	157.0	6.2	35.0	B	2003	Slot Drilling Method (φ 102mm), Machinery excavation
Mitaka	Hiroshima Pref.	PG	32.7	2.4	13.0	A, B	2004	Machinery excavation

*1) Ministry of Land, Instrastructure and Transport
Purpose of the dam tunneling A: Discharge for downstream area B: Discharge for Power generation

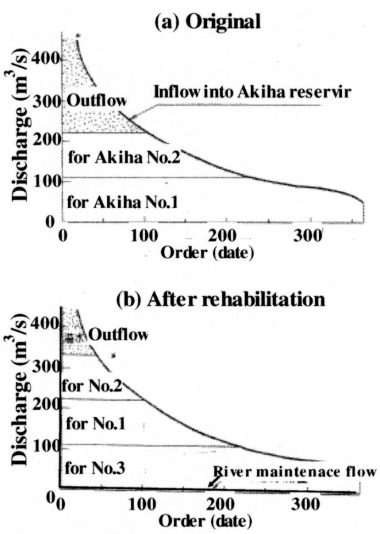


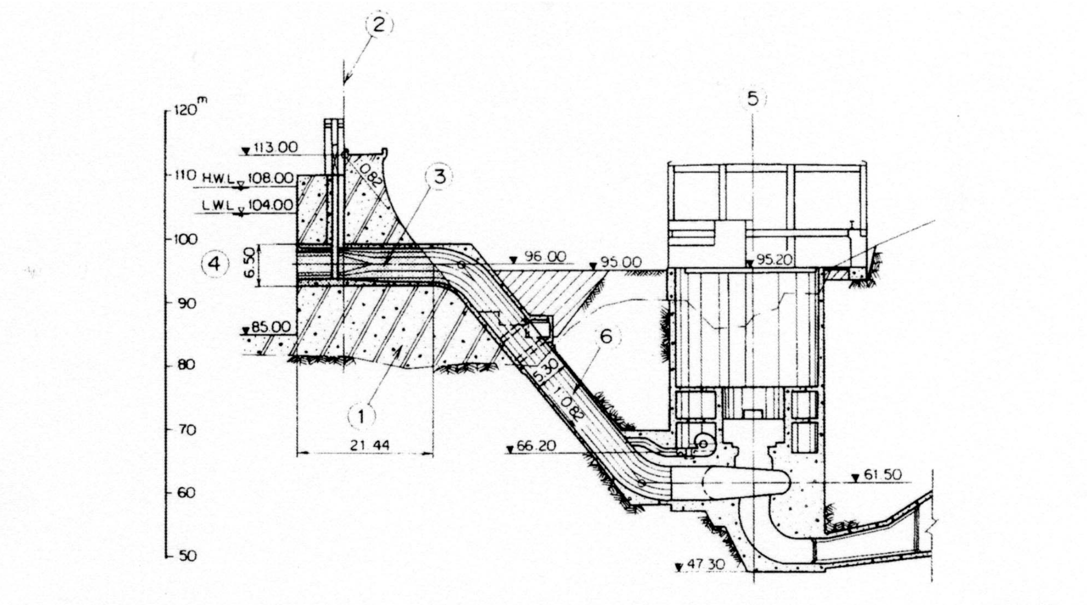
Figure-2 Discharge allocation for each hydropower plant

gular shaft 2.5 m x 7 m in cross-section and 11 m in depth for the intake gate. The total volume of excavation is about 720 m^3. Figure-3 depicts the structure.

3-2. Designing requirements

Regarding dam stability, such elements as dam stress, safety factor for sliding and foundation stress were examined under various conditions with water level and seismic intensity as parameters. As a result, it was confirmed that the safety requirements had been satisfactorily fulfilled.

To evaluate the stress around the tunnel, a profile of the dam that crosses the tunnel axis at right angles was regarded as a two-dimensional perforated plate, where the plate was assumed to be subject to dam stress induced by water pressure, seismic inertia force, dynamic water pressure and self weight of the dam. As a result, the maximum tensile stress of the upper area around the tunnel was calculated as 0.81 MPa under normal conditions without seismic effect. This value was sufficiently smaller than the tensile



① Existing Dam, ② Dam axis, ③ Dam tunneling, ④ Intake, ⑤ Powerhouse, ⑥ Penstock

Figure-3 Profile of the Akiha No.3 power plant

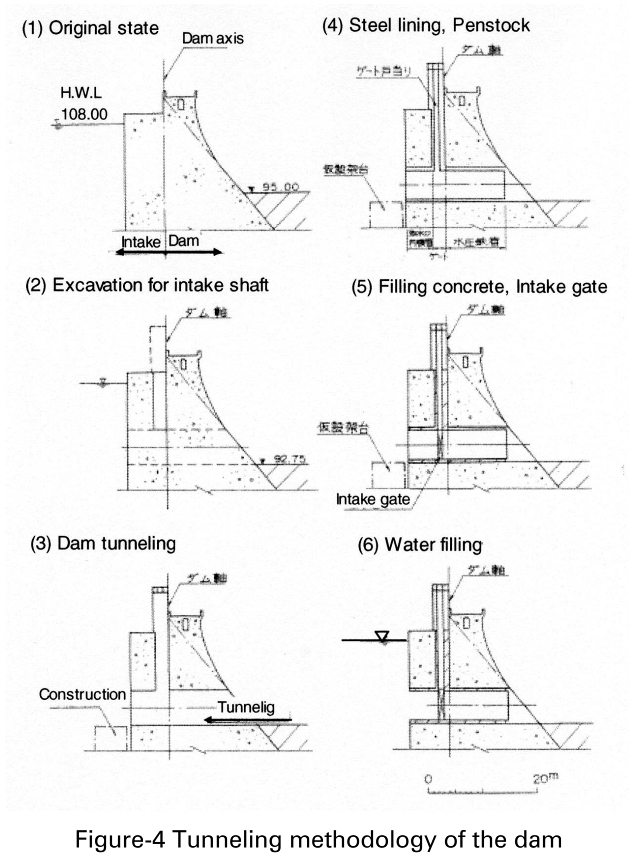


Figure-4 Tunneling methodology of the dam

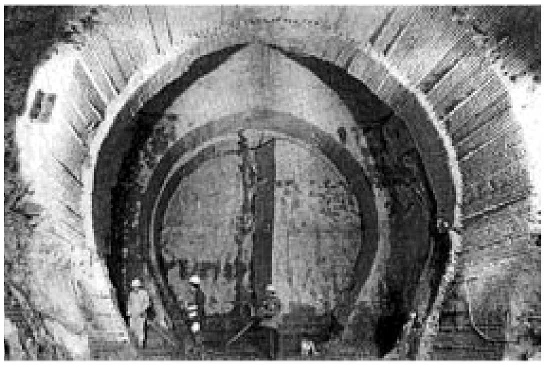
strength of 2.2 MP, which is estimated by the 21.6 MPa of the uniaxial compressive strength of the core (190 mm in bore diameter) extracted from the dam body and, suggesting a safe degree of stress after tunneling.

3-3. Construction method

Because there had been few precedents for this particular tunneling method, the slot drilling technique (SD Technique) was adopted by referring to the non-blasting tunneling method for underground tunnels. The process of the SD Technique is shown below and also in Figure-4. The total length to be drilled in the dam was approximately 2,260 m.

1) Consecutive holes are drilled in the outer periphery and a few strips in the inner section of the drilled area.
2) Crushing is performed by means of a hydraulic breaker (2.3 t class).
3) Crushed slag is removed.

a) Drilling machine



b) After tunneling

Photos-1 Dam tunneling of Akiha dam

During the crushing operation, vibration control was conducted to prevent damage to the dam concrete around the tunnel. As a control value against vibration-induced stress $\sigma = \rho V v$(where, σ: vibration-induced stress, ρ: concrete unit weight, V: the elastic wave velocity of concrete and v: vibration velocity), the stress allowable on the concrete was fixed at 0.22 MPa and the vibration velocity was set below 2.0 kine (cm/sec) by taking a safety factor of 10 into account.

The works of drilling into the dam concrete and of installing the intake gate and the penstock within the dam body had to be conducted below the low water level of the reservoir. Therefore, while these works were being carried out and the water was being discharged, the No. 1 hydropower station was suspended and its forebay closed with stop logs. The shutdown period of the No. 1 hydropower station was set at a minimum duration of 90 days. In the meantime, a series of working processes had to be completed: tunneling into the dam body, installing the intake gate and penstock and laying the filling concrete. These steps required careful preparation on the part of the administrator and precise construction management. Photos-1 was taken during the construction.

3-4. Comparison between observed values and analytical values of stress

As a part of the construction management, the changes in stress induced by dam tunneling were monitored with a stress meter that had been placed around the tunnel before commencement of the tunneling operation. At the same time, a three-dimensional FEM tunneling simulation was carried out for comparison with the observed values.

The stress on the existing dam body was also examined in the overcoring method. Figure-5 illustrates the location of a stress meter, the FEM model and both actual and analytical results.

Based on these results, a confining pressure of 0.3 MPa from the adjacent monolith was estimated, which allowed the tunneling-induced stress transient to be reproduced more appropriately. This kind of stress increment is not regarded as the tensile stress that was estimated in the early designing stages but as stress on the compressive side, since it is attributed to the confining pressure of the adjacent monolith.

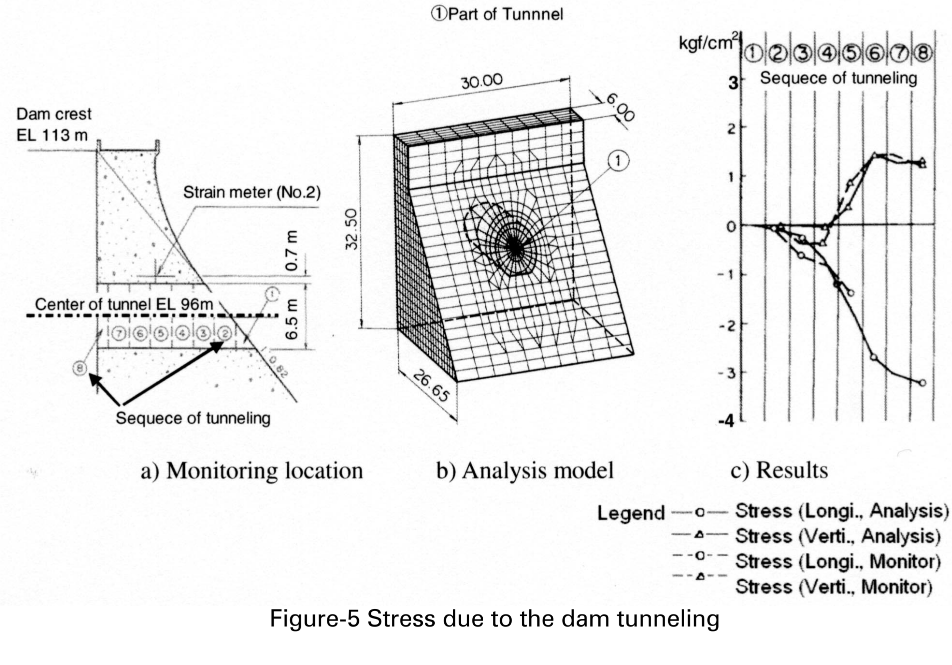


Figure-5 Stress due to the dam tunneling

4. Extension to the Okutadami hydropower station

4-1. Outline of the plan

The Okutadami hydropower station extension plan makes use of Japan's largest reservoir (600 MCM in gross reservoir capacity) on the existing Okutadami dam (PG, completed in 1960, dam height: 157 m and dam volume: 1,636,000 m^3), located in the uppermost stream of the Tadami River in central Japan. The plan was to develop a maximum output of 200 MW with the addition of the new generation facility in order to supply peak power by consuming 138 m^3/sec of water at maximum (work on the operations started in 2003)[5)-9)]. The existing Okutadami power plant takes water from the Okutadami reservoir at a maximum rate of 249 m^3/sec to generate up to 360 MW of power.

In this construction plan, as per policy, special attention was given to conservation of the natural environment, as the area is surrounded by unspoiled nature and is the natural habitat of the golden eagle, a rare bird registered in the Red Data Book. With the aim of protecting the golden eagle, the construction period was restricted to four months from July to October so as to avoid disturbing the nesting season.

4-2. Designing requirements

The intake for the extension hydropower station was installed by placing a bulkhead at the upstream surface of the existing Okutadami dam and drilling into the dam body. Because the normal operations of the existing reservoir were not interrupted during the period of construction, a steel-concrete semicircular bulkhead was constructed to withstand high water pressure at a depth of about 50 m. The structure is depicted in Figure-6.

Regarding the penstock, a volume of 1,200 m^3, 6.2 m in width and 6.2 m in height was excavated from the existing Okutadami dam body, and into this, a 32 m long penstock with a diameter of 5 m was laid. Its profile is shown in Figure-7.

Although a circular or horseshoe shape might be commonly adopted for the configuration of the tunnel, a tangential square cross section was employed in this project to reduce the stress induced by stress concentration in the existing dam. By adopting a square cross section, the flat bottom and ceiling surfaces make it easier to perform drilling operations and place the filling concrete. The validity of this structure was confirmed by a three-dimensional linear elasticity FEM analysis, in which the stress under various load conditions (e.g. seismic intensity and reservoir water level) was proved to be less than the strength of the dam concrete.

4-3. Bulkhead

The plan called for placing a bulkhead on the upstream side of the dam so that the construction work could be conducted under dry condition in a restricted execution period without interrupting the existing reservoir operations. This bulkhead had to be capable of withstanding high water pressure at a depth of 50 m in order for it to be engaged with the intake and penstock installed nearly 35 m below the full water surface. There was also a requirement for the structure to be built and removed in a short period of time.

Consequently, a steel-concrete semicircular bulkhead with a radius of approximately 8m was adopted to allow the axial force to take the water pressure. The structure was equipped with 13 box-type steel sheet piles with an arc length of nearly 2 m around the circumference and four piles in the vertical direction. The box-type steel sheet pile was filled with concrete underwater.

This structure enabled the construction of a relatively thin bulkhead with a thickness of 0.65 m that is capable of withstanding water pressure at a depth of

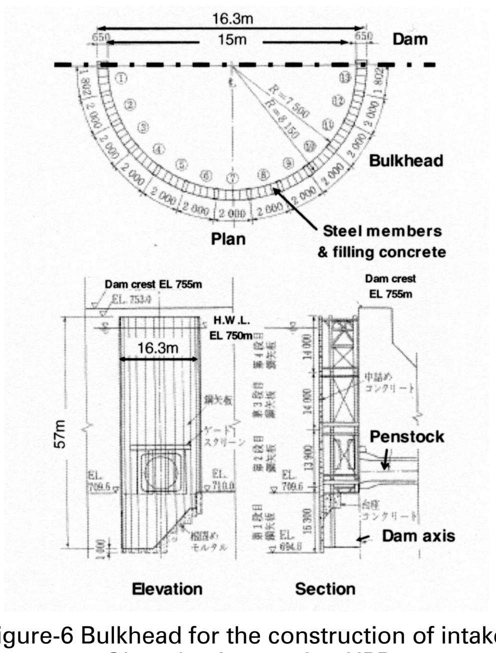


Figure-6 Bulkhead for the construction of intake, Okutadami extension HPP

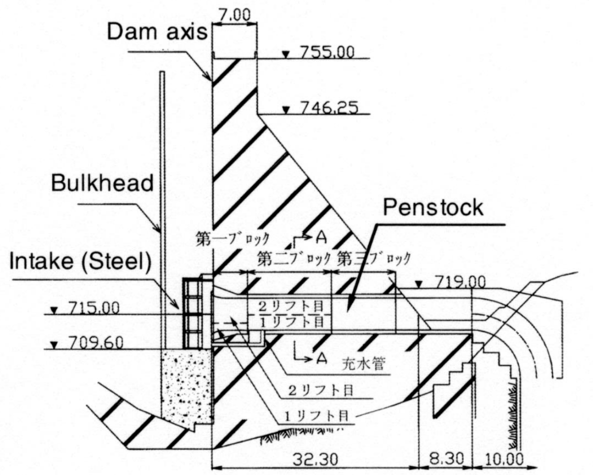


Figure-7 Profile of intake and penstock of the Okutadami extension HPP

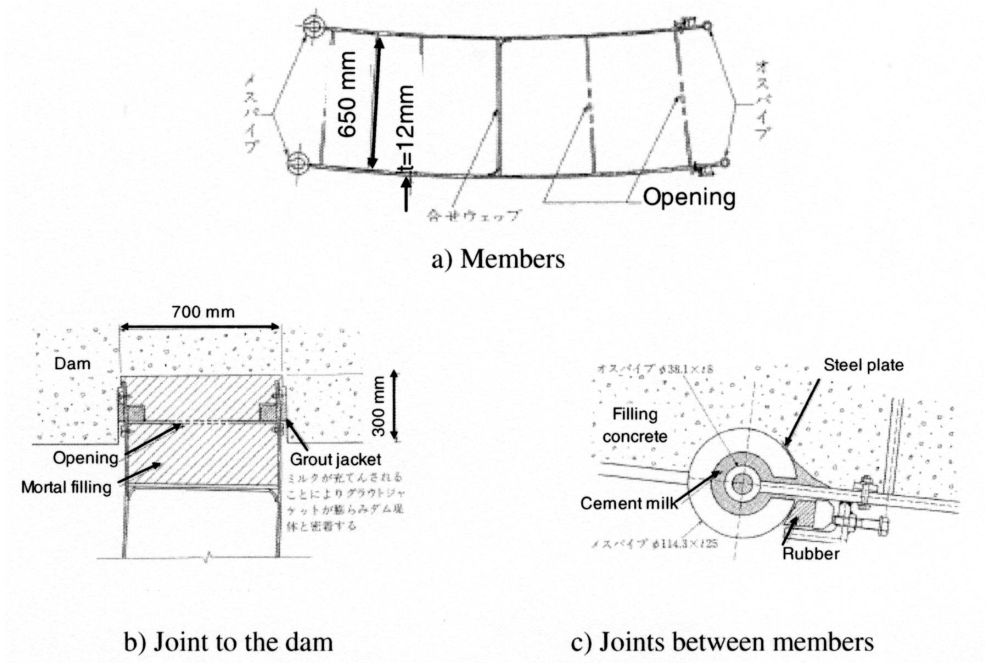


Figure-8 Details of bulkhead, Okutadami extension HPP

50 m. Figure-6 illustrates the structure of the bulkhead.

The possibility of leakage was considered at three sections: the bulkhead bottom, the joint to the dam and the joint sections of the bulkhead. To enhance the sealing capability, the following measures were taken.

(1) Bulkhead bottom

Seating concrete was laid up to a height of nearly 13 m between the lake bottom and the newly installed intake mouth. By connecting the seating concrete and the bulkhead, the sealing ability at the bottom of the bulkhead was reinforced. For the seating concrete to perform correctly underwater, anti-cracking measures had to be taken. By referring to the preliminary analytical result, precooling was carried out.

(2) Joint to the dam

The joint section to the dam was excavated nearly 0.3 m from the upstream side, where a steel sheet pile was installed to fill mortar between the pile and the dam. With the aim of enhancing the sealing ability of the joint, a grout jacket was used. The grout jacket expands under the pressure of its own weight when the milk of the grout is filled in, functioning as a packer. Figure-8 details the structure.

(3) Joint of the bulkhead

The detailed diagram of the joint of the bulkhead is shown in Figure-8.

4-4. Dam tunneling method

In determining the dam tunneling method for the Okutadami dam, reference was made to the operation record of the Akiha No. 3 hydropower station introduced in Chapter 3. A two-phase approach was chosen in order to reduce the execution period and to keep any impact on the existing dam concrete to a minimum. This method, consisting of slot drilling and breaker crushing, is similar to the one employed for the Akiha No. 3 hydropower station. Slots were drilled in such a manner that they continued around the outer edge of the tunnel section. These slots contributed to mitigating any vibration transferred to the dam and increased crushing efficiency through the creation of a free face during breaker crushing. To further enhance breaker-crushing efficiency, the coreless slots were combined. These can be seen in Figure-9.

4-5. Execution results

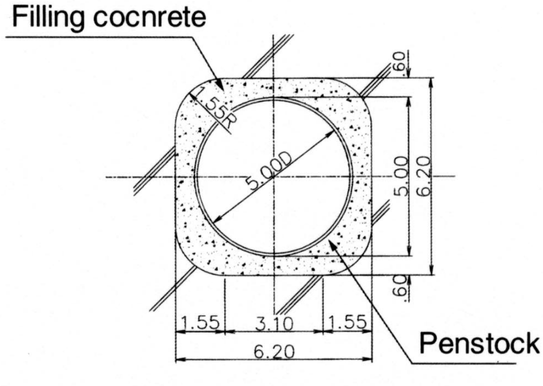
The strength of the aggregate used for the dam body was much higher than might have been expected, as the length of slot drilling in one cycle was 0.6m.

The degree of vibration transferred to the dam was monitored during drilling operations. As in the case of the Akiha No. 3 hydropower station, a vibration velocity of 2 kine (cm/sec) was set as the control criterion with the consideration of vibration-induced stress. The vibration measurements under construction were 0.2-1.1 kine at the time of slot drilling and 0.6-2.0 kine at breaker crushing, both registering within range of the control criterion.

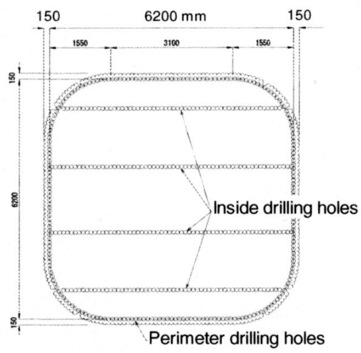
Photos-2 was taken during the execution.

The excavated section was divided into three blocks along a transverse direction to facilitate concrete filling. The two blocks on the upstream side were further divided into two lifts. The proportion of filling concrete is shown in Table-5.

The concrete was laid through a pump. Milk of cement was applied in places where the concrete was not sufficiently filled so that the dam concrete and the filling concrete would consolidate firmly.



a) Arrangement of penstock



c) Drilling arrangement

Figure-9 Section of dam tunneling of Okutadami dam

a) Bulkhead

b) Drilling machine

c) After tunneling

Photos-2 Construction of dam tunneling in Okutadami extension HPP

Table-5 Filling concrete around penstock, Okutadami extension HPP (Unit: kg)

Water	Cement	Flyash	Fine aggregate	Coarse aggregate	Admixture	
					Water reduce	Viscosity increase
180	328	231	721	828	7.27	0.18

5. Conclusion

This report introduces some technical examples for adding a new function to a dam that apply the dam tunneling method. Beside this method, several other examples have been reported in Japan, and they are evaluated as nearly-established technologies.

Recently, greater flood control capability has been required of some dams. In such cases, this method is effective from the perspectives of economical efficiency and environmental restrictions.

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Application of Cement Sand and Gravel (CSG) Dam in China and Lab Test on CSG Material Dissolution

Jia Jinsheng[1] **Ma Fengling**[1] **Li Xinyu**[2] **Chen Zuping** **Wang Xiaoqiang**

[1] *Prof. & Senior Eng., China Institute of Water Resources and Hydropower Research, Beijing, China*

[2] *Candidate Dr., Dept. of Structures & Materials, China Institute of Water Resources and Hydropower Research, Beijing, China*

Abstract

Experts in China have done some research work on CSG (Cement Sand and Gravel) Dam, but there is no dam built with CSG up to now. Based on the advantages of CSG in foundation requirement, quick speed in construction, low cost and etc., the first CSG cofferdam in China, which is 16.3m high, crest thickness is 4m, slope ratio is 1:0.4 on both sides, has been built in 2004 within 13 days and cost saving is about 25% . In order to investigate the durability of the CSG material, analyses have been carried out on dissolution performance.

Key Words: *Cement Sand and Gravel Dam (CSG), Material dissolution, Cofferdam*

1. Brief introduction

Jiemian hydropower station, with 126m high CFRD and 300 MW capacity,is located in Youxi county, Fujian province of China. The upstream slope of the CFRD is 1:1.4 and the downstream slope is 1:1.35. Because there are adequate and well-gradation sand and gravel near the dam site along the river, Jiemian downstream cofferdam was finally decided to adopt CSG dam type under the joint efforts of the author,clients, designer and contractor,based on technical progress showed in Table 1 and references in the world[1-9].

There are two loading cases for CSG cofferdam of Jiemian CFRD. One is used as downstream cofferdam and access road during construction , the other is used as weir for measuring leakage through CFRD during operation. Two states are shown in Fig. 1. The rockfill thrust on the weir has been taken into consideration.

The CSG cofferdam has a maximum height of 16.3m and longest crest of 49.5m, with a maximum bottom width of 17m and crest width of 4m. Its upstream and downstream slopes designed are 1:0.4 and the real slope is 1:0.6 after construction. The total volume of CSG material is 4500m^3. Analyses on stability and stress under two loading cases have been made.

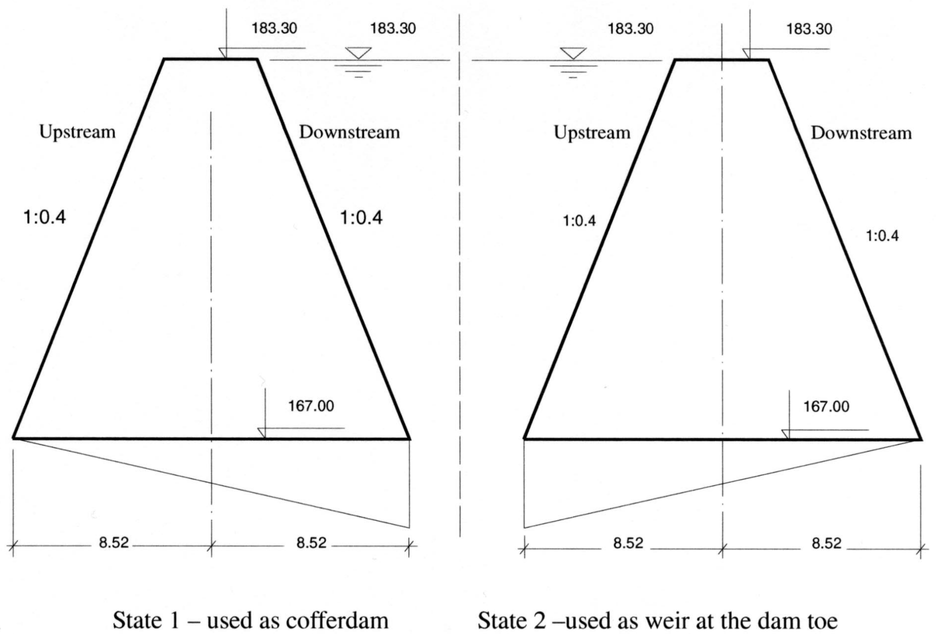


Fig. 1 CSG dam section and loading cases

2. Dissolution Property of CSG Material

With low cementitious material content (generally less than 100 kg/m^3) and high proportion of mineral admixture (such as fly ash, generally more than 50%), the hydration product of CSG Material has little $Ca(OH)_2$. Furthermore, the strength of CSG is lower and the permeability is higher due to using simple processed aggregates compared with conventional concrete. Along with CSG material to be used in coffer-dam and dams in China, some questions will be put forward, for example, dissolution and strength degradation of CSG with consideration of corrosion.The macroscopic and microscopic properties of CSG have been studied based on Jiemian project.

2.1 Experiment

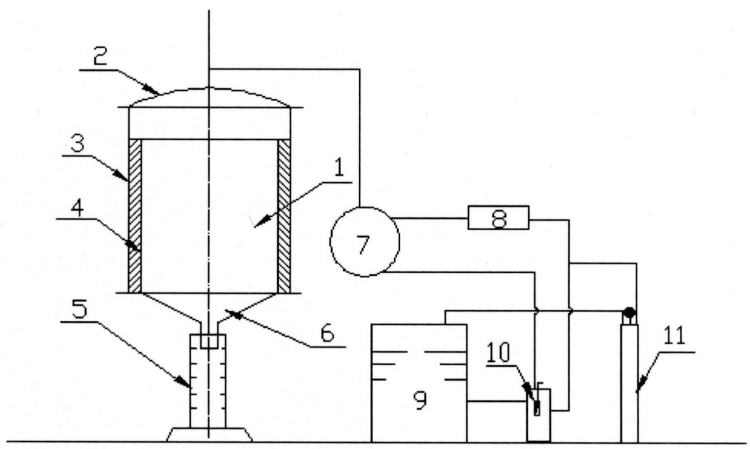
The physical mechanics properties of CSG Material including compression strength with 40cm×40cm×40cm specimen and shear strength with 20cm×20cm×20cm specimen have been studied. Riverbed sand and gravel near the dam site and other local available materials to produce experimental CSG specimen such as cement, fly ash, water-reducing admixture, etc have been used. Mix proportion and requirements are as following, cement 40kg/m^3, fly ash 40kg/m^3, sand ratio 21%, water content 70kg/m^3, and water-reducing admixture content 0.8% of cement, strength grade $C_{180}7.5$.

The experiment has been carried out by multi-function pressure dissolution instrument that jointly designed by IWHR and Tianjin construction instrument plant, as shown in Fig. 2. The specimen used for corrosion experiment is cylinder with Φ15×15cm, and prepared according to RCC requirements. After 28d standard curing period, the specimen have been tested. In order to shorten test period, 2MPa water pressure and soft water were adopted in the experiment although the practical hydraulic gradient acted on CSG Material is very low. The leaching of CaO in the percolating water was tested by titration method.

Table 1 Some typical CSG dams

No.	Name	Country	Height(m)	Completion year
1	Marathia	Greece	28	1993
2	Ano Mera	Greece	32	1997
3	Moncion afterbay	Dominica	28	
4	Can-Asujan	Philippines	40	Under construction
5	St Martin de Londress	France	25	
6	Cindere	Turkey	107	Under construction
7	Nagashima	Japan	33	1998

Some property test on CSG material, especially dissolution has been studied.



1 - Specimen with Φ15×15cm; 2 - Cap; 3 - Sealing chamber; 4 – Sealing element;
5 - graduated cylinder; 6 - Cylinder bottom; 7 - Buffer tank; 8 - Digital pressure controller;
9 – Water tank; 10 – Water pump; 11 – Alternated high-pressured gas tank.

Fig. 2. Instrumental Scheme of Leakage Corrosion

2.2 Dissolution Property

CaO Cumculative leaching of individual specimen, as shown in Fig.3 is tested. The experiment results show CaO leaching of relative specimen start to decrease step by step after 20d percolation, and reach stable state after 30d percolation. According to the leakage corrosion and percolation rule of conventional concrete and RCC, the percolation of this kind of CSG Material may stop after long-term percolation without failure in this condition. One specimen is still high after 40d percolation.

The structure and morphology of the hydration product has been obtained by SEM (scanning electron microscope) before and after leakage corrosion, as shown in Fig. 4 and Fig. 5.The hydration product of CSG material is quit different before and after leakage corrosion.The compactness of the hydration product decrease along with CaO leaching. After leakage corrosion, the gel in hydration product become relatively fine compared with orginally one.

The compression strength of CSG Material decrease after leakage corrosion, as the results shown in Table 2. It can be seen that more CaO leaching, more strength degradation, but the strength of CSG material can be satified with engineering requirement when the leakage corrosion of CSG material reaches steady state.

For CSG material, it is no problem for a cofferdam based on the result achieved and it is clear that some emphases should put upon the design and construction of the watertight barrier when constructing a permanent project using CSG, in order to avoid the CSG material enduring long-term leakage corrosion.

3. Jiemian CSG Downstream Cofferdam construction

Basic mechanics method and FEM are used for stability and stress analysis. According to the experimental result of the CSG material, shear frictional coefficient (f') of the interface between the CSG cofferdam

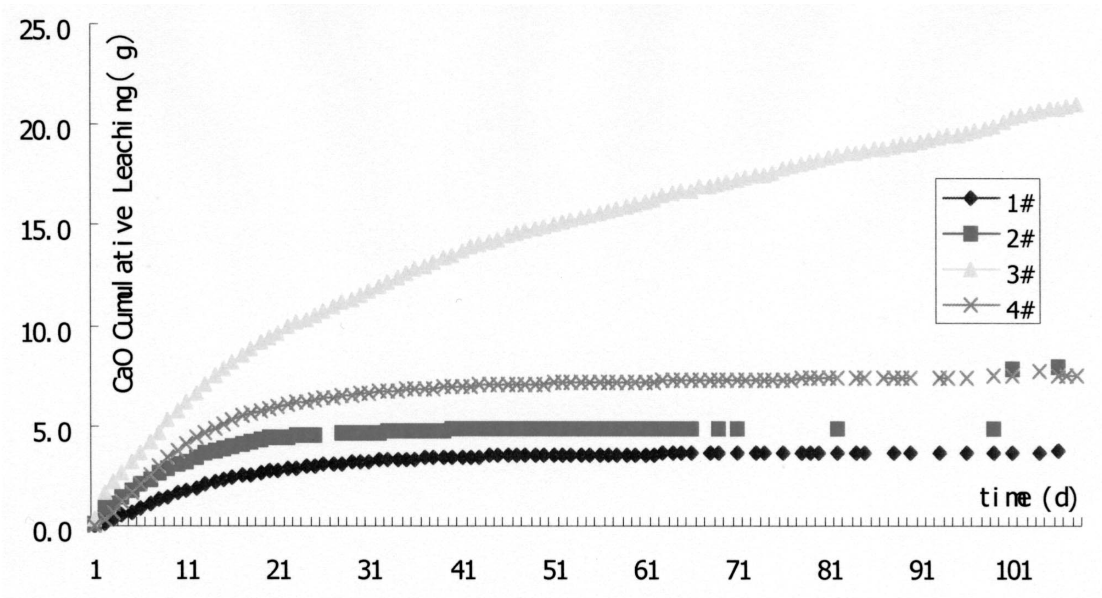


Fig. 3. CaO Cumulative Leaching of Individual Specimen

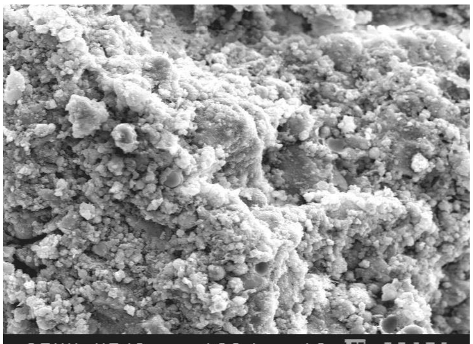


Fig.4. The hydration product of CSG

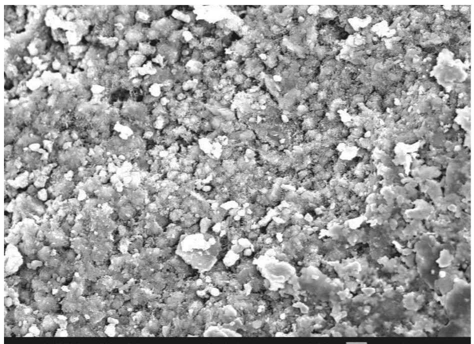


Fig.5. The hydration product after 44d leakage corrosion

Table 2. The compression strength afer leakage corrosion

Ratio of CaO leaching to total CaO (%)		Compressive strength after leakage corrosion（MPa）	Compressive strength under standard curing（MPa）	Strength degradation (%)
1#	3.8	11.8	14.8	20
2#	8.2	9.7	14.8	34
3#	21.4	9.5	14.8	36
4#	7.6	9.9	14.8	33
Ave.	10.25	10.25	14.8	31

Table 3. Some indexes of CSG Material

Material	Indexes				
	Elastic modulus ($\times 10^4$MPa)	Poisson ratio	Saturated unit weight (KN/m^3)	Compressive strength (MPa)	Tensile strength (MPa)
C7.5	1.75	0.15	24.00	10.4	0.90
C15	2.20	0.15	24.00	19.6	1.50

body material and rock mass is 0.85 and the shear cohesion (C') is 0.75Mpa. In addition, frictional coefficient (f) of the interface between the CSG cofferdam body material and rock mass is 0.70. Some indexes of CSG material were listed in Table 3.

The result indicated that the CSG dam has high stability factor, no tensile stress at any position under any cases, and little compression stress (the maximum principal stress is only 0.42MPa under loading case 2), which fully satisfy the requirement specified in the corresponding technical codes.

According to the site condition and the above experimental result, the riverbed sand and gravel near the dam site with non-procession was used as raw materials after stockpiled some time before mixing to avoid higher water content . The CSG material was mixed by loader, transported by dump truck, spreaded by backhoe excavator and compacted by roller. Before construction, full-scale trial wascarried out to check the reliability.

Mix proportion of CSG material: riverbed sand and gravel, cement 40kg/m^3, fly ash 40kg/m^3, water content was about 80kg/m^3. conventional concrete was placed at the bank part and dam foundation.

The construction of the CSG downstream cofferdam of Jiemian CFRD started on Nov. 21, 2004, and finished on Dec. 3, 2004, only 13 days, 17 days shorter than the design schedule.

At present, the CSG downstream cofferdam runs well. Compared to originally design scheme of conventional concrete cofferdam, CSG can speed the construction progress and shorten construction period with 17 days (the construction period of originally design is 30 days). With low cementitous material content and hydration heat, the construction joint was cancelled. Cost saving of CSG cofferdam is about 25% compared with the conventional concrete scheme.

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PAPERS OF CONTRIBUTION

Reservoir Sedimentation Management at the Asahi Dam

Koki Kataoka
Deputy General Manager, Civil Engineering Group,
Civil Engineering and Architecture Department
Kansai Electric Power Co., Inc., Japan

Takashi Tada
Manager, Civil Engineering Group,
Civil Engineering and Architecture Department
Kansai Electric Power Co., Inc. Japan

1. INTRODUCTION

The Kansai Electric Power Co., Inc. (the KANSAI) is one of several electric utility companies in Japan, and owns and operates 148 hydropower plants amounting to a capacity of 8,184 MW.

Some of the hydropower plants have been suffering from sedimentation, which is inevitable to dams. The KANSAI has studied various countermeasures against sedimentation to coexist with natural environment and local people.

This paper introduces one good practice of sedimentation management in reservoirs; sediment bypass system at the Asahi Dam. The KANSAI successfully put the bypassing method to practical use, which is highly estimated.

2. PLAN AND DESIGN OF THE SEDIMENT BYPASS SYSTEM

2.1 Outline of the Asahi Dam

The Asahi Dam is on the lower regulating reservoir of the Oku-yoshino Power Plant, which is the pure-pumped storage type hydropower plant.

The Oku-yoshino Power Plant was designed as a peaking plant to meet augmented electricity demand, and plays an important role for improving the grid efficiency and reliability with thermal and nuclear power plants. The construction started in 1975 and ended in 1980. The specifications are shown in Table-1 and the location is shown in Figure-1.

2.2 Background

Since the completion of the Asahi Dam, a prolonged turbidity problem had become serious due to frequent collapse of mountain slopes and heavy rainfalls in the upstream basin. In particular, in 1990, four successive large-scale runoffs caused remarkable prolonged turbidity lasting over 200 days.

Additionally, the mean annual sedimentation from 1989 to 1995 increased sharply to 85,000 m^3/year, while the one during 10 years from 1978 to 1988 was approximately 20,000 m^3/year. This was caused by widespread collapse of slopes in the upstream due to frequent typhoons in 1989 and 1990. The KANSAI made a simulation to predict the sediment profile and its influence to the power generation. The results showed that the sediment level 10 years after (2003) would rise up to the intake level and would intervene the power generation.

Table-1: Specifications of the Asahi Dam

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

*: when constructed

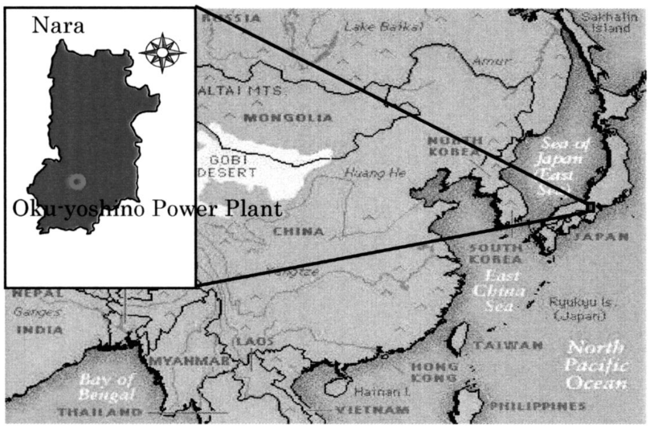


Figure-1: Location of the Asahi Dam

The KANSAI studied countermeasures that would effectively reduce the prolonged turbidity and the advance of sedimentation. From the study, the most appropriate measure for the Asahi Dam was to install sediment bypass system. There are primarily two reasons for this conclusion. First, the Asahi Reservoir does not need to store inflow water because the reservoir is for the pure pumped-storage power plant. Second, the drainage basin of the Asahi Dam is small and the runoff is relatively low.

2.3 Design of the Sediment Bypass System

The sediment bypass system at the Asahi Dam was designed to treat bed loads and suspended loads besides wash loads from the purposes of mitigating both prolonged turbidity and sedimentation. There were two requirements for the bypass system. One was to eliminate most of the prolonged turbidity when the peak inflow was 200 m^3/sec, which is the 1-year return period inflow at the Asahi Dam. The other was to flush all of bed loads from the upstream when the peak inflow was 660 m^3/sec which is the maximum inflow in the past, or when 1,200 m^3/sec which is the design flood, at the Asahi Dam. The capacity of the bypass system was determined as 140 m^3/sec by performing simulations (1- and 2-dimensional models), which was derived by the former requirement.

The schematic layout of the bypass system was designed in consideration of the site characteristics. Based on the uniform flow calculation, the cross-section of the bypass tunnel was planned so that the tunnel could pass the flow with the water depth of 80 % of the tunnel height. The entrance of the tunnel was composed of a diversion weir and an orifice intake, which would be desirable for flushing bed loads. With these structures, the volume of water and sediment into the tunnel could be naturally regulated. The flow direction from the outlet of the tunnel was set as parallel as possible that of the original river channel. The elevation of the tunnel invert was designed above the flood water level (1,200 m^3/sec).

After the schematic design, hydraulic model tests were conducted besides the numerical simulations so as to evaluate the capacity of the bypass tunnel to discharge bed loads.

The specifications of the bypass system, the layout and the outlines of the components are shown in Table-2, Figure-2 and Figure-3 respectively.

Table-2: Specifications of the bypass system

Weir	Height	13.5m
	Crest Length	45.0m
Intake	Height	14.5m
	Width	3.8m
	Length	18.5m
	Type	Reinforced Concrete with Steel Lining
	Gate	1
Bypass tunnel	Height	3.8m
	Width	3.8m
	Shape	Hood
	Gradient	Approx. 1/35
	Max.Discharge	140m^3/s
	Type	Reinforced Concrete Lining
Outlet	Height	15.0m
	Width	5.0～8.0m
	Type	Reinforced Concrete

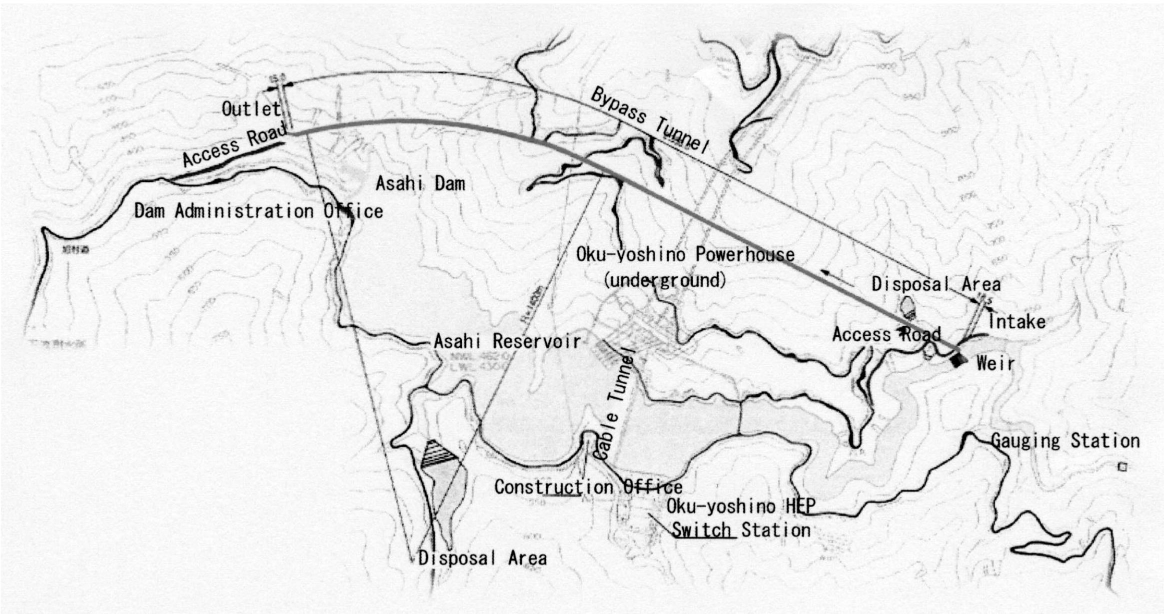


Figure-2: General layout of the bypass system

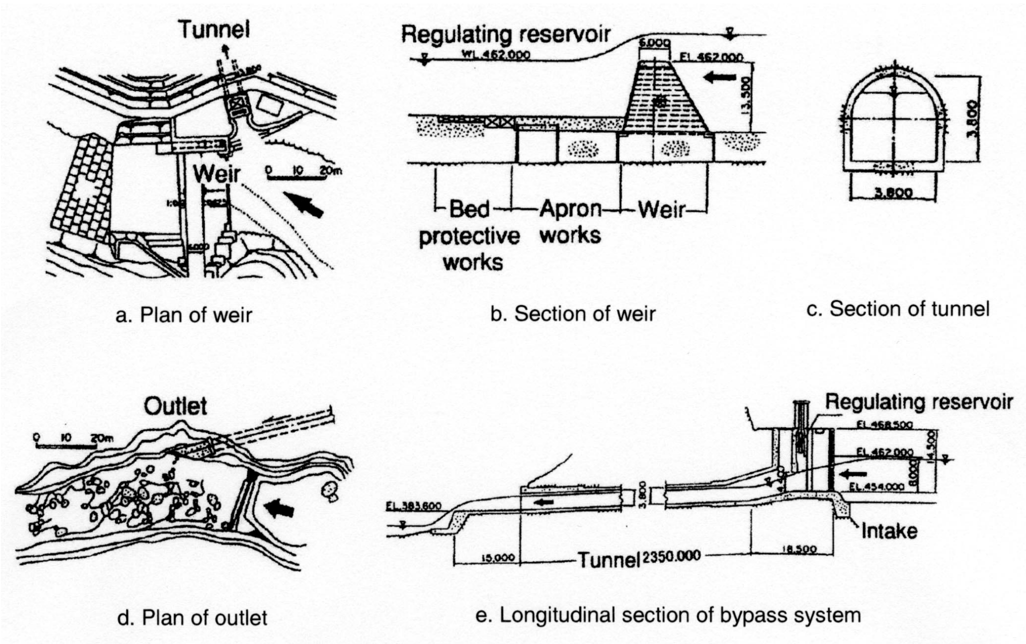


Figure-3: Main components of the bypass system

3. PERFORMANCE AND EFFECT ON THE BYPASS SYSTEM

3.1 Actual Results of Operation

Figure-4 shows the monthly inflow to the Asahi Dam and the monthly discharge through the bypass tunnel in 2004. This figure tells us that approximately 62 percent of the total inflow to the Asahi Dam is directly bypassed to the downstream.

Figure-5 summarizes the yearly operation of the bypass system from 1999 to 2004. This figure tells us that approximately 60 percent of the total inflow to the Asahi Dam is directly bypassed to the downstream every year. We should notice that the inflow in 2004 is larger than that in the other years. We have encountered large flood over 330 m^3/sec three times in 2004, which is 5-year return period flood.

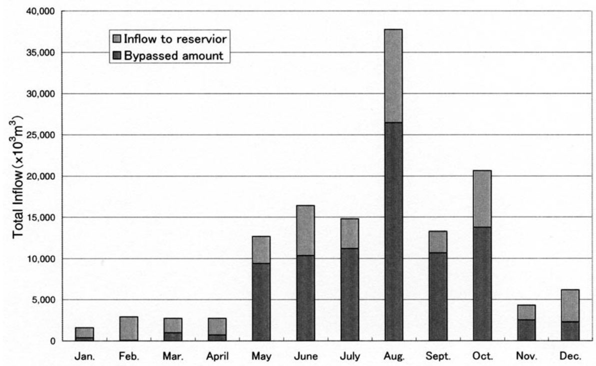


Figure-4: Monthly inflow in 2004

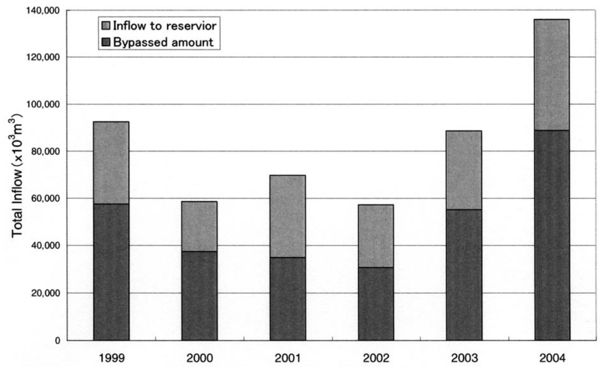


Figure-5: Yearly inflow in 1999 ~ 2004

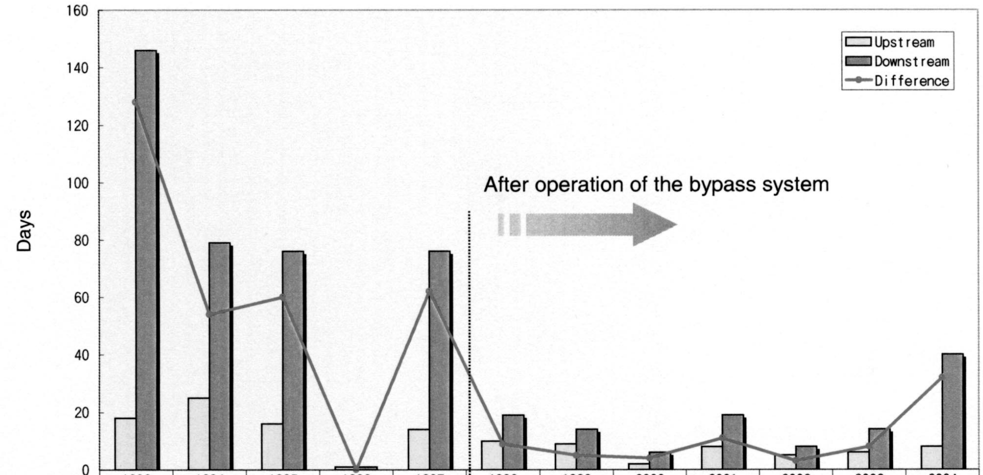
3.2 Effect on Prolonged Turbidity

Figure-6 reveals that the bypass system mitigated the prolonged turbidity problem. Suspended solid concentration (SS) has been observed once a day at the two points, 4.3 km upstream and 1.6 km downstream from the Asahi Dam. The number of days when SS is over 5 ppm at the downstream measuring point drastically decreased to less than about 10 days on average after the operation of the bypass system in 1998. Please note that the year 2000 was a dry year and that the year 2004 was a wet year.

Let us focus on the cases on June 20, 2001 (the peak inflow of 288 m^3/sec) and on July 13, 1987 (the peak inflow of 271 m^3/sec). The results measured 1 week later from the peak inflow at the two points are represented on Figure-7 and Figure-8 respectively. The SS at the downstream measuring point in 1987 is about 20 ppm, when the bypass system was not constructed. On the other hand, the SS at the point in 2001 is below 5 ppm, when the bypass system was installed. These results clearly show that the bypass system is mitigating the prolonged turbidity.

3.3 Effect on Reservoir Sedimentation

Figure-9 shows the changes of sedimentation volume in the reservoir. After the operation of the bypass system in 1998, the reduced sediment volume with the bypass system is calculated by the total sediment



	1993	1994	1995	1996	1997	1998	1999	2000	2001	2002	2003	2004
Upstream	18	25	16	1	14	10	9	2	8	5	6	8
Downstream	146	79	76	1	76	19	14	6	19	8	14	40
Difference	128	54	60	0	62	9	5	4	11	3	8	32

Figure-6: Number of days with turbidity over 5 ppm

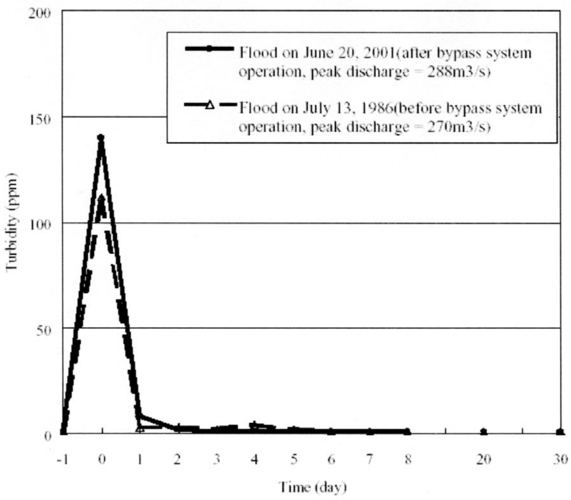


Figure-7: Turbidity 4.3 km upstream from the Asahi Dam

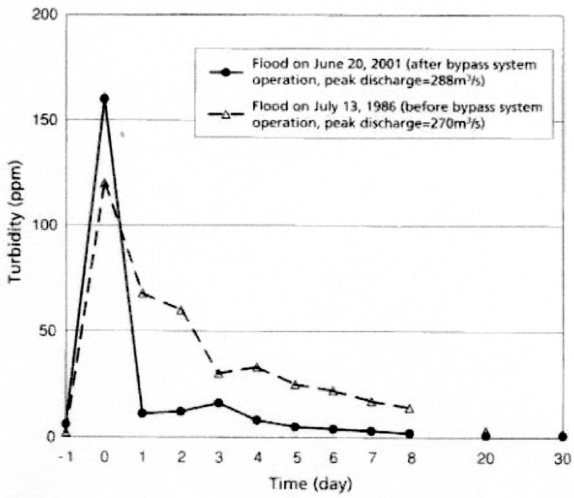


Figure-8: Turbidity 1.6 km downstream from the Asahi Dam

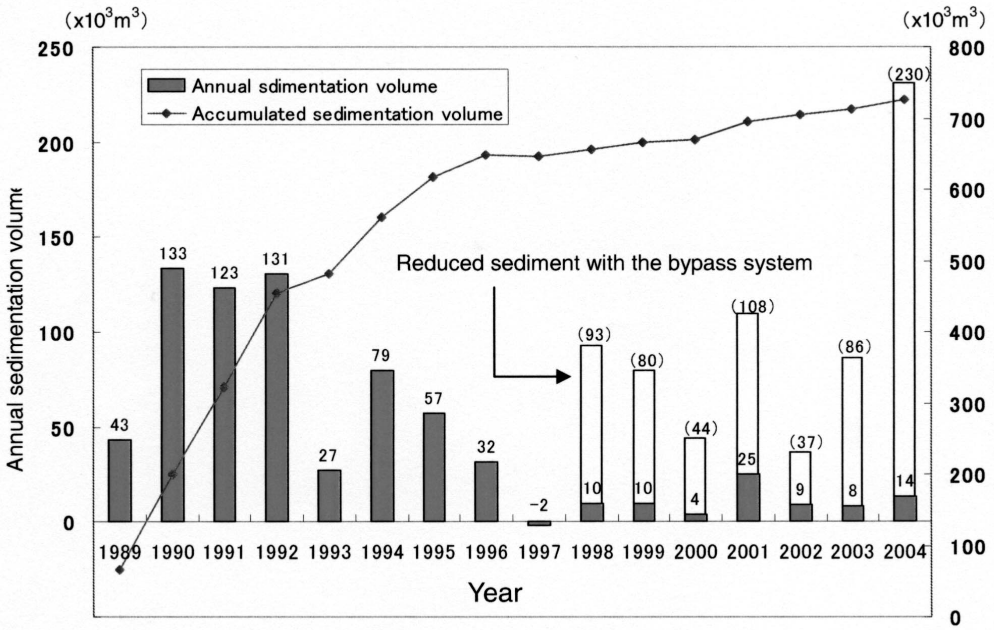


Figure-9: Sedimentation volume in the Asahi Reservoir inflow minus the difference of the volume between successive years through the bathymetric survey. The total sediment inflow is calculated with the Ashida-Michiue Formula, which shows the volume of bed load. In the calculation, the grain size distribution is derived from a field survey (D_{50} = 5 cm) and the porosity of the bed loads is assumed to be 0.4. For labor saving, the inflow under 30 m^3/sec is neglected.

It can be concluded from Figure-9 that only 10 to 20 % of the total sediment is deposited in the reservoir, while 80 to 90 % is discharged through the bypass tunnel.

After several operations of the bypass system, the elevation of the upstream riverbed was almost no change. Thus the weir of the bypass system does not trap the sediment and almost all of bed loads are discharged through the bypass tunnel.

3.4 Effect on Water Quality

The bypass system does not give any significant harmful effect on the water quality of the reservoir. Figure-10 shows the changes of turbidity (SS), total nitrogen (T-N) and total phosphorus (T-P) in the reservoir.

Turbidity has been low because the total amount of the turbid water into the reservoir has decreased after the operation of the bypass system. For the T-N and T-P values, there is no significant problem regarding the water quality of the reservoir.

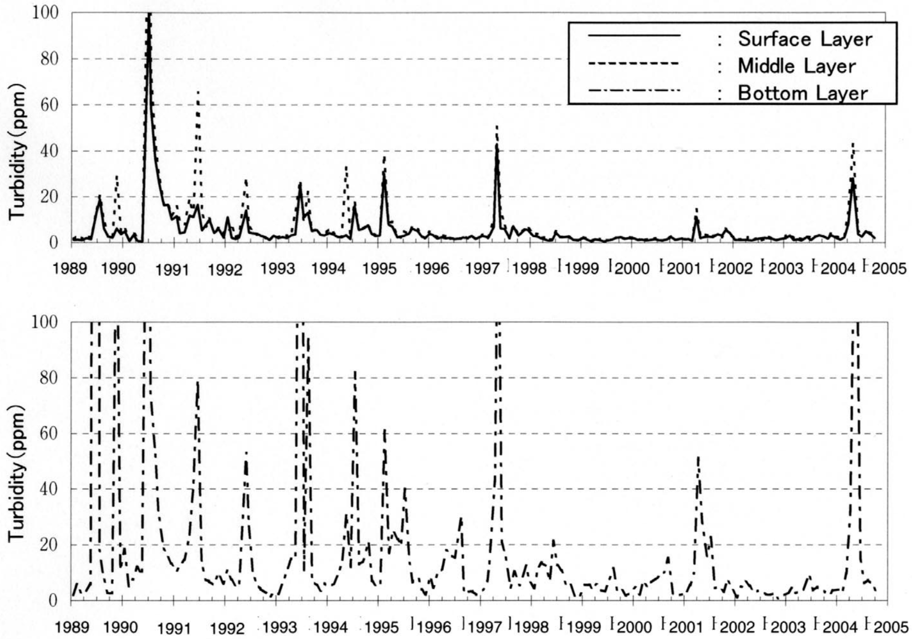


Figure-10(a): Variations of Turbidity

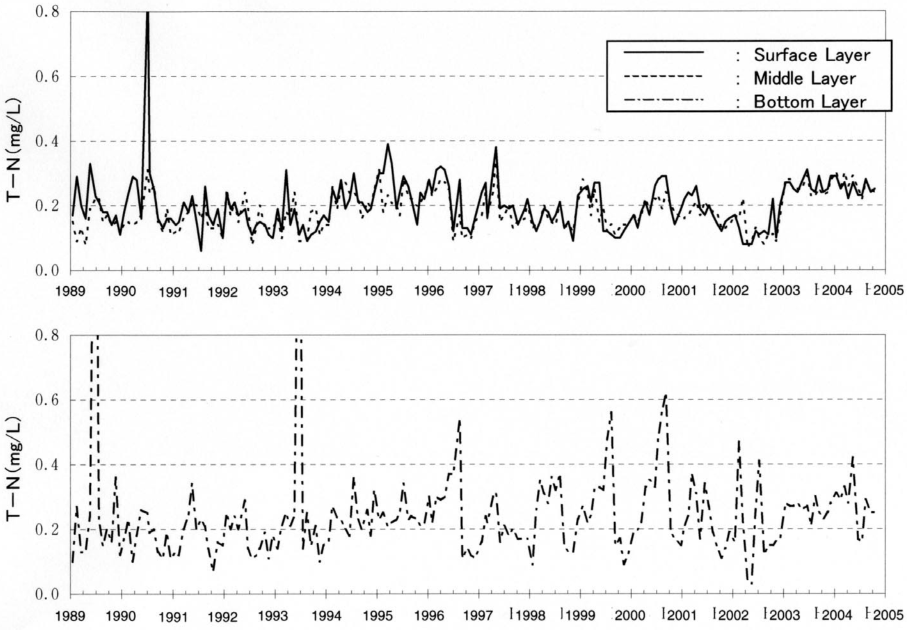


Figure-10(b): Variations of T-N

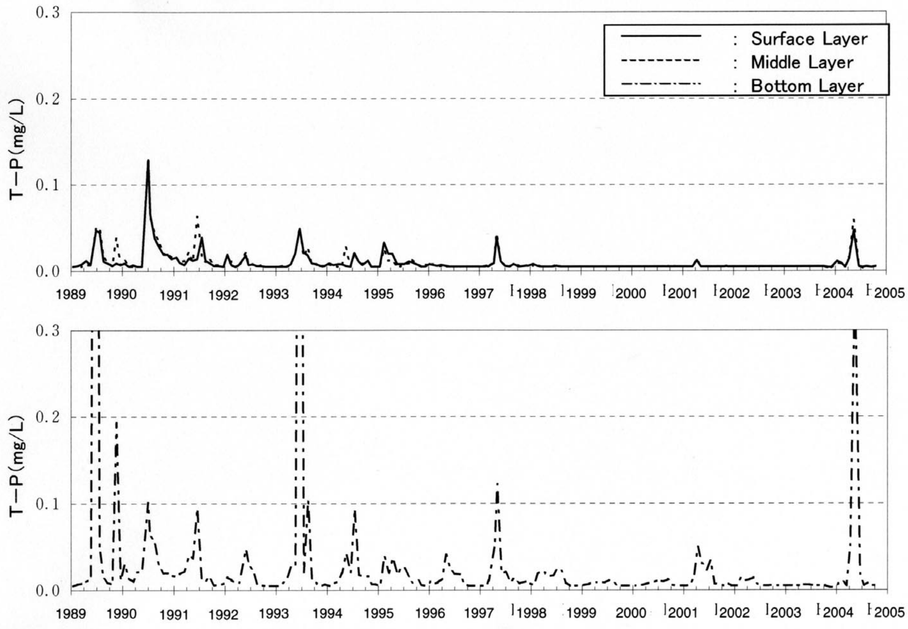


Figure-10(c): Variations of T-P

3.5 Effect on the Downstream River

To examine the impact on the downstream river caused by the bypass system, the distribution of the grain size was investigated for three years after the operation of the bypass system. Figure-11 shows the distributions of the grain size at the riverbed 400 m downstream from the bypass tunnel outlet. The results reveal that gravel of which diameter is more than 200 mm exists after 1999, although gravel with the same size does not exist in 1998. The site reconnaissance survey also shows that the amount of gravel with medium and small size on the downstream riverbed is increasing.

Figure-12 shows survey results of the elevations on the downstream riverbed. Before the bypass operation there was a tendency to scour the riverbed, after the operation the elevations of the riverbed have increased up to those of the river before the dam construction. It is estimated that the bypass operation is effective in preventing the downstream riverbed from being scouring and effective in maintaining the elevations of the downstream riverbed.

3.6 Abrasion of the Tunnel Invert

Figure-13 shows the measured volume of the bypassed sediment (the porosity equals 0.4) and the mean abrasion depth on the tunnel invert (about 9,000m^2 in area) after the sediment bypass system was operated. It can be found that the quantity of abrasion on the invert concrete, of which the design strength is

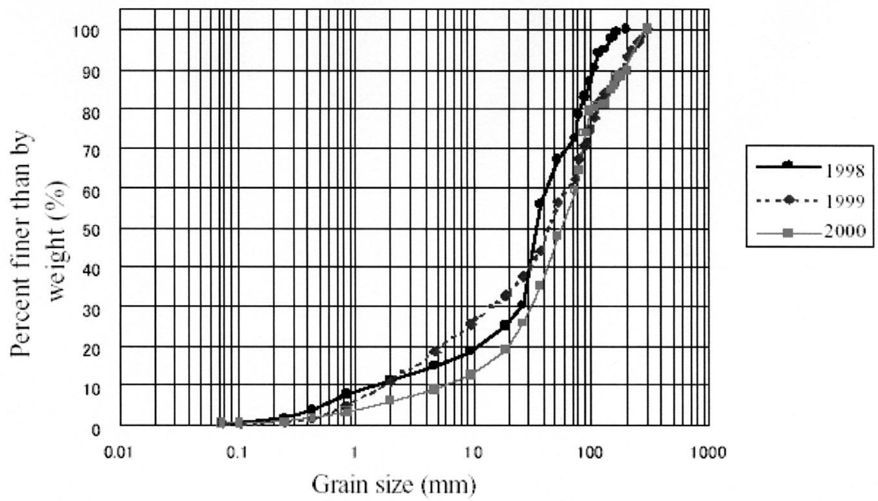


Figure-11: Grain size distribution at the downstream riverbed

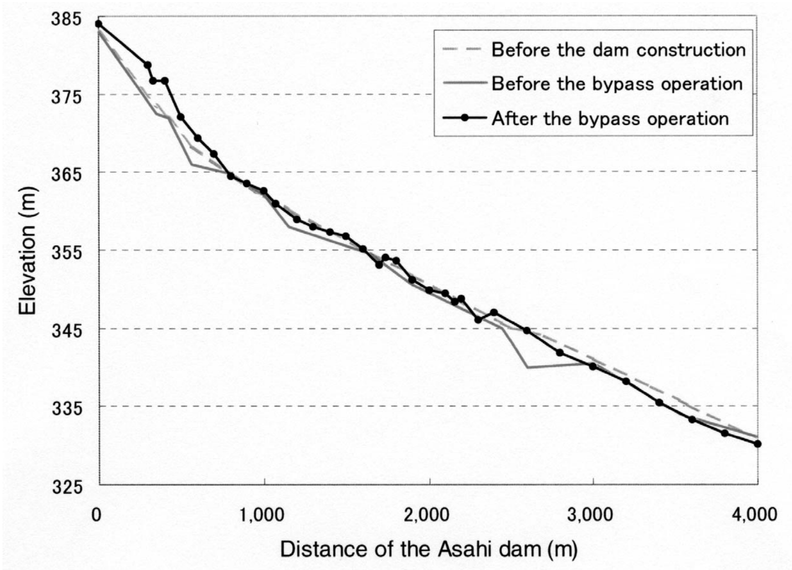


Figure-12: Survey results of the elevations on the downstream riverbed

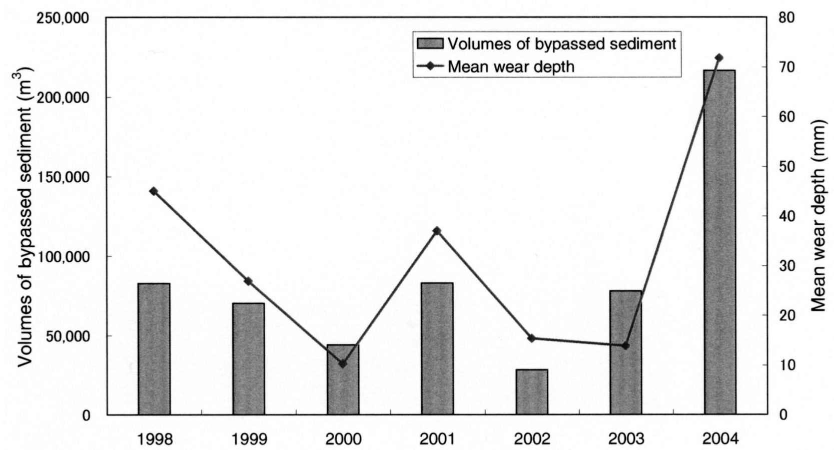


Figure-13: Amount of the bypassed sediment and the mean abrasion depth on the tunnel invert

36 N/mm^2 varies with the volumes of the bypassed sediment. The abrasion depth is within the range which was forecasted at the design stage. The sections worn seriously on the bypass tunnel invert are repaired during the non-flood season.

4. CONCLUSIONS

The environmental investigation into the river and reservoir clearly reveals that the sediment bypass system at the Asahi Dam can mitigate the prolonged turbidity and sedimentation problems. And it is likely that a downstream river state has been gradually restored to its former state since much sediment was supplied to through the bypass tunnel.

Furthermore, the KANSAI is planning for medium- and long-term investigation and measurement to research the effects of the bypass system, focusing on the following items;

a) water quality and sedimentation status in the reservoir and river
b) changes in the downstream river ecosystem
c) abrasion in the bypass tunnel and its countermeasures through in-situ tests

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Estimation of Non-uniform Sediment Discharge Using Actual Results of Dam Reservoir Sedimentation

Tomoyuki Suzuki
Assistant Manager, Dr. Eng., Engineering Dept., Planning Division, INA Corporation, Japan

Josuke Kashiwai
Team Leader, River and Dam Hydraulic Engineering Team, Hydraulic Engineering Research Group, Public Works Research Institute, Japan

Yoshihiro Yoshioka
Kashiwazaki Regional Office, Nigata Prefecture, Japan

1. Introduction

The dams with the sedimentation volume exceeding or approaching their design storage capacity start to appear due to the greater-than-designed sediment inflow and the increasing of management years. It is an urgent matter to investigate and implement measures to combat sedimentation in these dams. In order to devise the most appropriate measures for each dam, however, it is necessary to have an accurate picture of sedimentation conditions and to formulate dependable forecasts of future sedimentation likely to result from any adopted measures. A numerical model for the riverbed variation is a useful tool for such forecasts, but requires accurate estimates of sediment inflow: a key boundary condition of the model.

One set of parameters necessary for such a simulation is the relationship between the flow rate and the sediment discharge for each typical within a dam deposit. The relationship can be obtained by combining data from monitoring records at each dam with the results of additional surveys, however, the estimating procedures have not to be established yet.

We recently obtained the opportunity to estimate the sediment discharge at the Sabaishigawa Dam, Niigata Prefecture, which is one of the dams facing the urgent sedimentation problems. Additional surveys included boring of dam deposit and sampling of inflow and outflow water during flood, and we gathered the detailed information of sediment diameter in the river and analyzed using the hydrologic data and the annual sedimentation records. As a result, we obtained the relationship between the flow rate and the sediment discharge reproducing fairly well the annual sedimentation records. This paper describes the estimation method for sediment discharge and its validity.

While this study investigates conditions solely at the Sabaishigawa Dam, we attempted to systematize the survey methods and analytical procedures to provide a framework for similar investigations at other sites. In the dam, a large amount of fine sediment passes through the reservoir. We developed a procedure for estimating a trap ratio of the sediment inflow (trap efficiency) from numerical simulations. The contents appear to be of general utility in studies of this type.

2. Overview of Estimation Method

2.1 Flowchart of general procedure

The flowchart in Fig. 1 shows the procedure we propose. The method is based on the actual records for dam sedimentation. A relationship between the flow rate and the sediment discharge can be identified for each typical particle size contained in dam deposit.

2.2 Classifying dam deposit by sediment tarp condition

We use surveyed data for boring, sedimentation topography, etc. to determine the spatial distribution of sediment diameter within a dam deposit. The deposit can be classified as either “completely trapped sediment”, which is completely trapped within a reservoir because of coarse particle, or “partially trapped sediment” that is partially trapped in a reservoir because of fine particle (①: means “1st step”). The threshold diameter that distinguishes these two types is unique for each reservoir, and is different from the conventional classification of bed load, suspended load and wash load. Here, we assume that the bed load is not included in the partially trapped.

2.3 Estimation of sediment discharge for the completely trapped particles

First, the upstream riverbed is analyzed to find the typical cross section and gradient, and we examine how accurately the bed load and suspended load are predicted by the formulas generally used by researchers (②). For this assessment, the minimum discharge in which sediment transport occurs (critical discharge Q_c) is also considered as a parameter. The index of applicability of the formula is the annual sedi-

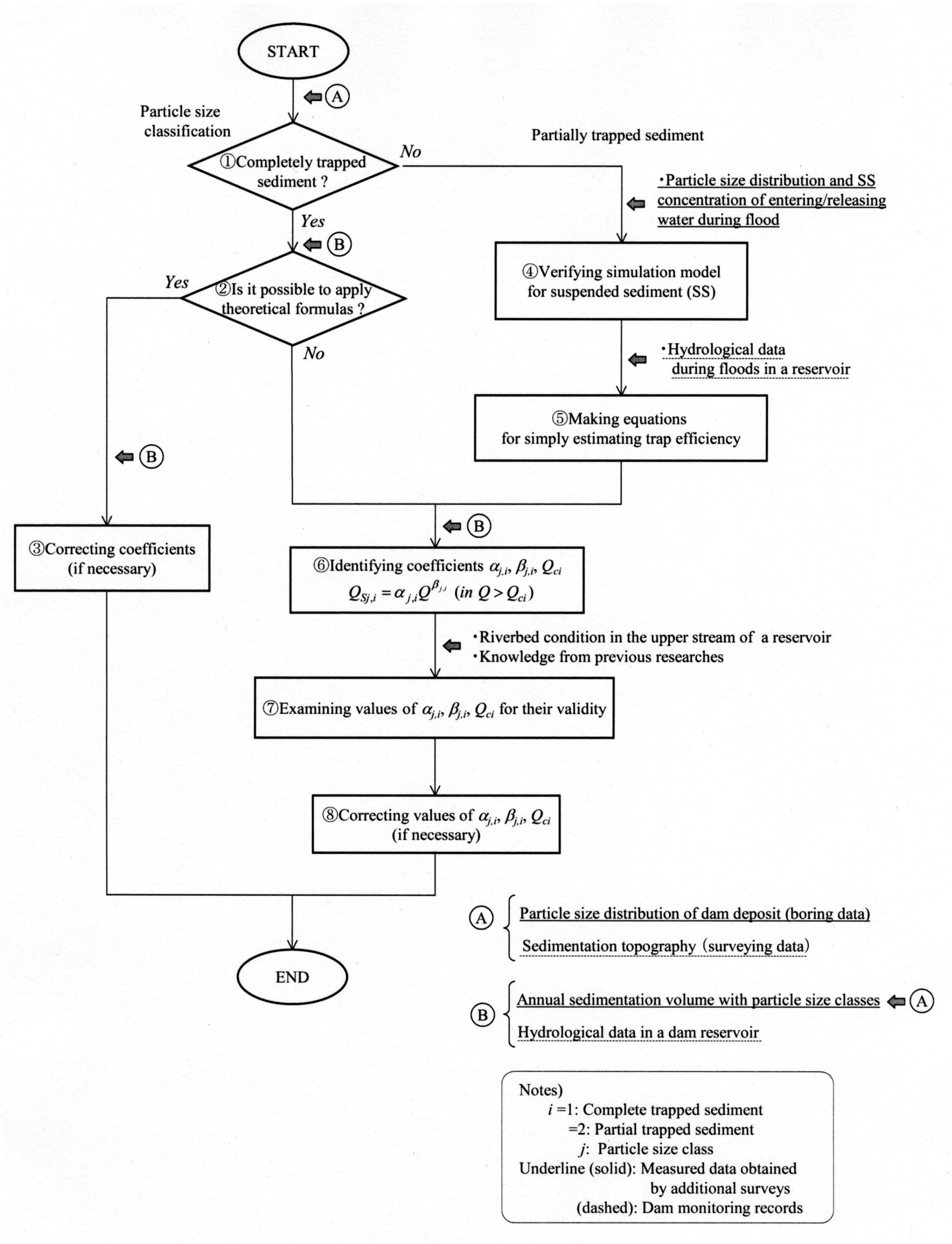


Fig. 1 Flowchart of procedure for estimating "non-uniform" sediment discharge

mentation volumes with the typical particle sizes, which can be calculated using the spatial distribution of sediment diameter from the boring survey and the annual sedimentation volume. If the formula is considered to be valid, it is then used to estimate the sediment discharge of the completely trapped sediment.

When there is much bare rock and armoring upstream of a reservoir, or when the completely trapped sediment contains a large amount of fine sediment, the sediment influx volume given by the formula commonly diverges significantly from the actual sedimentation volume. In such cases, the formula is considered to be meaningless. This situation is encountered in our study at Sabaishigawa Dam. The simple exponential function shown below is used to relate the sediment flow rate Q_{Sj} of the typical particle class (j) to the river flow rate Q. The variables α, β and Q_c are identified from the actual deposit volumes (the annual sedimentation volume for each typical particle size; ⑥).

$$Q_{Sj} = \alpha_j Q^{\beta_j} \qquad (Q>Q_c) \tag{1}$$

The above equation is usually used to describe the relationship between the wash load and the flow rate. An equation of the same form is used later for the partially trapped sediment. The obtained values of coefficients and critical discharge are verified and corrected if necessary (⑦, ⑧).

2.4 Estimation of sediment discharge for the partially trapped particles

Equation (1) is used to find the coefficients and critical discharges for inflow of the partially trapped sediment, just as for the completely trapped one when the formula is inapplicable. However, as some of the inflow of fine sediment usually passes through a reservoir, the passing volume must be subtracted from the inflow volume calculated using Eq.(1) to obtain the trapped volume. The following equation is used to calculate the trap efficiency γ_{jk} for the typical particle class (j) under flood (k), yielding the annual sedimentation volume [VSj]i during year (i).

$$[V_{Sj}]_i = \sum_{k=1}^{n_i} \gamma_{jk} \int_k Q_{Sj} dt = \sum_{k=1}^{n_i} \gamma_{jk} \int_k \alpha_j Q^{\beta_j} dt \tag{2}$$

Here, ni represents the number of floods above the critical discharge during year (i).

The trap efficiency γ_{jk} is estimated using a numerical simulation model whose validity has been demonstrated from observed data from the water sampling during floods (④,⑤). A simple method for estimating γ_{jk}, which is explained in detail in Chapter 3.5, is desired as it would have required much time and labor to carry out simulations of "all past floods".

In the following section, we show how the above procedure is used to determine sediment discharge at the Sabaishigawa Dam.

3. Estimation of Sediment Discharge at Sabaishigawa Dam

3.1 Overview of Sabaishigawa Dam

Figure 2 shows a catchment area of the Sabaishigawa Dam. The dam is on the Sabaishi River, the class B river running through the city of Kashiwazaki, Niigata Prefecture. The gravity-type concrete dam is 37 m high and 170 m wide and has a reservoir capacity of 6,000,000 m^3. The dam is multipurpose, with the chief purposes of flood control and supply of irrigation water. Construction was completed in 1974. The dam is also fed by the Ishiguro River, and the two rivers account for 85% of the 46 km^2 catchment area of the dam. The reservoir is made up mainly of the former valleys of the Sabaishi and Ishiguro Rivers and comprises the junction of the rivers.

Much of the sediment discharge to the dam enters within flood waters following rains in the summer, autumn seasons, and during runoff associated with spring thaw. The flood history detailed in Fig. 3 shows that floods with peak discharges in excess of 100 m^3/s occurred 5 times in the 29 years since the dam was constructed. Discharges exceeding 40 m^3/s occurred approximately yearly during that period. Discharge during spring run-off varied greatly from year to year, but was in the range of 20 – 30 m^3/s for periods of about 60 days in the years of greatest runoff. The reservoir water level is usually lowered during periods of the spring run-off, and this operation would have released the accumulated sediment downstream of the dam.

Figures 4 show variations in the longitudinal topography of sedimentation over past years. Fig. 4(a) shows the section from the junction to the Ishiguro valley upstream, while Fig. 4(b) shows the section from the dam to the Sabaishi valley upstream. Sedimentation occurred throughout the reservoir, and no significant differences in sedimentation form were observed between the Sabaishi and the Ishiguro valleys. Sedimentation delta wasn't clearly observed in both valleys. This was probably due to the accumulation of large amount of fine sediment and the effects of lowering the reservoir water level.

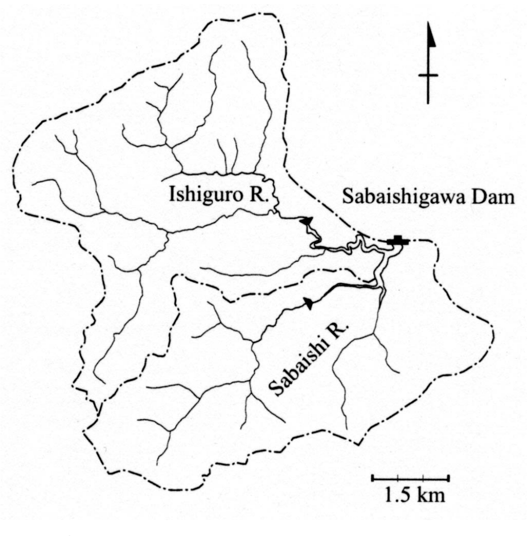


Fig. 2 Sabaishigawa dam and its catchment basin

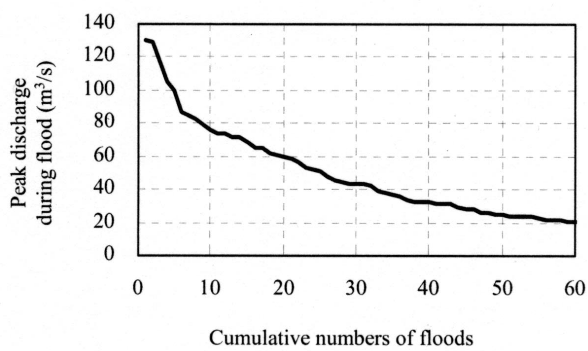
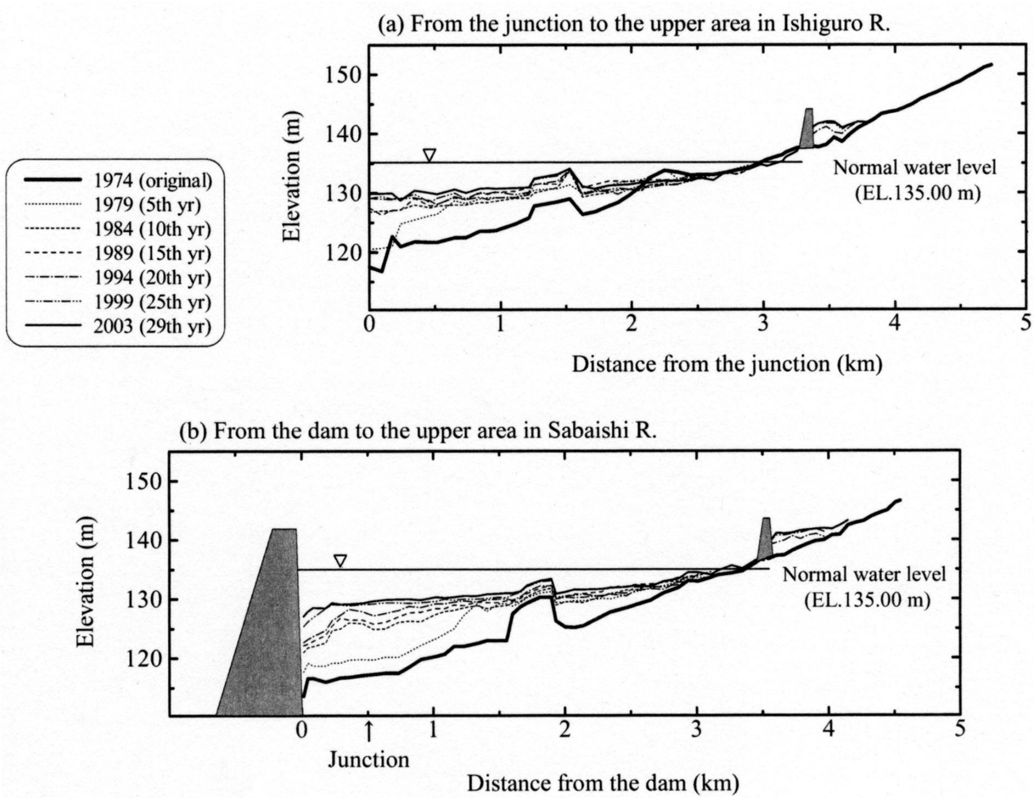


Fig. 3 Cumulative numbers of floods (1974-2002) (excepting floods of melted snow)



Figs. 4 Temporal changes in longitudinal topography

Table 1 List of data used in this study

Survey Item	Data Item	Chief Objective	Specifications (in case of Sabaishigawa Dam)
Dam monitoring records	Water discharge	Estimating trap efficiency; Selecting sedimentation record; Numerical simulation	At 10 min–1 hr intervals (peak flood > 30 m³/s)
		Estimating trap efficiency; Selecting sedimentation record	Daily mean (over season)
	Water level	Selecting sedimentation record	Daily mean (over season)
		Numerical simulation	At 10 min–1 hr intervals (flood only)
Topographic survey of dam deposit	Longitudinal shape	Setting locations for boring	Every year (entire period)
	Cross sectional shape	Calculating sedimentation volume; Numerical simulation	
Boring of dam deposit	Particle size distribution	Classifying dam deposit by particle size; Calculating annual sedimentation volume with typical particle size classes	Vertical profiles (1 m increments) at 18 locations (conducted in 2002)
	Porosity		
Water sampling during flood	SS concentration	Examining characteristics of SS discharge; Verifying simulation model; Verifying estimated values of coefficients α, β	10 – 12 samples each taken at up/down-stream of a reservoir (conducted in 2003)
	SS size distribution		

3.2 Data used in this study

Table 1 shows the data used for this study. In the table, the dam monitoring records and the sedimentation topographic data are regularly accumulated by the dam management office in Japan. The boring of dam deposit and the water sampling are additional surveys for the study, which have been carried out in 2002, 2003. Figure 5 shows the locations of the additional surveys.

3.3 Particle size classification of dam deposit

Fig. 6 presents the particle size distributions of dam deposit calculated from the boring results and the sedimentation volume records. The figure indicates that all of the three zones had nearly identical patterns of distribution; particles less than 100 μm in size accounted for about 80% of the entire sedimentation volume. The measured relationship between D_{60} and porosity is typical for dams in Japan[2].

Fig. 7 shows the particle size distribution of sedi-

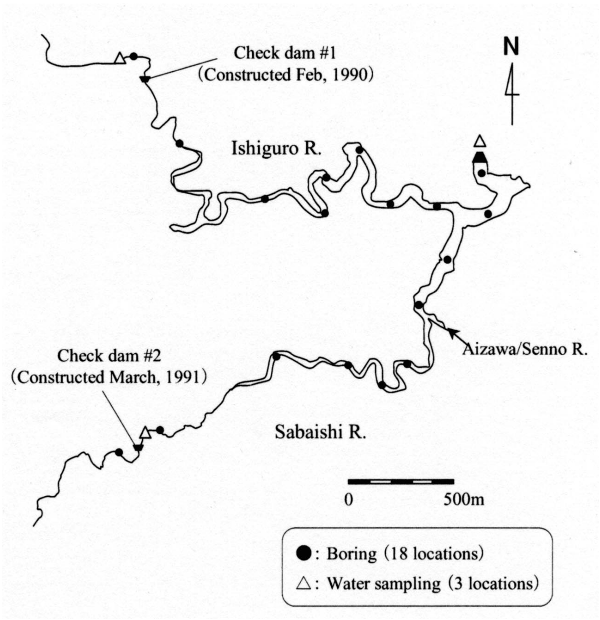


Fig. 5 Locations of additional surveys

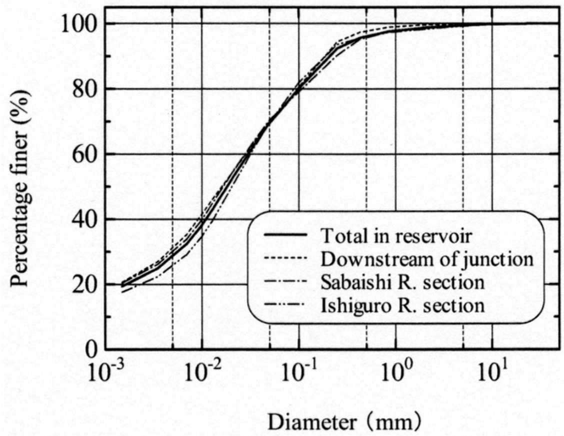


Fig. 6 Particle size distributions of the dam deposit ("Downstream of junction" in the legend means the zone between the dam and the junction of the rivers, "Sabaishi R. section" means the Sabaishi valley upstream of the junction, and "Ishiguro R. section" means the Ishiguro valley upstream of the junction.)

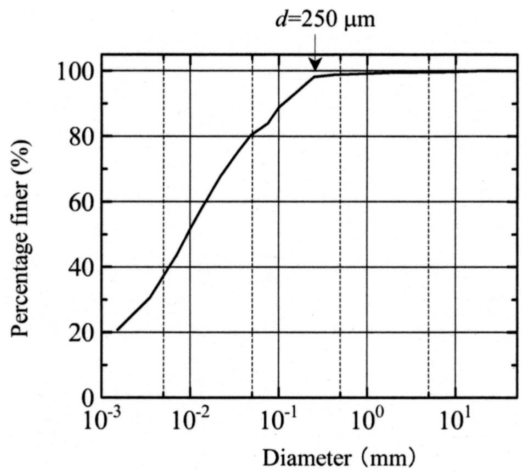


Fig. 7 Particle size distributions of the dam deposit (just upstream of the dam)

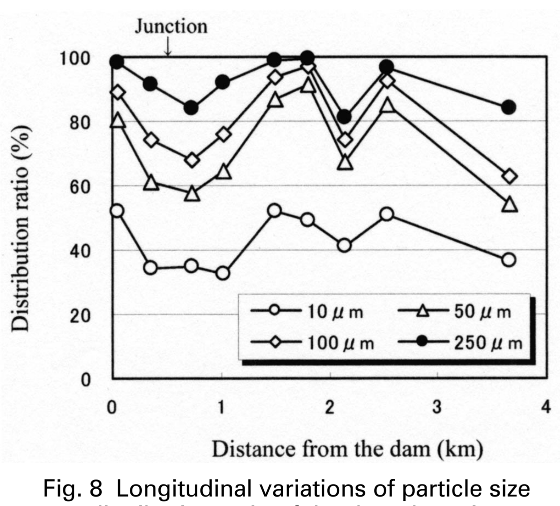


Fig. 8 Longitudinal variations of particle size distribution ratio of the dam deposit (example: Sabaishi R. section)

ment deposited just upstream of the dam. The sediment > 250 μm in size was hardly contained. This information indicates that the sediment > 250 μm were almost completely trapped upstream of the dam.

Fig. 8 shows longitudinal variations of particle size distribution in the dam, within the Sabaishi Valley. Some sediment coarser than 250 μm was found in the vicinity of the river junction, but this sediment did not extend to the dam. Upstream of the junction, the observed sediments varied gradationally between coarse and fine, and Fig. 8 is probably a good representation of the sediment constituents. Form the above observations, 250 μm was selected as the diameter that represents the boundary between complete and partial trapping. As a result, the partially trapped sediment makes up about 90% of the total sedimentation volume at the dam, a very high fraction. It is therefore more important to predict the estimation of the partially trapped sediment than of the completely trapped at this dam. As shown in Table 2, when this diameter is used, we classified six particle diameter classes finer than 250 μm and four classes larger than 250 ?m, and attempted to estimate discharge of these 10 size classes.

Coarse sediment was contained near the 1 and 2 km upstream from the dam much more than other area (Fig. 8). While no clear explanation has been found for this, it is likely that a contributing cause at the 1 km site is the presence of a tributary (Aizawa/Senno Riv.) just upstream from that location (see Fig. 5), which would be expected to increase the mean flow speed and reduce the accumulation of the fine sediment. Other possible factors are the change of the reservoir cross sectional shape in the longitudinal direction, but no relationship was observed between this factor and the phenomenon.

Table 2 Particle size classification

	Class #	Size range (mm)	Typical size (mm)
Partially trapped	1	– 0.0035	0.0035
	2	0.0035 – 0.010	0.0059
	3	0.010 – 0.022	0.015
	4	0.022 – 0.048	0.032
	5	0.048 – 0.10	0.069
	6	0.10 – 0.25	0.16
Completely trapped	7	0.25 – 0.85	0.46
	8	0.85 – 4.8	2.0
	9	4.8 – 19	9.5
	10	19 – 38	27

3.4 Estimation of sediment discharge for the completely trapped particles

We initially attempted the estimation using the Ashida and Michiue's formulas for sediment discharge (bed load, Ref. 3; suspended load, Ref. 4), but the prediction far exceeded the observed. This indicates that the sediment supply is far less than the carrying capacity of the river channel. We therefore concluded that the formulas were not applicable to this river channel. The coefficients α, β and Q_c were therefore determined from Eq.(1) ,"the simple exponential expression". For the completely trapped sediment, β and Q_c were selected to minimize the residue of the following:

$$\tilde{\varepsilon}_j = \sum_{i=1}^{N} \left| [\tilde{V}_{S0j}]_i - [\tilde{V}_{Sj}]_i \right| \tag{3}$$

Here, N is the number of years in the period of interest, and $[V_{S0j}]_i$ and $[V_{Sj}]_i$ are the measured and estimated dimensionless annual volumes of sediments of size class (j) during the period, respectively. The values are given by the following equations:

$$[\tilde{V}_{S0j}]_i = [V_{S0j}]_i \Big/ \sum_{m=1}^{N} [V_{S0j}]_m \tag{4}$$

$$[\tilde{V}_{Sj}]_i = [V_{Sj}]_i \Big/ \sum_{m=1}^{N} [V_{Sj}]_m = \sum_{k=1}^{n_i} \gamma_{jk} \int_k Q^{\beta_j} dt \Big/ \sum_{m=1}^{N} \sum_{k=1}^{n_m} \gamma_{jk} \int_k Q^{\beta_j} dt \tag{5}$$

Here, $[V_{S0j}]_i$ in Eq.(4) is the measured volume of sediments of size class (j) during year (i), and the trap efficiency γ in Eq.(5) is 1 for the completely trapped sediment. $[V_{Sj}]_i$ in Eq.(5) is substituted into Eq.(2). α is also eliminated by the process that eliminates the dimensions. Therefore, β and Q_c are identified in Eq.(3). As reference in the search for values for β, the previous observations have been reported $\beta = 2 \sim 3$; the value of β was searched from 1 to 5 to cover an actual value sufficiently. Q_c values were trialed within the range between 0 and 20 m^3/s. The search ranges for β and Q_c were set for convenience here, but in future studies, more observed data must be gathered to enable identification of these parameters by more rigorous methods.

The next task is to find α. This is difficult because α is considered to vary greatly with river characteristics and with particle diameter, and it is also difficult to obtain measured data. Therefore, the established values of β and Q_c were used in the following equation to match the estimated and measured total sedimentation volumes, and the value of α was calculated backwards.

$$\alpha_j = \sum_{i=1}^{N} [V_{S0j}]_i \Big/ \sum_{m=1}^{N} \sum_{k=1}^{n_m} \gamma_{jk} \int_k Q^{\beta_j} dt \tag{6}$$

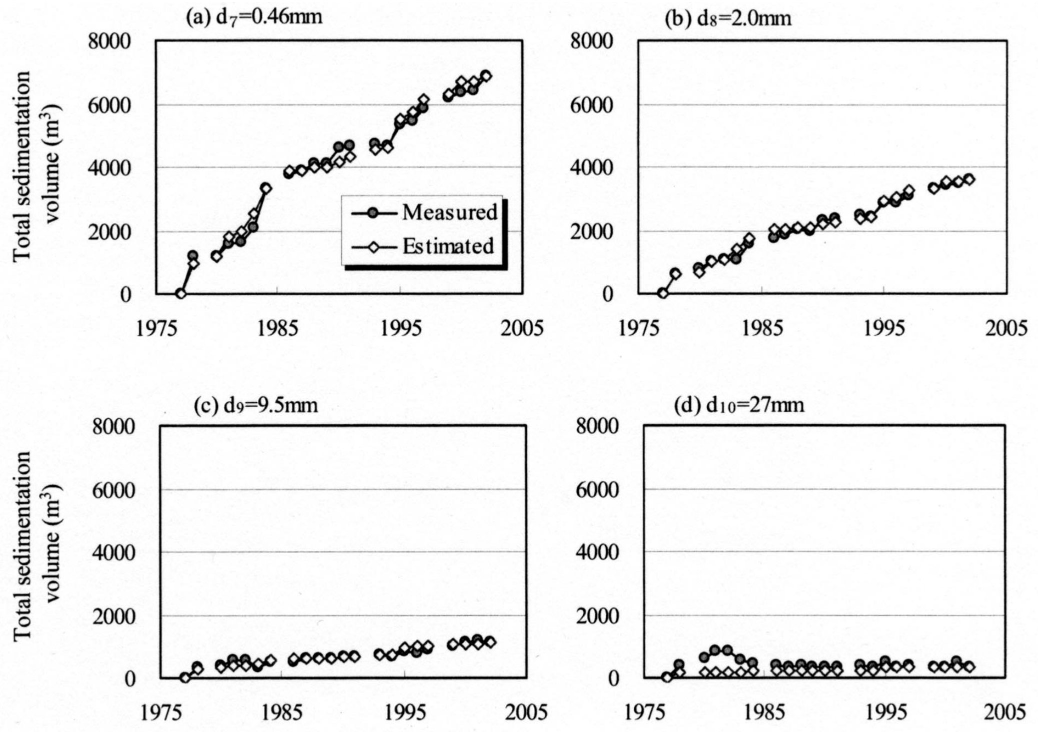
The measured data used for this exercise was the annual volume by size class $[V_{S0j}]_i$ in Eq.(4), but there were some years in which rather large sediment volumes occurred in spite of the absence of flooding, or other unusual events occurred that obscuring any correlation between $[V_{S0j}]_i$ and discharge. The reasons for this are unclear, but the accuracy of surveyed data for sedimentation is questionable because the number of boring and measuring points was not enough for the scale of the reservoir. Therefore, a five-term power series Q^m ($m = 1 \sim 5$) was constructed and integrated for comparison with the annual sedimentation record and Q^m. Years for which almost no correlation was shown with any value of m were not used for identifying values of the variables. As a result of the data selection, we chose the measured data from 21 of the 29 years for which data was collected. The neglected sedimentation volume is not large; it represents only a small fraction of the total amount.

The annual sedimentation volumes originating from the Sabaishi and Ishiguro Rivers were distinguished as follows. Above the junction, they were distinguished by their geographic locations within either the Sabaishi valley or the Ishiguro valley. Downstream of the junction, the contributions from the two rivers were distinguished by the ratio of sediment volume at each particle class for each valley.

Table 3 shows the identified parameters, and Fig. 9 shows the estimated results for the typical particle classes in the Ishiguro River. The figures compare annual changes in accumulated sediment volumes. The blanks in the graph represent years for which data were not chosen, as described above. The calculations were good approximations of the measurements for the d_7 (0.46 mm), d_8 (2.0 mm), and d_9 (9.5 mm) classes. There was a period during which the measured accumulated volumes of d_{10} (27 mm) were decreased. This

Table 3 Identified values of α, β for the completely trapped sediment

Class #	Typical (mm)	Sabaishi R. α	Sabaishi R. β	Ishiguro R. α	Ishiguro R. β
7	0.46	4.9×10^{-4}	1.0	3.1×10^{-6}	2.0
8	2.0	1.1×10^{-4}	1.0	8.5×10^{-7}	2.2
9	9.5	3.1×10^{-6}	1.6	1.7×10^{-8}	3.0
10	27	9.0×10^{-12}	5.0	2.8×10^{-12}	5.0
Critical discharg		Q_c=12m³/s		Q_c=12m³/s	



Figs. 9 Estimated results for the completely trapped sediment (example: Ishiguro R.)

could not be expressed, but since this class makes up a very small portion (0.1%) of the total volume, this discrepancy was considered negligible.

3.5 Estimation of sediment discharge for the partially tapped particles

3.5.1 Verification of numerical simulation model

The partially trapped sediment was also estimated using Eq.(1). But the trap efficiency γ had to be estimated before the parameter identification.

A numerical model was made to estimate γ. The basic equations and main parameters are shown in Table 4. The model is a 1-D unsteady flow model with SS transport equations for typical size classes. Settling velocity of sediment was calculated using the Rubey's equation, and we assumed that resuspension of settled SS did not occur. The calculations were carried out using the MacCormack scheme.

We attempted to reproduce the September 2003 flood in order to demonstrate the validity of the model. In this simulation, a time series of SS concentration is required as a boundary condition at the upstream end. Here, the boundary data was given using Eq. (1), whose coefficients were determined from the data observed during the same flood. Table 5 shows the values for α and β obtained from the observed data mentioned above, and Fig. 10 shows the relationship between the flow rate and the sediment discharge in the Sabaishi River for each size class given in Table 5. The number of the classifications was limited to 4 in order to make easier to see the trend. The suspended load for all of the classifications showed a relatively good correlation with the flow rate.

Figures 11 compare the calculation results with the observational data. These calculations were carried out by inserting α and β for each typical diameter given in

Table 4 Basic equations and parameters for simulation model

	Basic Equation	Parameter
Continuity equation	$\frac{\partial A}{\partial t}+\frac{\partial Q}{\partial x}=0$	Δx=30(m)
Equation of flow motion	$\frac{\partial Q}{\partial t}+\frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right)=-gA\left(\frac{\partial H}{\partial x}+\frac{n^2\lvert Q\rvert Q}{R^{4/3}A^2}\right)$	Δt=0.1(s) n=0.03
Equation of SS transport (each size)	$\frac{\partial(\overline{C}_jA)}{\partial t}+\frac{\partial(\overline{C}_jQ)}{\partial x}=B_{su}w_{sj}\overline{C}_j$	$B_{su}=A/h$

Note) A: Flow cross section; Q: Discharge; H: Water level; R: Hydraulic radius; $\overline{C}_j$: SS concentration of size class (j) averaged in A; h: water depth; n: Manning roughness coefficient

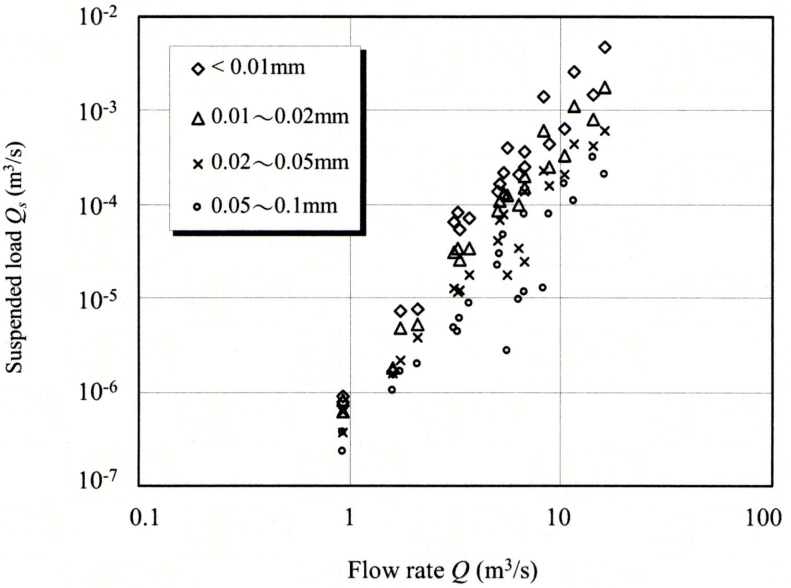


Fig. 10 Relationship between flow rate and suspended load for each particle classes (example: Sabaishi R.)

Table 5 Values of α, β obtained from the observed data

Size range (mm)	Sabaishi R. α	Sabaishi R. β	Ishiguro R. α	Ishiguro R. β
- 0.01	$1.2x10^{-6}$	3.0	$4.6x10^{-7}$	3.3
0.01 - 0.022	$9.2x10^{-7}$	2.8	$4.8x10^{-7}$	3.0
0.022 - 0.048	$5.7x10^{-7}$	2.6	$3.7x10^{-7}$	2.9
0.048 - 0.10	$3.4x10^{-7}$	2.3	$2.9x10^{-7}$	2.7

Table 2. Fig. 11(b) confirms that SS (circle mark) released from the dam is generally well reproduced by the model (see thick solid line on Fig. 11(b)). The predicted changes in the concentrations of the size classes in Fig. 11(c) also show relatively good consistency with the observed values, except for the periods of low SS concentration. Release of all the particles d_1 – d_4 from the dam was observed and predicted.

The model used here is comparatively simple, but the results described above indicate that the model provides an effective approximation of the trap efficiency at the Sabaishigawa Dam.

The observations presented in Fig. 11(c) indicate that the relatively coarse component (d_4) continued to be released after flooding. A factor that may have influenced this is the gate operation at the Sabaishigawa Dam. The dam's outlet facilities include a crest gate (elevation 135.00 m) and a hollow jet valve (normal outlet conduit, portal elevation 129.75 m). As is clear from the longitudinal topography shown in Fig. 4, the sediment just above the dam reaches almost to the level of the intake of the normal outlet conduit. The releasing water carries a large fraction of relatively coarse sediment, which settles near the surface of the sedimentation. It is likely that during the latter period of flooding, this would make up the dominant portion of the suspended load exiting the conduit, and this would explain the appearance of such high volumes at that time.

3.5.2 Dominant factors in trap efficiency

Although the numerical model described above is comparatively simple, it still requires much time and labor to be used for simulating historical floods. Therefore, in the alternative procedure used in this study, dominant parameters of γ were considered reservoir shape, flood condition, α and β. SS simulations were made while these parameters were set at several values in order to obtain a simple expression for estimating γ for each typical particle class (Table 2).

Before beginning detailed descriptions of this procedure, let us first outline the relationship between γ and the parameters α, β.

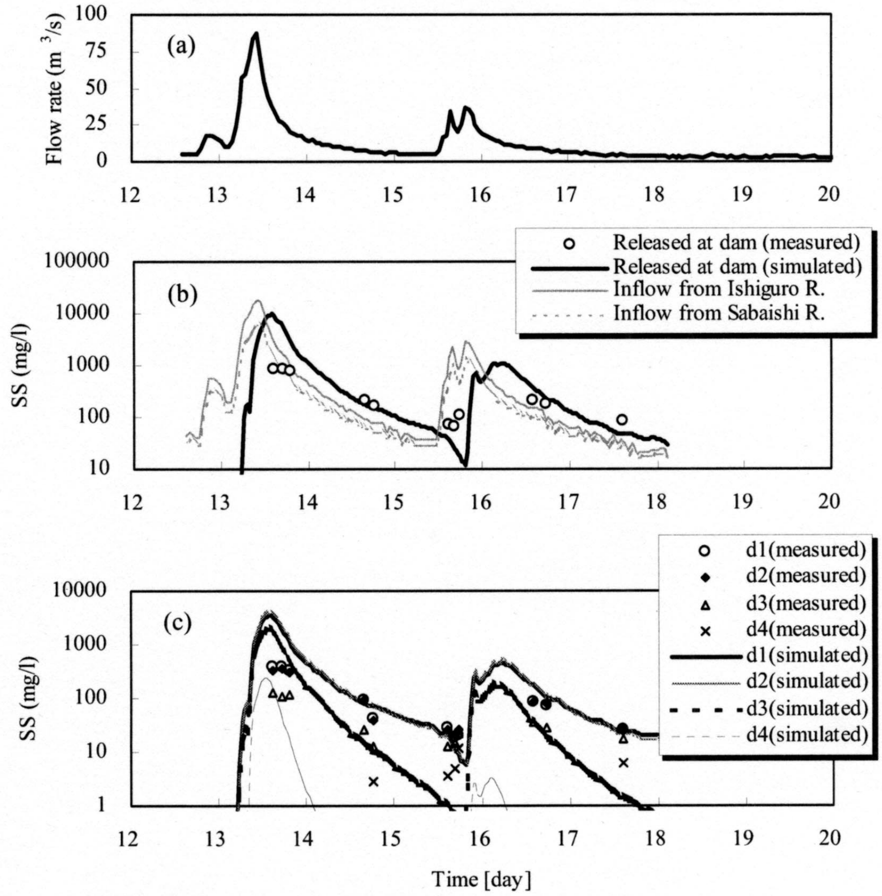
Let us consider the trap efficiency γ_{jk} for particles of size class (*j*) during flood (*k*). First, the volume of SS entering the reservoir per unit time Q_{sj} is expressed by Eq.(1).

Next, let us consider the phenomena of trapping in a reservoir which is assumed to have a uniform, rectangular channel of length L, filled from the upstream end at the rate Q by turbid water containing uniformly distributed sediments with a concentration C_j. The trapping occurs here only as a result of settling of the SS. If it is assumed that the SS concentration does not change greatly as the water moves over distance L, all the way through the reservoir, the trapped volume per unit time over distance L, ΔV_{sj}, is given by the following:

$$\Delta V_{Sj} = A_0 w_{Sj} \overline{C}_j = A_0 w_{Sj} \alpha_j Q^{\beta_j - 1} \tag{7}$$

Here, A_0 is the area of the river bed over length L.

If the inflow Q is assumed to change gradually over time while the SS concentration in the channel changes with time while remaining spatially uniform, the trapping efficiency γ_k for flood wave (*k*) is given by Eqs.(1), (7):



Figs. 11 Verification of numerical model: (a) the observed inflow rate; (b) the comparison between the observed and calculated total SS concentrations of released water from the dam; and (c) the comparison between observed and calculated SS concentrations with typical diameters in the released water. (b) also shows the time series for the calculated incoming SS concentrations based on the equation derived from the observed data as shown in Fig.10.

$$\gamma_{jk} = \frac{\int_k \Delta V_{Sj} dt}{\int_k Q_{Sj} dt} = A_0 w_{Sj} \frac{\int_k Q^{\beta_j - 1} dt}{\int_k Q^{\beta_j} dt} \tag{8}$$

From Eq.(8), it is observed that α can be removed, and has no effect on γ. In contrast, β appears to have no affect on γ as long as Q is constant, but once Q varies with time, as happens during a flood, γ changes in response to β.

The above summary is a simplified model of the phenomenon of sediment trapping in a reservoir, and may be of only approximate application to a general case, but it can be instructive in terms of the relationship between α, β and γ.

3.5.3 Calculation of trap efficiency

As stated at the beginning of the previous section, the SS simulations were conducted with the reservoir shape, flood condition, α and β varied. The equations for simply calculating γ were derived from these simulated results. As revealed in 3.5.2, almost no influence from α is seen on γ, so that simulated results for α are omitted below.

Hydrological data for ten floods with different peak discharge were used for the SS simulation. Three values of β were chose as a reference of the observed values in Table 5. We explain a procedure for deriving a simple expression below, using the Sabaishi River as an example.

Figure12 presents an example of the simulated relationship between γ and β under the same flood and reservoir's topographic conditions. For all the particle size classes, γ decreases monotonically with increase in β. This is because the incoming water discharge changes in an unsteady manner, as shown in Eq.(8).

Figure13(a) and (b) show the simulated relationship between γ and flood index when β is fixed at a value of 2.7. The turnover ratio of a reservoir during a single flood event R_f is known to be a useful index of γ and flood characteristics[5], but for this study, the mean residence time T_r, a parameter which incorporates time, was also examined. Here, R_f is the total volume of entering water divided by the reservoir water volume just before the flood, and T_r is the volume of reservoir water just before the flood divided by the mean discharge during the flood period. The results indicate that both R_f and T_r have a relatively consistent relation with γ, but T_r shows a stronger correlation with γ; it

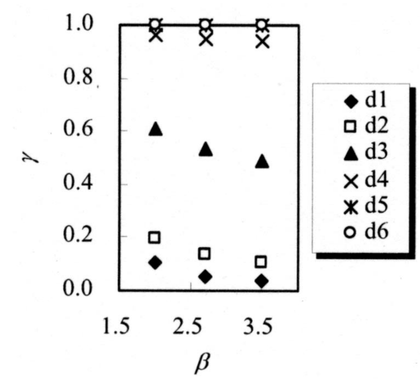


Fig. 12 Relationship between α and β
(fixing flood and topographical conditions)

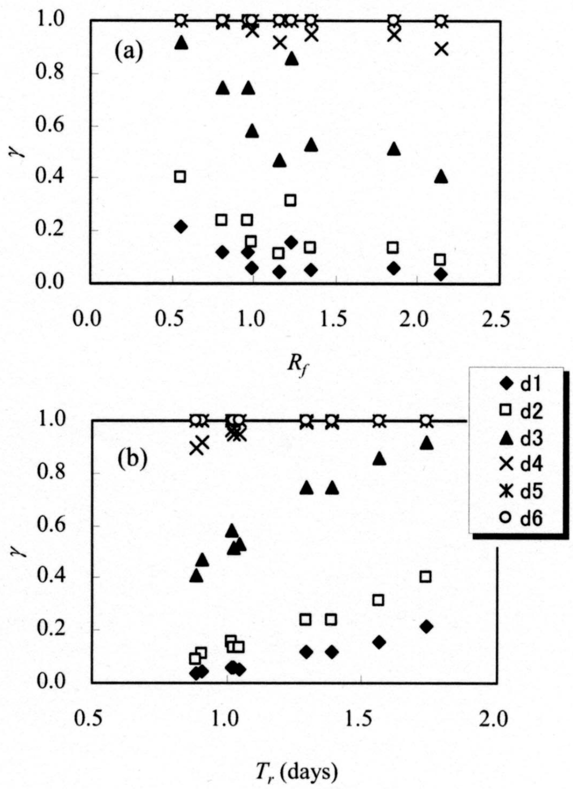


Fig. 13 Relationship between γ and flood indexes (β=2.7)
(R_f : Turnover ratio per flood event;
T_r : mean residence time)

was therefore decided to use T_r as an index of γ at Sabaishigawa Dam.

Figures 12 and 13 demonstrate the linear relationships of γ with β and T_r, so it seems likely that γ at typical diameters can be expressed as a linear function of β and T_r. Therefore, the following expression was conceived as a simple equation for estimating γ, and the coefficients were calculated using the least-squares approximation.

$$\gamma = a \cdot \beta + b \cdot T_r + c \tag{9}$$

Coefficients a, b, and c can be determined for each particle size class and reservoir shape.

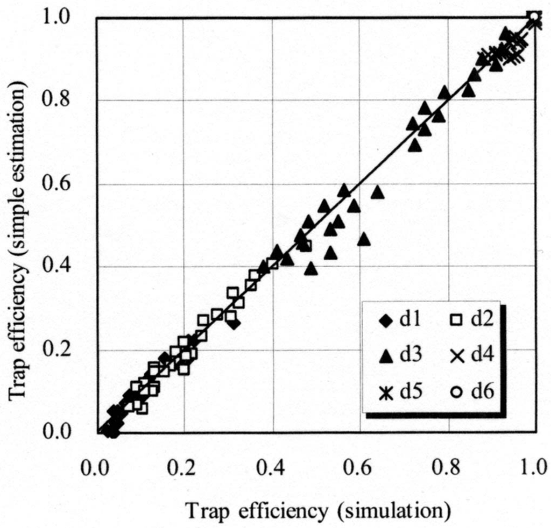


Fig. 14 Comparison of values predicted by simple estimation and numerical simulation

Table 6 Coefficients of equation (9) for simply estimating trap efficiency (example: Sabaishi R.)

Coeff.	Typical particle size class #					
	1	2	3	4	5	6
a	-0.058	-0.062	-0.048	-0.007	0.000	0.000
b	0.200	0.340	0.580	0.110	0.000	0.000
c	0.024	-0.034	0.077	0.850	1.000	1.000

Figure 14 compares the values of γ determined by the simple method outlined above with those determined by the numerical simulations. Table 6 shows the derived coefficients a, b and c to be used by the simple equation. There was a scatter in a certain particle, but the simple equation accurately reproduced the trap efficiencies derived from the numerical simulation. It should be noted that the above results apply only to the Sabaishi River section. a, b and c were also calculated for the Ishiguro River section, and showed the same level of the coefficient values as the Sabaishi River section, and the final results for the trap efficiency also had the same level of accuracy as in Fig. 14.

3.5.4 Data selection

In the Sabaishigawa Reservoir, the water level is occasionally lowered during the spring thaw, during which the accumulated sediment is entrained back into the current, resulting in release of the accumulated fine sediment downstream. However, our simulation model is not capable of considering the resuspension of fine sediment by means of "water level lowering operation", so in this study, the years during which the phenomenon was occurred were excluded from the analysis. Accordingly, actual sedimentation data from only 6 calendar years were used.

3.5.5 Parameter identification

Values of α, β and Q_c were determined in order to obtain minimum residues of the following equation:

$$\varepsilon_j = \sum_{i=1}^{N} \left| [V_{S0j}]_i - [V_{Sj}]_i \right| \tag{10}$$

The search ranges of α and β were selected on the basis of the previous observational reports and the additional survey in the dam. In the present study, α and β were searched for in the ranges $1.0 \times 10^{-6} - 8.0 \times 10^{-6}$ and 1.0 – 4.0, respectively, and Q_c was set to flow rates of 0 – 20 m^3/s.

3.5.6 Estimation results

Values of the coefficients identified from Eq.(10) are shown in Table 7. The α and β values are similar to those of observed SS presented in Table 5, so seem to be appropriate. The measured values reveal the trend of increasing β with decreasing sediment size class; the results in Table 7 show the same trend. There are no published survey-based data on the effects of particle size on the relationship between SS and flow rate, and it is therefore unknown why β showed the trend described above. Data must be gathered at other dams in the future in order to answer this question. Also, the critical flow Q_c was lower for fine sediment than for coarse one. Most of the coarse is supplied from river bed, while most of the fine is introduced by erosion of inclines by rainfall and surface flow. The different sources of these sediment types are reflected in different Qc values.

Figure 15 compares the estimated and measured volumes of the typical size classes in the Sabaishi River. This does not provide detailed data for annual changes in sedimentation, but on the whole the estimates reproduced the observed data well. The annual trap efficiency for the partially trapped sediment was 20% – 60% (Table 8). This confirms that the influence of the trap efficiency of the fine sediment cannot be neglected when estimating the fine sediment discharge.

Table 7 Identified values of α, β for the partially trapped sediment

Class #	Typical size (mm)	Sabaishi R. α	Sabaishi R. β	Ishiguro R. α	Ishiguro R. β
1	0.0035	6.1×10^{-6}	3.3	5.6×10^{-6}	3.0
2	0.0059	2.0×10^{-6}	3.2	2.5×10^{-6}	2.9
3	0.015	3.4×10^{-6}	2.6	4.3×10^{-6}	2.4
4	0.032	3.0×10^{-6}	2.4	6.1×10^{-6}	2.1
5	0.069	1.4×10^{-6}	2.4	3.6×10^{-6}	2.2
6	0.16	2.1×10^{-6}	1.9	3.7×10^{-6}	2.1
Critical discharge		Q_c=4m^3/s		Q_c=4m^3/s	

4. Summary

In this study, we propose an estimation method of non-uniform sediment discharge using actual results of dam reservoir sedimentation. The method includes developing a calculation technique for trap efficiency of fine sediment and systematizing estimation processes of field surveys and data analysis. The validity of this method is confirmed through the case of the Sabaishi-gawa Dam, Nigata Prefecture. A flowchart of the proposed procedure is shown in Fig. 1, Chapter 2.

Our estimates of sediment discharge at the Sabaishi-gawa Dam are reasonably accurate, but this is the only

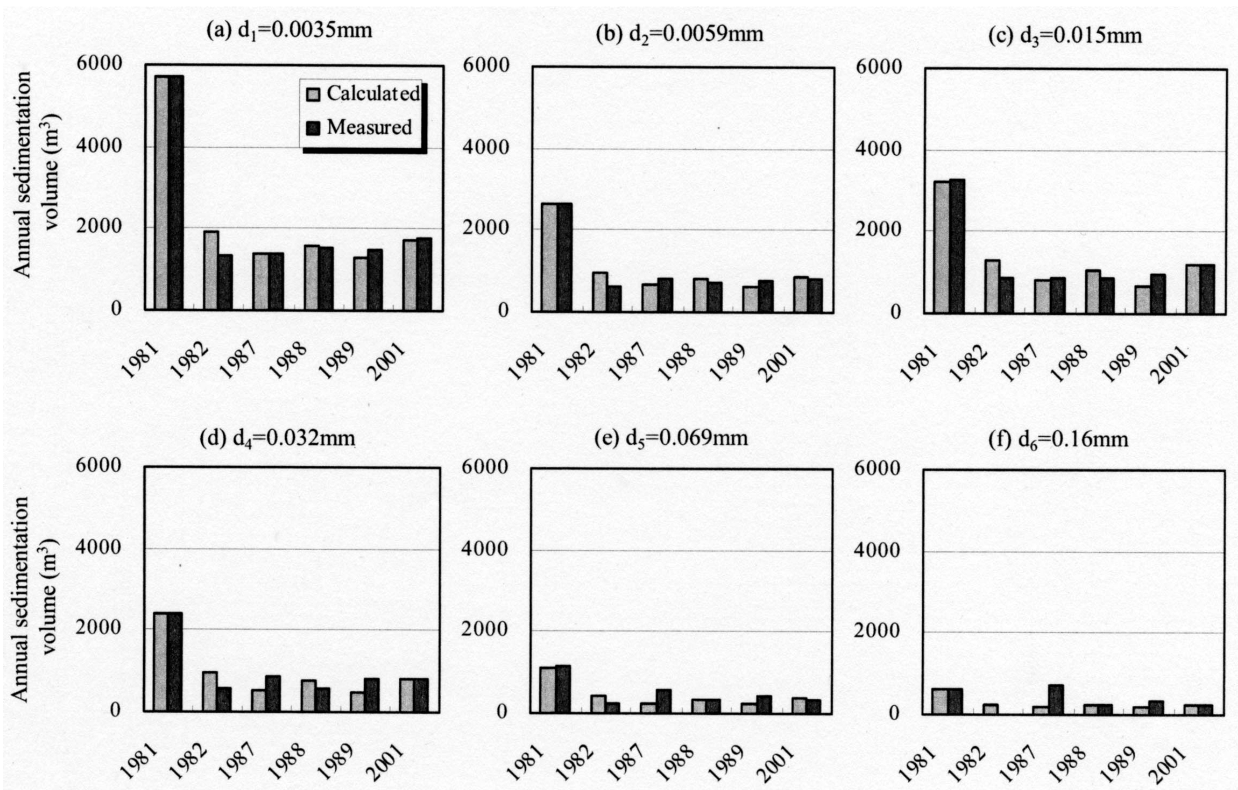


Fig. 15 Estimated results for the partially trapped sediment (ex: Sabaishi R.)

Table 8 Annual trap efficiencies for the partially trapped sediment (estimated results)

	Sabaishi River			Ishiguro River		
	Q_{sin} (m^3/yr)	V_s (m^3/yr)	γ	Q_{sin} (m^3/yr)	V_s (m^3/yr)	γ
1981	$5.2x10^4$	$1.6x10^4$	0.30	$7.7x10^4$	$2.8x10^4$	0.36
1982	$2.5x10^4$	$5.8x10^3$	0.23	$3.3x10^4$	$1.1x10^4$	0.34
1987	$8.0x10^3$	$3.8x10^3$	0.48	$1.4x10^4$	$7.8x10^3$	0.54
1988	$1.5x10^4$	$4.7x10^3$	0.33	$2.3x10^4$	$9.7x10^3$	0.43
1989	$6.0x10^3$	$3.5x10^3$	0.58	$1.2x10^4$	$7.4x10^3$	0.63
2001	$1.3x10^4$	$5.3x10^3$	0.40	$2.3x10^4$	$1.1x10^4$	0.46

Note) Q_{sin}:Annual sediment inflow volume; V_s:Annual trapped volume; γ:Annual trap effciency.

dam to which this method has been applied. The method will be used to model sedimentation at other dams in the future to further verify its applicability.

The modeling procedure described in this study takes into account both coarse and fine sediments, and is a promising method of estimating the loads of a wide range of sediment size classes. The method for obtaining a relationship between the flow rate and the sediment discharge for a dam sedimentation have not been systematized yet, and we hope that this method will prove useful for designing sedimentation measures and general sedimentation management.

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River Environment Improvement Method by Sediment Transport Experiment in the Downstream Reach of a Dam

Hiroshi Ogawa
Director, Shimokubo Dam Operation and Maintenance Office, Japan Water Agency, Japan

Akihiro Kanayama
Senior Engineer, Shimokubo Dam Operation and Maintenance Office, Japan Water Agency, Japan

1. Introduction

The Shimokubo Dam is a multipurpose dam managed by the Japan Water Agency. The objectives of the dam are flood control, river discharge maintenance, water supply (domestic and industrial use), and hydropower generation. 36 years have past since the beginning of dam operation. Thus, there are various themes for the dam, such as reservoir function maintenance measures against sediment flowing into the reservoir and measures against riverbed degradation in the downstream reach of the dam because of interrupted sediment transport by the dam.

An approximately 1.5km section of the downstream reach of the dam has a particularly beautiful landscape designated as a scenic spot and natural monument called as Sanba Seki Kyo (gorge). Landscape deterioration has been conspicuous due to the riverbed degradation, diminished sandbars, and weakened cleansing effects of sediment transport impinging on the rock (sediment transport polished rock surface by impinging action when the material flows with water).

This paper introduces the sediment transport experiment being conducted since 2003 as a measure to improve the landscape of the river downstream of the dam and as measures for river environment preservation and reservoir function maintenance.

2. Effects of Shimokubo Dam Operation to the Sediment Transport System of the Kanna River

The Shimokubo Dam is built on the Kanna River that is one of the right-side tributaries of the Tone River system. The dam's catchment area is 323 km^2 that occupies 79% of the Kanna River's entire catchment area 407 km^2. The area of the mountainous region that produces riverbed material within the entire catchment area of the river is 374 km^2. This area's 86% is included in the catchment area of the Shimokubo Dam.

The continuity of the sediment transport system in the downstream reach of the Shimokubo Dam was cut off by the sediment deposit in the reservoir created by the Shimokubo Dam. As a result, severe riverbed degradation has occurred in the river section from the dam to the tip of the alluvial fan created by the river system. Judging from those photographs that were taken before dam construction, there are some river sections whose riverbeds have been degraded almost 5m deep. The riverbed material in the river immediately downstream of the dam has become coarser. In the section of "Sanba Seki Kyo (gorge)," riverbed gravel has disappeared thereby causing the deterioration of landscape. (Fig. 1)

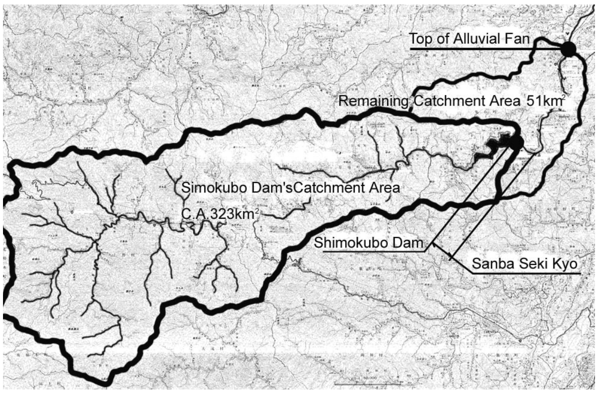


Fig. 1 Kanna River and the Catchment Area of Shimokubo Dam

3. Condition of Sediment in the Reservoir Created by Shimokubo Dam

The Shimokubo Dam has been operated 36 years after completing construction. There is a tendency that the amount of sediment in the reservoir extremely increases after a relatively large flood. The amount of sediment accumulated by the end of 2004 was $8,057x10^3$ m^3 that was 81% of the reservoir's design sediment capacity $10,000x10^3$ m^3. The actual sedimentation is progressing with a speed 2.2 times faster than that of initially predicted. (Fig. 2)

4. Present Condition of Sanba Seki Kyo and the Purpose of Sediment Transport Experiment

Deterioration of the Landscape of Sanba Seki Kyo has progressed due to the reason that the sediment transport system of the river is cut off by the Shimokubo Dam as follows:

① Riverbed degradation, coarsened riverbed material, and diminished sediment material on the riverbed.
② Deteriorated cleansing effects

Photographs taken before and after dam construc-

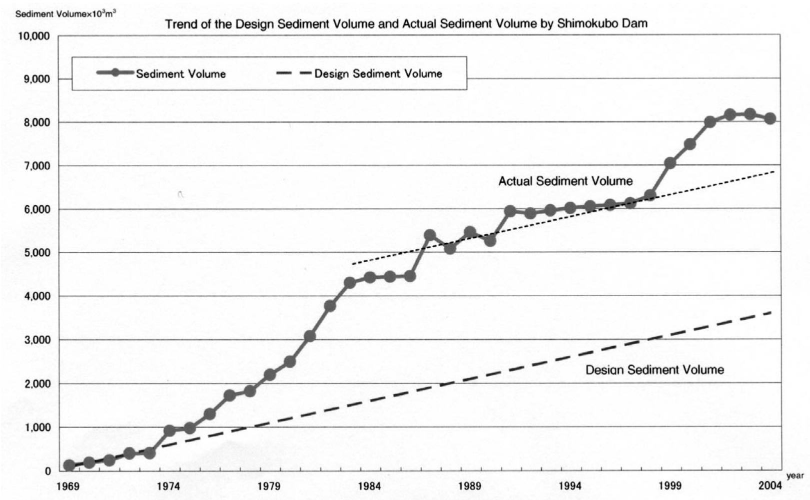
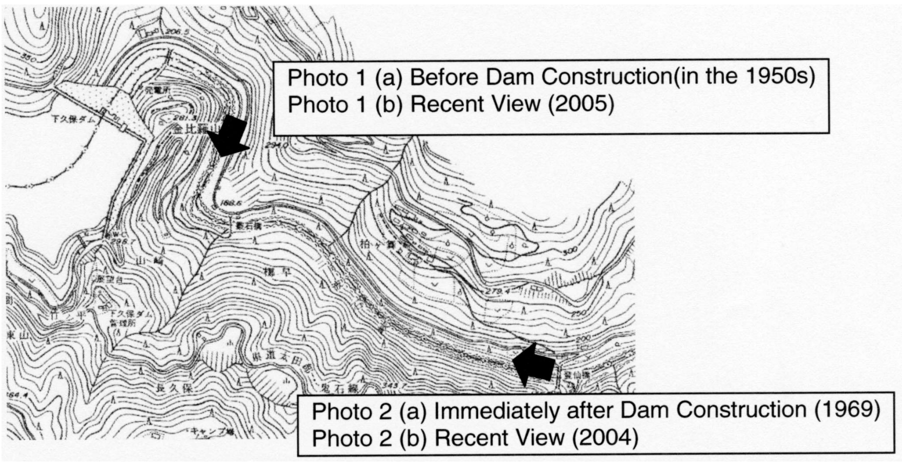


Fig. 2 Change in Sediment Volume

tion were compared with each other in order to learn the conditions of the diminished sediment material on the riverbed and the amount of riverbed degradation then set up landscape improvement target.

There were gravel areas on the riverbed in Sanba Keki Kyo before dam construction as seen in Photo 1 (a). However, after dam construction, the sediment material on the riverbed was washed away by floods and no sediment material has been supplied and, as a result, the riverbed has been degrading and the gravel areas recently disappeared as seen in Photo 1 (b). When comparing Photo 2 (a), showing the river condition immediately before dam construction, with Photo 2 (b) of recent river condition, it can be estimated from the size of the large piece of rock in these photos and site survey result that the riverbed has degraded approximately 2 m. For this reason, a landscape improvement target is set up to restore the diminished gravel areas to the original state and the degraded riverbed up to an approximately 2 m height from the present elevation (called the riverbed restoration target).



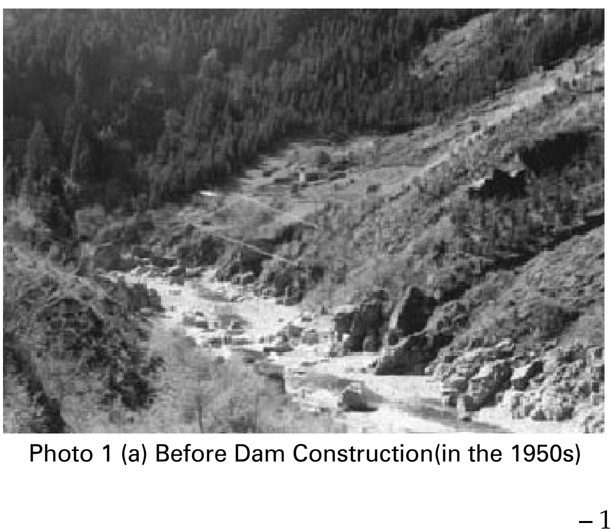


Photo 1 (a) Before Dam Construction(in the 1950s)

Photo 1 (b) Recent View (2005)

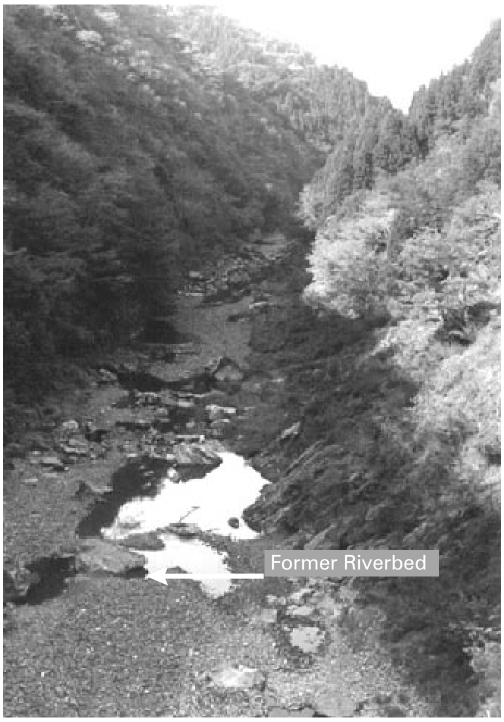


Photo 2 (a) Immediately after Dam Construction (1969)

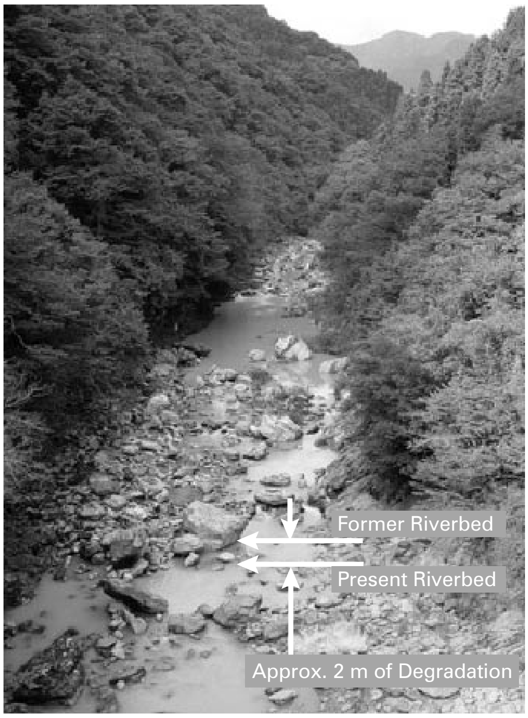


Photo 2 (b) Recent View (2004)

The cleansing effects of the sediment transport in the river should be learned by comparing photographs to be taken before and after conducting the sediment transport experiment.

5. Method of the Sediment Transport Experiment

The sediment transport experiment has been conducted since 2003. In the experiment, sediment material deposited in the sediment trap dam within the reservoir is taken, transported to and dumped into the downstream of the dam by dump trucks. The dumped sediment material is washed away by discharge from the reservoir. The sediment trap dam is located 8.5 km upstream of the Shimokubo Dam. When the reservoir level rises higher than the limited water level of E.L. 283.8 m during the flood season (July through September), sediment material can be mined. (Photo 3-6, Fig. 3)

6. Sediment Transport Experiment Conducted

The first sediment material dumping was conducted on July 16, 2003. An approximately 1,000 m^3 of the dumped sediment material was washed away by the 5-hour period reservoir discharge having about 100 m^3/s of the maximum that was released from the midnight of July 26 in order to handle the flood caused by the front in the July period.

The second sediment material dumping was conducted on October 22, 2003. An approximately 200 m^3 of the dumped sediment material was washed away by the reservoir discharge having a maximum of approximately 40 m^3/s for a 9-hour period that was released during a period of October 10 through 15 to handle the flood caused by Typhoon No. 22 in 2004. Approximately 800 m^3 of the remaining sediment material was washed away by the reservoir discharge having a maximum figure of about 300 m^3/s for a 2-hour period released during a period from October 19 to 26 to handle the flood caused by Typhoon No. 23 in 2004.

The third dumping of an approximately 2,000 m^3 sediment material was carried out on March 25, 2005. In this experiment the sediment material was pushed till the middle of the river so that the material could be washed away even by a relatively small amount of reservoir discharge. For the purpose of crest gate inspection, an approximately 5 m^3/s one-hour period reservoir discharge was conducted on May 8, 2005. A small amount of the sediment material was washed away by this discharge. (Table 4, Photo 8, 9)

7. Restoration of Riverbed Material

Restoration of riverbed material can be seen in the river section downstream of the sediment material dumping place due to the effects of the dumped sediment material. However, it is feared that the riverbed material restored only by the above-mentioned dumping method and the amount dumped might be washed away when the river section receives a magnitude of reservoir discharge that handles a certain size of floods or a design maximum discharge (discharge in the range of 500 m^3/s to 800 m^3/s).

It would be necessary to examine the shape of piled up material and the amount of material to be dumped to suit the amount of reservoir discharge and the discharge duration. (Photo 9, 10)

Photo 3 Mining of Sediment Material

Photo 4 Dumping of Sediment Material

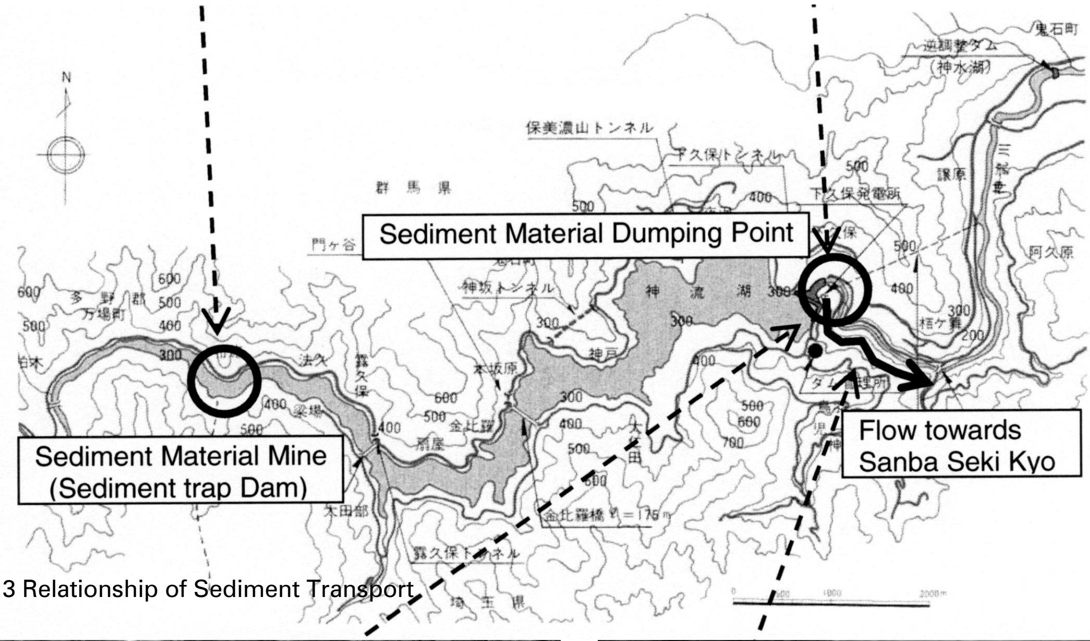


Fig. 3 Relationship of Sediment Transport

Photo 5 Sediment Transport by Reservoir Discharge

Photo 6 Sediment Material Deposited on Riverbed

Table 4 Result of Sediment Transport Experiment

Dumped/ Washed Away	Date	Amount Dumped Material	Reason for Reservoir Discharge	Maximum Discharge and Period	Amount Washed Away
Dumped	16 Jul 2003	1,000 m^3			
Washed Away	26 Jul 2003		Front	Approx 100 m^3/s for 5 hours	Approx 1,000m^3
Dumped	22 Oct 2003	1,000 m^3			
Washed Away	10 through 15 Oct 2004		Typhoon No. 22	Approx 40 m^3/s for 9 hours	Approx 200m^3
Washed Away	19 through 26 Oct 2004		Typhoon No. 23	Approx 300 m^3/s for 2 hours	Approx 800m^3
Dumped	25 Mar 2005	2,000 m^3			
Washed Away	8 May 2005		Gate Inspection	Approx 5 m^3/s for one hour	Small amount

Photo 7 Flow Condition on 11 Oct 2004 (Q = 40 m^3/s)

Photo 8 Flow Condition on 22 Oct 2004 (Q = 100 m^3/s)

Photo 9 Riverbed before Sediment Transport Experiment (July 2003)

Photo 10 Present Riverbed Condition (May 2005)
Note: Restoration of riverbed material can be seen.

8. Cleansing Effect

It is considered that the cleansing effects of the river flow increased because of the reason that the river water contained dumped sediment material. Photos 11 and 12 show the riverbed conditions before and after conducting the sediment transport experiment. It can be seen that the previous dark surface of Sanba Seki became a shiny surface by the cleansing effect.

The cleansing effect is larger at the downstream side of rock than the upstream side that is hit by the river flow. Because river flow mixes sediment material in behind the rock with the turbulent water. Photos 13 and 14 show the upstream-side surface and downstream-side surface of rock. It can be seen the great difference in the cleansing effect. Photo 15 is the side view of the rock.

Photo 11 Sanba Seki Kyo in 1997

Photo 12 Sanba Seki Kyo in 2004

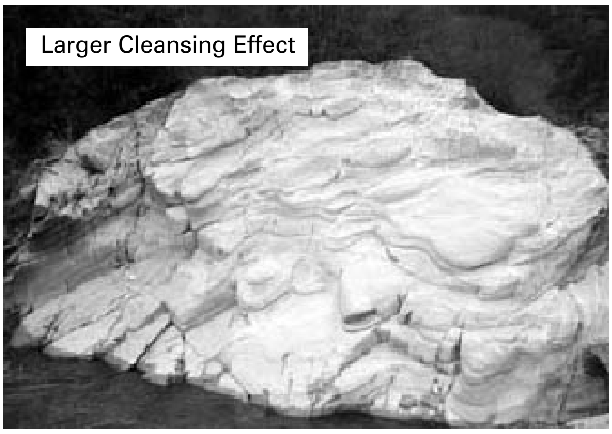


Photo 13 Downstream-side Surface

Photo 14 Upstream-side Surface

Photo 15 Side View of Rock

9. Result of Sediment Transport Experiment

Through the sediment transport experiment that has been carried out since 2003, the restoration of the riverbed and the improvement of the landscape by the cleansing effect of sediment transport are confirmed in some section of Sanba Seki Kyo (gorge). However, it is feared that restored riverbed might be washed away again depending upon the amount of reservoir discharge and the discharge duration. Thus, the continuous examination of the measure against riverbed degradation would be necessary in the future. It is considered that the cleansing effect of sediment transport will last long as long as the sediment transport is maintained.

The experiment result is periodically reported to area residents and they highly evaluate the landscape improvement effect.

10. Measures for Reservoir Function Maintenance and Sediment Transport Experiment

The Japan Water Agency plans to start a sediment transport experiment by taking into account continuous and concrete reservoir function maintenance measures from fiscal year 2005.

We are going to clarify the necessity and themes of sediment transport in view of both reservoir function maintenance and the preservation of the river environment downstream of the dam.

And these themes are planed to be discussed with river management agencies, water users and area residents in Kanna River Sediment Transport Committee.

Monitoring survey of river environment will be carried out by taking into the advices and opinions of scholars and specialists in order to learn the quantitative effects of sediment transport and submit the obtained data to the above-mentioned Kanna River Sediment Transport Committee.

11. Monitoring Survey Plan

It is planned to conduct the monitoring surveys of the river's plan and cross sections, riverbed conditions, river bank vegetation, fish species and benthos in order to evaluate the effects of the sediment transport experiment. The river's plan will be monitored by taking aerial photographs from a helicopter. The river flow, riverbed conditions, river bank vegetation (physiognomic vegetation), etc. will be clarified on the aerial photograph maps and basic environmental information maps (photographs) will be prepared in order to visually learn their secular changes and make the maps a comprehensive data book.

In view of safety against floods, cross section surveys will be conducted in those river sections that may need to be monitored for possible flooding. The changes of these sections will be carefully investigated.

As an impact and response biotic indicator that may be influenced by the sediment transport experiment, fish species and benthos surveys will be carried out. As for fish species, by asking scholars and specialists and fishery related people, it was decided upon to study Ayu fish (Japanese sweet fish), Ugui (dace), Kajika (Japanese freshwater bullhead), and bottom fish. As for benthos, it is planned to classify the aquatic insects by the living patterns and feed and, as a result, riverbed environment will be evaluated based on the changes in the living conditions of those insects.

12. Conclusion

Measures against the sedimentation in a reservoir to maintain reservoir functions are big issues for many dams. We consider that, in addition to the ordinary method to use sediment material for concrete aggregate, the method to move deposited sediment material into rivers downstream of dams to improve river environment would be extremely effective means as a measure against sediment problems.

Measures against Sedimentation in the Makio Dam Reservoir

Tsuneo Midorikawa
Manager, Aichi Canal Project Management Office, Japan Water Agency, Japan

Shinobu Ishimura
Section Chief, No. 2 Design Section, Chubu Regional Bureau, Japan Water Agency, Japan

1. Introduction

The Makio Dam is located at the foot of Mt. Ontake (3,067 m above sea level) in Nagano Prefecture. It is a rockfill dam constructed as a water resources facility of the Aichi Canal Project during the period from 1957 through 1961 by the Aichi Water Public Corporation. (The corporation merged in October 1968 with the Water Resources Development Public Corporation that became an independent administrative institution called the Japan Water Agency in October 2003).

In addition to the Makio Dam, the Aichi Canal Project consists of an approximately 112 km long main canal and a total of about 1,063 km long branch channels. This Water Project was implemented as a first comprehensive large-scale water resources development project in Japan to develop water resources and supply water for irrigation, drinking and industrial use purposes in Gifu and Aichi prefectures and, in addition, generate hydroelectric power by the Makio Dam. Since the opening of the system in 1961, it has greatly contributed to the development of the areas.

However, due to the eruption of Mt. Ontake in 1979 and the landslide on the slope of the mountain caused by the Nagano Seibu Earthquake in 1984 (close to the Makio Dam having a magnitude of 6.8 and the epicenter approximately 2 km deep), an extremely large amount of debris flew into the Makio Dam Reservoir thereby hindering the reservoir functions.

To solve the above-mentioned problems, when debris inflow into the reservoir caused by the earthquake settled down the Ministry of Agriculture, Fishery and Forestry (MAFF) conducted a survey during the period from 1991 through 1994 in order to restore reservoir functions and, as a result, set up a plan for measures against reservoir sedimentation problems. Based on this plan, the Japan Water Agency has been implementing the sedimentation countermeasure project since 1995 with the schedule to complete the project in Fiscal Year 2006.

The Forestry Agency and Nagano Prefectural Government constructed debris and sediment control facilities on streams coming out from the failed slope of Mt. Ontake by the previous earthquake to control the debris flow into the reservoir and planted trees on the devastated slopes. In addition, the volunteer groups in Ootaki Village where the dam is locating, and the people from beneficiaries of the Aichi Canal Project planted trees. As a result, the catchment area of the dam has been restored to the previous condition and the amount of sediment flowing into the reservoir has been reduced to almost the same level as it was before the earthquake.

This paper reports the outline of the sedimentation countermeasure project being implemented at the Makio Dam Reservoir (that once lost its storage function because of accumulated sediment) by the Japan Water Agency in order to restore its water storage function.

Photo 1 Panoramic View of the Makio Dam

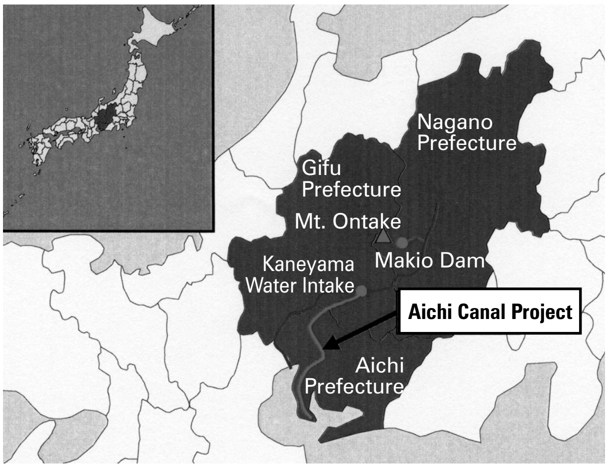


Fig. 1 Aichi Canal Project's Facility Arrangement and Outline

Table 1 Amount of Developed Water by Makio Dam for Each Use Purpose

Use Purpose	Amount of Water Developed	Remarks
Agricultural Use	21.514 m^3/s	Farm land: Approx. 15,000ha
Drinking	2.594 m^3/s	
Industrial Use	6.41 m^3/s	
Hydroelectric	35,500 kw	Kansai Electric Corporation

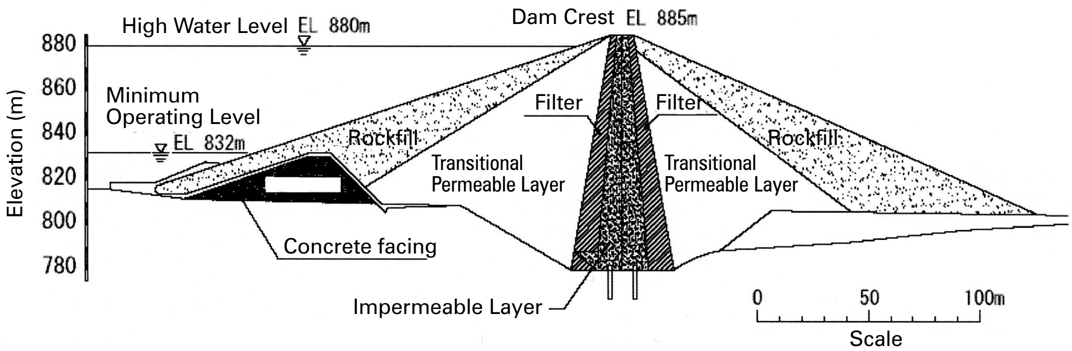


Fig. 2 Standard Cross Section of the Makio Dam

Table 2 Features of Makio Dam

List of the Features of Makio Dam			
Name of River:	Ootaki River of Kiso River system.	Effective Capacity:	68,000,000 m^3
Location:	Ootaki and Mitake villages, Kiso county, Nagano Prefecture.	Design Sediment Cap:	7,000.000 m^3
		Dam Type:	Central impervious core type rockfill
Catchment Area:	Approx. 304 km^2	Crest Elevation:	El. 885.0 m
Reservoir Area:	Approx. 2.47 km^2	Crest Height:	105 m
Normal Water Level:	El. 880.0 m	Crest Length:	264 m
Minimum Operating Level:	El. 832.0 m	Embankment Volume	3,615,000 m^3
Reservoir Capacity:	75,000,000 m^3	Spillway Capacity:	Max. 3,100 m^3/s

2. Outline of the Makio Dam

The Makio Dam is constructed on the Ootaki River, a tributary of the Kiso River, at a location approximately 120 km upstream of the Kaneyama Water Intake of the Aichi Water Project built on the left bank of the Kiso River in Yaozu-cho, Gifu Prefecture. It is a central impervious core type rockfill dam constructed by introducing American technologies. The dam's specific technological features are the use of the core material for the first time in Japan that contained an average of 60% of grain size larger than 4.4 mm in diameter and a very thin core width (ratio of the maximum core width to the embankment height is 1/4) comparing with other dams in Japan. In addition, another feature was the use of large sized construction equipment for embankment construction that was quite unusual at that time in Japan and the embankment construction speed that completed the dam in a one and a half year period.

The amount of water developed by the Makio Dam by each category is shown in Table 1. The embankment's standard cross section and the dam's features are shown in Fig. 2 and Table 2 respectively.

3. Amount of Sediment Inflow caused by the Nagano Seibu Earthquake

1) Nagano Seibu Earthquake

The Nagano Seibu Earthquake occurred in September 1984 that had a magnitude of 6.8 and its epicenter was in the southern slope of Mt. Ontake. The earthquake caused the failure of the slope of Mt. Ontake and a large amount of soil flew into the Denjyo River and the Suzugasawa Stream, the tributaries of the Ootaki River, and caused the debris flow into the Ootaki River. The earthquake caused 29 deaths and inflicted a tremendous amount of damage to public facilities, forests and housing. The largest slope failure occurred on the slope of Mt. Ontake at its eighth stage. It was called the Ontake Failure over a wide area of the length of 600 m and a maximum width of 480m. The amount of failed soil reached approximately 36,000,000 m^3 of the volume. Over 21,000,000 m^3 of soil flew into the 4.3 km section of the Ootaki River.

2) Amount of Sediment Flew into the Makio Dam Reservoir

A large amount of debris produced by the land failure on the slope of Mt. Ontake flew into the Ootaki River. The debris was entrapped by the narrow gorge

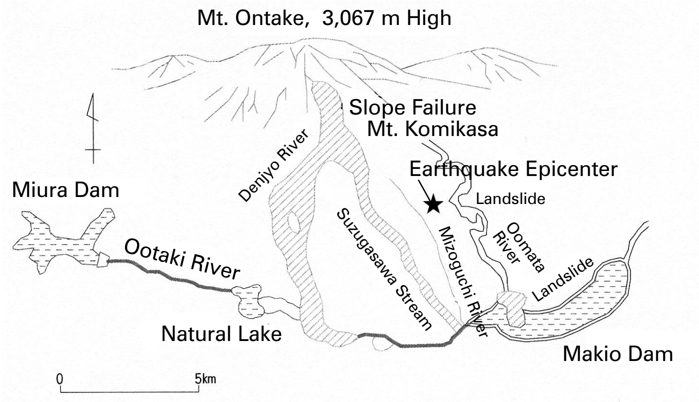


Fig. 3 Distribution of Debris Flow

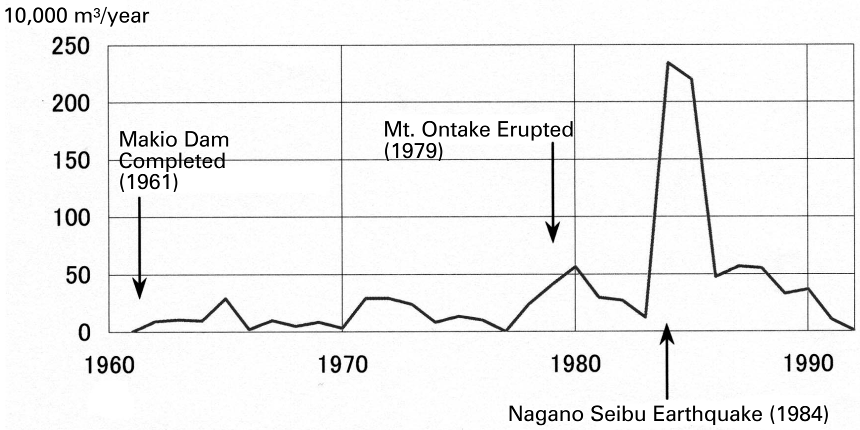


Fig. 4 Change in the Annual Inflow Amount of Sediment in the Makio DamReservoir

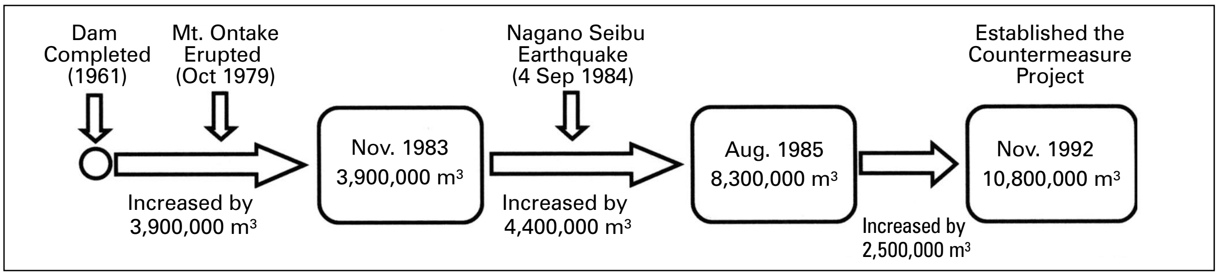


Fig. 5 Change in the Amount of Sediment Accumulated in the Makio Dam Reservoir

Photo 2 Condition of Deposited Sediment in the Makio Dam Reservoir (Upstream part of the reservoir bottom upgraded to the maximum of 15m by sediment deposit)

Photo 3 Slope Failure on Mt. Ontake

Photo 4 Sediment Deposited in Reservoir

of the Ootaki River and most of it settled in the river upstream of the narrow gorge. Thus, although some amount of debris flew into the Makio Dam Reservoir, the reservoir function was not hindered immediately after the earthquake. However, later on the debris accumulated at the narrow gorge flew into the reservoir during each storm. The amount of debris that flew into the reservoir reached 2,000,000 m^3 by the year after the earthquake and debris inflow still continued there after. The amount of sediment that flew into the reservoir reached 10,800,000 m^3 in 1992 that exceeded the design sediment storage capacity of the reservoir by 7,000,000 m^3 and hindered the storage function of the reservoir.

The distribution of debris flow caused by the Nagano Seibu Earthquake and the change in the annual inflow of sediment in the Makio Dam Reservoir are shown in Figs. 3 and 4 respectively. The change in the amount of sediment accumulated in the reservoir is shown in Fig. 5.

4. Sedimentation Countermeasure Project Plan

The Water Resources Development Public Corporation (presently an independent administrative institute, Japan Water Agency) held a series of meetings concerned to the concrete plan for the restoration of the functions of the Makio Dam with various agencies related to the Makio Dam including the Ministry of Agriculture, Fishery and Forestry (MAFF) immediately after the Nagano Seibu Earthquake. As a result, when the amount of sediment inflow was stabilized in Fiscal Year 1991, MAFF conducted the planning and study of sedimentation countermeasures, prepared the design of the measures and, as result, established the sedimentation countermeasure project in Fiscal Year 1994.

1) Distribution and Characteristics of Sediment in the Reservoir

The distribution of the amount of 10,800,000 m^3 sediment deposited in the reservoir by elevation was as listed in Table 3. 3,820,000 m^3 of the sediment, i.e. 35% of the total deposit amount, occupied the dead storage of the reservoir below the elevation of EL. 832.0 m. 6,980,000 m^3 of the sediment, that is, 65% of the total amount, occupied the effective capacity of the reservoir at an elevation between EL. 832.0 m and El. 880.0 m.

Table 3 Distribution of Sediment Deposit in the Makio Dam Reservoir by Elevation (Unit in 1,000 m^3)

Elevation	<832 m	832 ~860 m	860 ~870 m	870 ~880 m	Total
Amount of Sediment	3,820	5,710	830	440	10,800

The geological profile of sediment deposit in the reservoir was as shown in Fig. 6. At an elevation higher than the zone between El. 850 m to El. 860 m (upstream reach of the river), sediment material was mainly gravel and sand. At a lower elevation (downstream reach of the river), sediment material was mainly clayey soil.

2) Amount of Sediment Concerned to the Sedimentation Countermeasure Project

It was necessary to estimate the amount of sediment to be handle by the sedimentation countermeasure project to take into consideration the already deposited sediment in the reservoir that was hindering the reservoir's storage function and also the amount of sediment that would flow into the reservoir in the future in order to maintain the service water capacity of the reservoir in the future.

The amount of sediment deposited in the reservoir

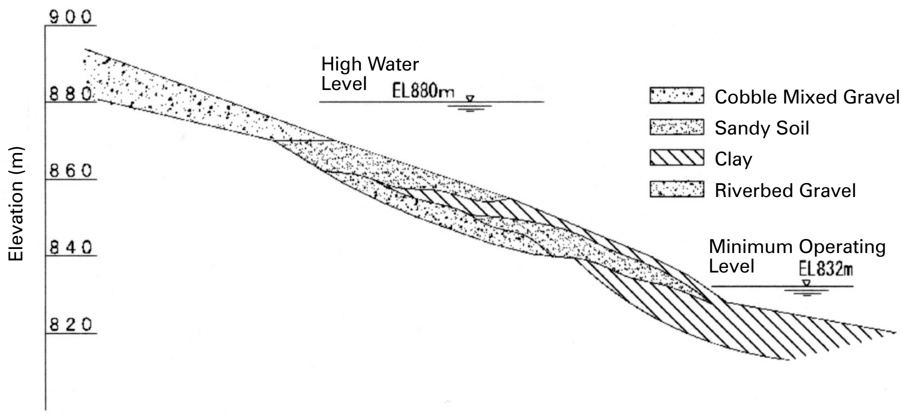


Fig. 6 Geological Profile of Sediment Deposit in the Makino Dam Reservoir

as of 1992 exceeded the reservoir's design sediment storage capacity by 3,800,000 m^3 (10,800,000 – 7,000,000). By assuming that the amount of sediment that accumulates in the reservoir in the future would be equivalent to the amount that accumulates during the remaining service life of the Makio Dam, i.e. 69 years (based on the entire service life of 100 years from 1961 to 2061), this amount was estimated as 4,830,000 m^3 based on the annual sediment accumulation rate of 70,000 m^3 (70,000 m^3/year x 69years).

Due to the above reasons, the amount of sediment to be handled by the sedimentation countermeasure project was decided upon as 8,630,000 m^3 that was the total of already deposited sediment amount and the amount that accumulates in the future.

3) Selection of the Method of the Countermeasure

The following five methods were investigated:

Plan 1: To increase the sediment storage capacity of the reservoir to store the concerned amount of sediment for the project (8,630,000 m^3) by raising the dam height.

Plan 2: To secure the sufficient sediment storage capacity to store the concerned amount of sediment for the project (8,630,000 m^3) by constructing a new dam.

Plan 3: To install a scour pipe to discharge deposited sediment out of the reservoir into the downstream reach.

Plan 4: To remove the entire amount of the concerned sediment for the project (8,630,000 m^3) from the reservoir.

Plan 5: To remove the amount of sediment necessary to secure the effective storage (5,480,000 m^3) and construct a sediment storage dam.

The above five methods were examined by comparing with each other and, as a result, Plan 5 was selected as a favorable method from the viewpoint of its reality to implement, environmental concern, and economy for the following reasons:

① Sediment material deposited at a higher elevation in the reservoir consists mainly of gravel and sand. It is possible to excavate and remove it when it dries.

② The Makio Dam Reservoir is mainly operated for hydroelectric power generation during a period from December 1 through March 31 and the reservoir operation is scheduled to make the reservoir empty on March 31. Thus, the high elevation area in the reservoir becomes dry during this period.

③ By excavating the sediment deposited in the reservoir by dry work and temporarily placing the excavated sediment into the sediment storage dam as much as possible during this period of the year, most of the concerned amount of sediment for the project can be removed under dry work conditions.

By constructing a new sediment storage dam, the effective storage of the reservoir may be secured even after the original service life of the dam.

4) Outline of the Sedimentation Countermeasure Project

Based on the above-mentioned policies, the countermeasure project was established to remove sediment already deposited in the reservoir and the sediment that inflows in the future by constructing a sediment storage dam in the upstream part of the reservoir in order to restore and maintain the storage function of the reservoir and thereby prevent a natural disaster. The established project is outlined as follows:

Major Work

(1) Sediment Removal: 5,480,000 m^3 (the reservoir's effective storage can be secured when the sediment material removal work is completed)

Amount of sediment that exceeds the design sediment storage capacity:

10,800,000 m^3 – 7,000,000 m^3 = 3,800,000 m^3

Expected amount of sediment that flows into the reservoir prior to project completion date:

70,000 m^3/year x 14 years = 980,000 m^3

Capacity of the sediment storage dam: 700,000 m^3

Total: 5,480,000 m^3

Note: As the sediment storage dam is to be constructed within the reservoir in order to remove the amount of sediment in the reservoir equivalent to the sediment storage capacity of the storage dam and to secure the effective capacity of the reservoir, the capacity of the storage dam was decided upon to be sufficient to store the sediment that will flow into the reservoir for a 10 year period.

(2) Construction of One Sediment Storage Dam
The purpose of the Sediment Storage Dam is to entrap sediment inflow into the reservoir after the removal of deposited sediment and maintain the service water capacity of the reservoir.

(3) Consolidation Work: One place
This work is to prevent the inflow of sediment material accumulated on the upstream part of the reservoir when the sediment deposited in the reservoir is removed and when the reservoir bottom is lowered.

(4) Project Implementation Schedule
12 years starting from Fiscal Year 1995 and ending in Fiscal Year 2006 (12 years)

5. Detail of the Sedimentation Countermeasure Project

1) Use of Reservoir Water and Sediment Removal

The relationship between the operation of the Makio Dam Reservoir and the amount of sediment removal is shown in Fig. 7. The operation water level shown in the figure means the reservoir level that is targeted for reservoir water use. The sedimentation countermeasure project was set up to excavate and remove sediment deposited at an elevation higher than the operation water level so that sediment removal work will not disturb the operation of the reservoir. In detail, it was set up to remove 5,480,000 m^3 of mainly gravel and sand material deposited at an elevation higher than El. 855 m under the dry work condition

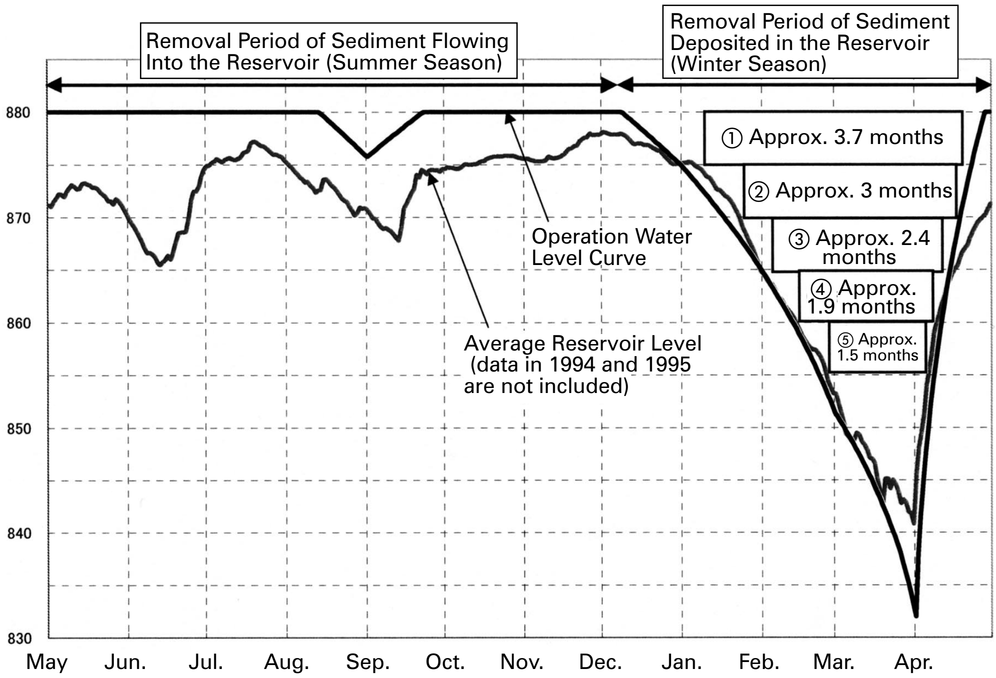


Fig. 7 Relationship between Operation Water Level and Sediment Removal

Table 4 Sediment Removal Schedule by Elevation

Elevation	Amount to be Removed (1,000 m³)	Number of Work Block	Work Period	Work Year Phase	Total Work Period
El.880 to El.875	511	5	3.7 months (112days)	1st to 2nd	7.4 months
El.875 to El.870	822	8	3.0 months (91days)	2nd to 5th	12.0 months
El.870 to El.865	992	10	2.4 months (73 days)	2nd to 8th	16.8 months
El.865 to El.860	1,168	12	1.9 months (57 days)	2nd to 8th	13.3 months
El.860 to El.855	1,155	12	1.5 months (44 days)	2nd to 8th	10.5 months
Total	4,650	47	– –	–	

when the trafficability of heavy excavation equipment is secured during the above-mentioned several months of dry condition.

It was planned to implement the project during a winter period from the early part of January when the reservoir level is lowered through the middle of April when the reservoir level starts to rise. In addition, it was also planned to conduct sediment material excavation and removal work whenever the reservoir level is lowered even during the summer season.

① During Winter Period: Reservoir level decreases and a planed number of workable days can be set up. Sediment excavation work can be conducted by using large excavation equipment.

② During Summer Period: Sediment flows into the reservoir every year during the summer period. It is planned to remove sediment deposited at an elevation higher than El. 870m using ordinary earthwork equipment whenever it is possible.

2) Amount of Sediment to be removed at Each Different Elevation

Out of the total amount of 5,480,000 m^3 sediment to be removed by the project, the amount that should be removed at the upstream side of the reservoir during the winter season was decided upon as 4,650,000 m^3 judging from the number of workable days during this season as shown in Table 4. This amount of sediment is to be removed by setting up an amount at each different elevation.

As for the remaining 830,000 m^3 of sediment, of which 630,000 m^3 that would flow into and deposit in the reservoir during the project implementation period was planned to be removed during the summer season. As for another 200,000 m^3, it was planned to remove from the area at an elevation higher than EL. 855 m in the left side of the reservoir immediately upstream of the dam.

3) Securing of Disposal Sites

It was planned to move excavated sediment material in the reservoir to disposal areas. The Makio Dam site is surrounded by a mountain range. As it was difficult to find suitable disposal sites near the dam site to stockpile a total amount of 5,480,000 m^3 of sediment material, several disposal sites were secured. When selecting these disposal sites, the residents in the Ootaki and Mitake villages expressed a favorable understanding and offered a great deal of cooperation.

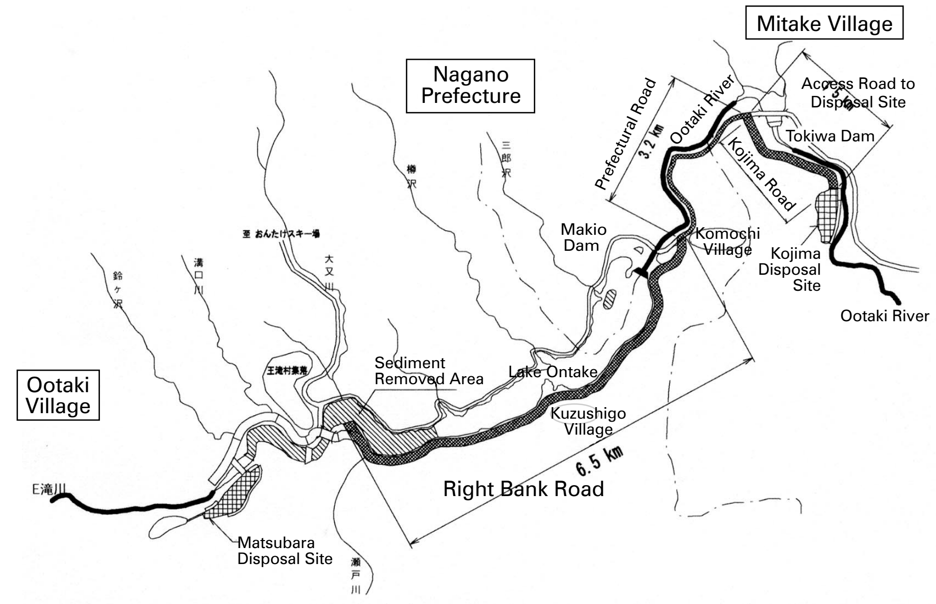


Fig. 8 Transportation Route to the Kojima Disposal Site

Table 5 Sediment Removal Plan in Each Fiscal Year

(Unit in 1,000 m^3)

Fiscal Year	1998	1999	2000	2001	2002	2003	2004	2005	2006	Total
Removal Amount of Deposited Sediment	30	640	650	830	884	775	628	413	—	4,850
Removal Amount of Flowing In Sediment	70	70	70	70	70	70	70	70	70	630
Total	100	710	720	900	954	845	698	483	70	5,480

Two major disposal sites among the selected ones are outlined below:

(1) Matsubara Disposal Site

The Matsubara Disposal Site is located on the right side of the upstream part of the reservoir and has an area of approximately 21.5 hectares. It is possible to store an approximately 3,000,000 m^3 of removed sediment material. As it is possible to remove sediment material directly from the reservoir to the disposal site without using road transportation, the sediment removal operation does not interrupt road traffic. It is possible to efficiently remove sediment material by using large-sized excavation equipment and dump trucks. This is an economical disposal site. (Fig. 8)

(2) Kojima Disposal Site

The Kojima Disposal Site is located on the right bank of the Ootaki River downstream of the dam and has an approximately 126 hectares of area. It can store an approximately 1,500,000 m^3 of removed sediment material. The site is approximately 12 km towards the downstream direction from the sediment excavation site at the upstream part of the reservoir. As excavated sediment material has to be transported using an existing village and prefectural roads, it was planned to transport excavated sediment material by 10-ton dump trucks. Thus, it was necessary to take measures for traffic congestion, vibration, and noise problems.

4) Transporting of Sediment Material to Disposal Site

(1) Transporting of Sediment Material to Matsubara Disposal Site

A temporary construction road was to be built within the reservoir and it was planned to use 42-ton large sized dump trucks to transport sediment material to the disposal site.

(2) Transportation of Sediment Material to Kojima Disposal Site

It was planned to transport sediment material to the disposal site using 10-ton dump trucks over an ordinary public road. However, as the amount of sediment to be removed in a day during the winter season is 4,600 m^3, if 10-ton trucks were to be used for the transportation, 920 dump trucks would be necessary. Further, it was impossible for 10-ton trucks to pass each other on the 5m wide village road located on the right bank of the river. In addition, as the prefectural road section planned for sediment material transportation is the only access road to the center of Ootaki Village and the Ontake Ski Resort, if traffic congestion occurs on the road, the effects of vibration and noise to area residents were feared.

5) Measures for Transportation to Kojima Disposal Site

(1) Securing of Both Direction Traffic on the Village Road on the Right Bank

The existing village road on the right bank of the Ootaki River was only 5 m wide and it was impossible for 10-ton dump trucks to pass by each other. The entire 6.5km section for dump truck use was widened to an 7m width.

(2) Measures for Traffic Congestion, Vibration and Noise Problems

Traffic congestion forecast by traffic simulation and traffic vibration and nose prediction by a vehicle test driving were conducted and the allowable daily traffic volume of the village road was decided upon as 800 vehicles a day (between 8:30 to 16:30).

As for the amount of sediment material that cannot be transported to the Kojima disposal site due to the limitation of the allowable daily traffic volume, it was decided to provide a temporary storage site, store the sediment material temporarily in the site and transport it to the Kojima disposal site when road traffic becomes lighter during the summer season.

In order to prevent traffic congestion to be caused by the slower speed of dump trucks due to the effects of snowfall and freezing, it was planned to take sufficient snow removal and freeze-prevention measures.

6) Sediment Material Removal Yearly Plan

The sediment material removal plan that was prepared based on the above-mentioned concept in each fiscal year is as listed in the following table: (Table 5)

6. Implementation Condition of the Sedimentation Countermeasure Project

Based on the above-mentioned policies, the excavation and removal of sediment material deposited in the reservoir began in Fiscal Year 1999. The sediment material removal work was 83% complete as of the end of March 2005. The removal work of the remaining sediment material is presently being conducted and the construction of a sediment storage dam is simultaneously carried out. The project is smoothly progressing on schedule to be completed by March 2007. It is expected that the storage function of the Makio Dam Reservoir be sufficiently restored.

7. Restoration of the Catchment Area by the Progress of Erosion Control Work

The point that should not be overlooked in the sedimentation countermeasure project is the restoration measure of the devastated catchment area of the Makio Dam. Since immediately after the Nagano Seibu Earthquake, the Nagano Prefectural Government and Forestry Agency have been carrying out the construction of debris control dams and channel work and tree planting on severely scoured slopes and debris deposited bare land along stream banks created by debris flow. The residents in the water source areas of the reservoir, such as Ootaki Village, and the water-use

Photo 5 Condition of the Catchment Area immediately after the Earthquake

beneficiaries of the Makio Dam conduct tree planting activities every year. Personnel exchange between the people in the water source areas and the people in the downstream region is eagerly taken place through the tree planting activities. In particular, since 2001 a large-scale tree planing activity called "Green Button Connecting to the Future Centuries" has been conducted with the participation of Ootaki Village as its center, Mitake Village, Chunichi Newspaper Company, Chubu Forest Management Bureau, Japan Water Agency, and the beneficiaries. This activity held on the slope of Mt. Ontake aims at handing over the abundance of water and greenery to the future generations by the cooperation of the people in the water source areas and the people in the downstream regions through the restoration activities of forests in the water-source areas of the Makio Dam that was devastated by the previous earthquake.

For the sake of debris control work and tree planting activities being carried out in the catchment area of the Makio Dam since immediately after the earthquake, the severely damaged catchment area of the Makio Dam has recovered to such a level that the amount of sediment flowing into the reservoir is almost equivalent to that prior to the earthquake.

As for the new sediment storage dam that is being constructed by part of the sedimentation countermeasure project, sediment material to be entrapped by the dam should be periodically removed after dam completion. This project being implemented is an extremely important measure to exhibit the effects of sedimentation countermeasure to control the inflow of sediment into the reservoir.

The sedimentation countermeasure project for the Makio Dam aims at the restoration of the reservoir functions. As this project suggests the importance of

Photo 6 Present Condition of the Catchment Area

Photo 7 Tree Planting Activities in the Makio Dam's Catchment Area

measures to be taken for the dam's catchment area, this paper introduced these measures.

8. Conclusion

The Japanese economy has entered into a stabilized period. It is forecasted that the population of Japan will reach to the peak in 2010 then begin to decline thereafter.

Demand for water is assumed to maintain almost the same level. Water resources development in Japan has achieved a certain result. From now on, it is the time to appropriately maintain and efficiently use existing facilities. For this reason, it is inevitable to proceed with sedimentation countermeasures.

This paper introduces the restoration method of reservoir functions without interrupting water-use operation that is being conducted at the Makio Dam whose reservoir functions were severely damaged by sediment inflow and themes and their measures related to sediment material transportation. In addition, the paper also introduces the debris control work, tree planting activities and the dam's catchment area restoration project being conducted by related agencies. In order for the dam to fully exhibit the reservoir functions, it is considered that, in addition to tree planting activities, continuous maintenance and preservation of the catchment area by conducting periodical thinning and weeding work under the cooperation with area residents and water-use beneficiaries and the promotion of effective use of wood produced by thinning would be very important.

It is the authors' great pleasure if this paper will provide some reference to those who are encountering similar reservoir sediment problems.

At last the authors would like to express their sincere appreciation to those in the water-source areas, water-use beneficiaries, and related organizations and agencies that provided extensive cooperation for the implementation of the project.

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1. Takeuchi and Honda, Overall Implementation Plan of Sediment Material Removal Works for Makio Dam, Japan Water Agency, 1999.
2. Ootsu and Hashiba, Traffic Forecast and Transportation Plan for Sediment Material Transportation using a Public Road, Japan Water Agency, 2004.

Development and Applications of Multi Hole Suction Pipe

Hideyoshi Nakashima
Association of Water Resources Sedimentation Technology, Japan

Hiroyuki Katayama
Association of Water Resources Sedimentation Technology, Japan

Yoshimitsu Tajima
Association of Water Resources Sedimentation Technology, Japan

1. Introduction

There are several countermeasures against sedimentation in a dam reservoir (Table-1). A sedimentation dam traps sediments that flow into a dam and a sediment bypass tunnel routes turbid water as well as sediment downstream of a dam. The sediments, which have been already deposited in a dam reservoir, may be dredged, released from the bottom gate of the dam (flushing), or removed by some sediment-removing facilities such as Hydro suction Sediment Removal Systems (HSRS), which utilize the water head difference to initiate sediment suction flow.

HSRS may be classified into two types depending on whether the sediment-suction devices are movable or fixed in their location. Although none of these systems has been applied for the actual sediment removal project in Japan, several systems have been proposed and their applicability has been investigated. Examples of fixed-type HSRS are: Hydro-P which applies continuous suction slits; Multi Hole Suction Pipe (MHSP) which applies multiple suction holes separately located along the main suction pipe; and the system that utilizes a sheet covering the reservoir bed around the suction pipe aiming to extend the sediment-suction area. Among these systems, this paper focuses on the development and applications of MHSP proposed by authors.

2. Development History of MHSP

The goal of our research group, Water Resources Sedimentation Technology (WRST), is to develop and improve technologies that reduce or remove sediments accumulated in a dam reservoir. For example, WRST has been engaged not only in the development of conventional dredging but also in slurry transportation technologies such as U-shaped conveyer and pneumatic transport system. In 2001, besides these technologies, we started the investigations of HSRS originally aiming for the removal of deposited sediment in Miwa Dam without causing serious environmental damages at downstream of the dam. Miwa Dam is located along Mibu River in the watershed of Tenryu River. HSRS is one of the preferable sediment-removal systems at Miwa Dam because the system utilizes the

Table-1 Countermeasures against sedimentation in a dam reservoir

	Movable suction	Fixed suction	The conventional machine dredge (for reference comparison)
Sand Collection	Floating body + Siphon dredge Hydro－J method SY method etc	Transport sediment by lowering the reservoir water level	pump dredger + mechanical pump
Suction		Orifice type discharge pipe discharge pipe with dispersed type slit Hydro-P method MHSP method Flexible slit discharge pipe with sheet etc	
Derivery	Pipe or Ship transport	Pipe	Pipe or Hopper barge
Storage	Storage outside at a reservoir or Temporal storage in a reservoir	----	Sedimentation pond
Down-stream discharge	---- (Discharge during flood)	Concentration control Discharge during flood	---- (Recycle or disposal)

Table-2 Development history of MHSP

Year	Experiment	Investigated features
2001	Lab. experiment	physical mechanisms around suction holes
2002	Lab. experiment	applicability of multiple separate suction holes
2003	Lab. experiment	MHSP / extension of suction area (shut & open device by sliding-cover, water jets around holes) / development of numerical model / design of system in the actual field
2004	Field study at Miwa Dam	Field-scale experiment wit sediment collected from Miwa Dam. MHSP with 300mm-diameter.

natural water head difference, i.e. clean energy, to initiate the suction flow.

In 2001, WRST carried out laboratory experiments to investigate the physical characteristics of the sediment-suction holes and proposed a new HSRS that has separate multiple suction holes located along the bottom of a straight pipe. In 2002, WRST carried out additional laboratory experiments to capture the physical mechanisms of the proposed HSRS and to investigate measures to enlarge the sediment trap area and also to prevent blockage of the system. In 2003, we modified this system by opening the upper-end of the suction pipe and placing it in the water above the sand bed so that the system can intake pure water from this upper-end opening. This modification worked to prevent blockage of the system as the supplied pure water decreases the slurry density and instead increases the flow velocity. We called this system as Multi Hole Suction Pipe (MHSP) and, through a number of laboratory experiments, further investigated the theoretical mechanisms of the system and explored possible improvements of the system. For example, we confirmed the effectiveness of "shut and open device," which shuts and opens the suction hole by a sliding-cover attached at the hole. By shutting the downstream suction holes, this shut and open device extends the "effective length" of MHSP, within which the system effectively intake and transport the slurry without causing blockage problems. Water-jet installed around the suction holes were also effective to fluidize the sediment accumulated around the suction hole and hence to enhance sediment suctions. Based on these experimental data, furthermore, we developed one-dimensional numerical model to predict the inflow rates at each suction hole as well as flow velocity distributions along the suction pipe.

In 2004, WRST performed a field experiment of MHSP at Miwa Dam using the sediments deposited in the dam and confirmed the overall performance of MHSP in the actual filed-scale. Through the field experiment, we also verified the applicability of the numerical model, which was newly developed for predictions of hydrodynamics of MHSP in the field scale. The numerical model is now utilized to determine the optimum design of MHSP based on given conditions such as available water head difference, characteristics of deposited sediment, and required discharge rate of the sediment, etc.

3. Advantages of MHSP

Primary advantages of MHSP are outlined as follows:

(1) The system efficiently utilizes the natural clean energy of water head difference.
(2) Since the entire system is based on a simple pipe-flow structure, cost of the system is relatively low.
(3) No sediment disposal area is necessary since the system discharges sediments directly to the downstream.
(4) Upper-end opening of the discharge pipe enables the system to retain high flow velocity and to prevent blockage of the system.
(5) Water-jet at the suction holes fluidizes and breaks up the ambient sediment and enhances the efficiency of the sediment suction.
(6) "Shut and open device" extends the effective length of MHSP

Figure-1 illustrates how MHSP may be applied in a dam reservoir and Figure-2 shows the detailed structure around the suction holes.

4. Laboratory Experiment

In 2003 and 2004, we performed laboratory experiment and investigated the physical mechanisms of MHSP. Outlines and primary outcomes of the experiment are summarized in the following sections.

4.1 Outline of the experiment

The experiment uses a basin with 0.7m deep, 1.0m wide, and 8.5m long. MHSP was 52mm in diameter and made of transparent vinyl chloride so that one can observe the features of sediment suction. This MHSP is 1/6-scale of the MHSP used for the field experiment discussed later. Round suction holes were separately opened along the bottom of the pipe with diameter of 23mm, i.e. 1/5 of the pipe diameter. Distance between each adjacent hole was set to be 350mm so that the ratio of open and closed area becomes 1:15 as proposed by Michiue and Oda (1986). The down-end discharge opening of the pipe was set in the air and a valve, pressure gauge, γ-ray density meter, and flow meter are installed near the discharge opening. Water head difference was changed by adjusting the elevation of the discharge opening. Overflow gate was installed in the basin so that the water level of the basin is stabilized. The basin was filled with either quartz sand or silt. Characteristics of the quartz sand

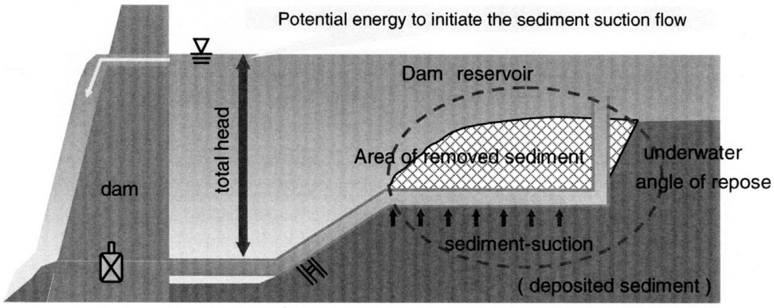


Figure-1 Application image of MHSP in a dam reservoir

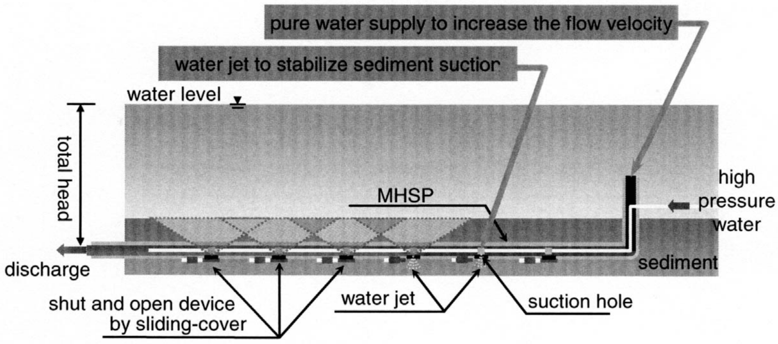


Figure-2 Illustration of MHSP

and silt are summarized in Table-3 and Figure-4. In the case of silt, deposited sediments were not fully consolidated. In addition to the measurement facilities near the discharge opening mentioned above, five pressure gauges were installed between each suction holes and another flow meter was installed near the upper-end opening of the pipe. Ultra-sonic distance measurements device was also installed to measure the spatial bed elevations. Figure-3 shows the cross view of the experimental facilities.

4.2 Observed features of MHSP for quartz sand

Through the experiments, applicability of MHSP for quartz sand was verified. Overall characteristics of MHSP for sand-discharge are summarized as follows:

- The system successfully discharged quartz sand when the depth of the sand above MHSP is less than five times of pipe diameter. Required discharging time depended on the sand depth above MHSP.
- As sand is discharged, following characteristics of geometry change of the sand bed was observed (Picture-1 and Figure-5). Seepage failure first occurs around the suction holes. The area of the seepage failure is extended to the sand surface and starts to form a vertical water path along the water flow (piping). Area of this vertical water path is horizontally enlarged until the slope of the sand bed reaches to the angle of repose. The earlier the piping occurs, the more efficient is the sand discharge.
- Sediment concentration and flow velocity of the discharged slurry was measured. Average volumetric concentration of the sand was relatively low and was ranged from 2 to 5 percent.

Bed geometry after discharge of quartz sand was downward-cone-shape with slope of 35-degree. This knowledge of sand-bed geometry can be used to estimate the total volume of sand to be discharged by multiple suction holes.

Table-3 Sediment characteristics

quartz sand	
d_{50}	0.07 (mm)
density ρ_s	2.642 (g/cm^3)
wet unit weight ρ_s	1.936 (g/cm^3)
dry unit weight ρ_d	1.505 (g/cm^3)
permeability κ	1.12×10^{-3} (cm/s)

Miwa sediment	
d_{50}	0.01 (mm)
density ρ_s	2.76 (g/cm^3)
liquid limit W_L	40.7 (%)
plastic limit W_P	32.3 (%)
plasticity index I_P	8.4

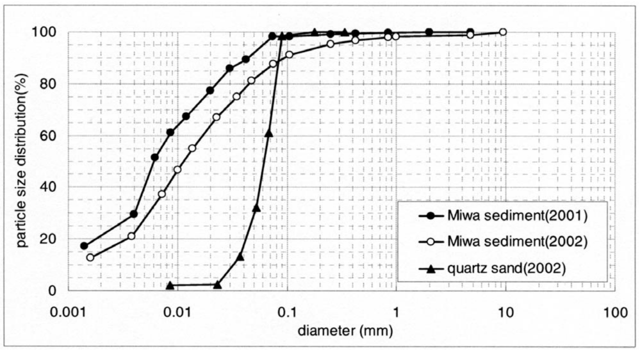


Figure-4 Diameter distributions of sediments used for the experiment

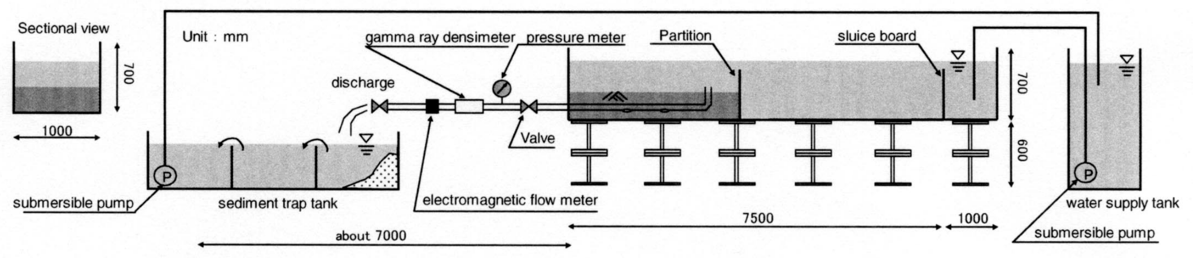
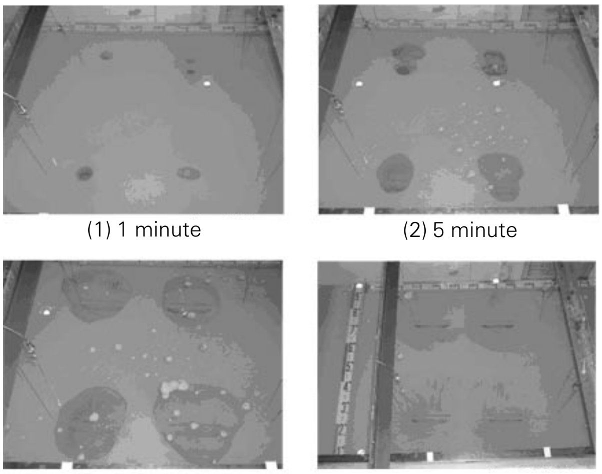


Figure-3 Cross-view of experimental facilities



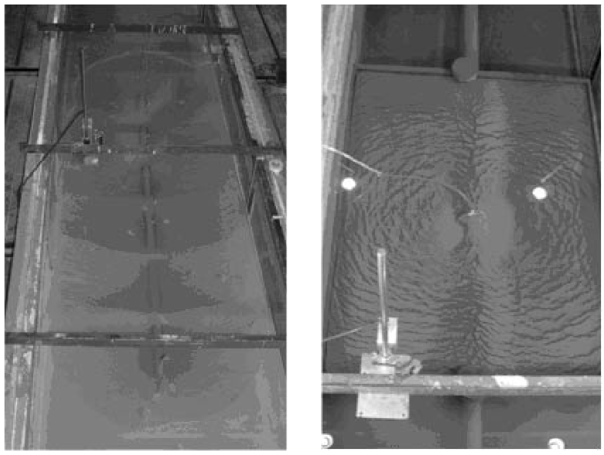
(1) 1 minute

(2) 5 minute

(3) 10 minute

(4) 15 minutes
(equilibrium state)

Picture-1 Geometry change of the sand bed after initiation of suction flow



quarts sand

silt collected from Miwa Dam

Picture-2 Bed geometry after discharge

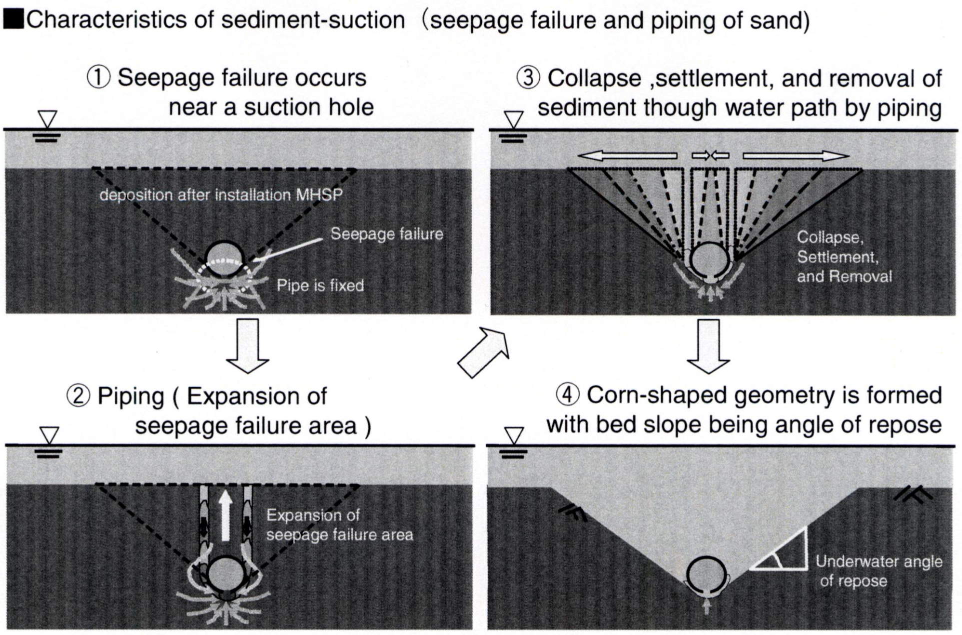


Figure-5 Characteristics of geometry change of the sand bed

- Effective length of the suction pipe was about 1.4m, which corresponds to four suction holes in this experiment.
- Following supplemental facilities were effective to enhance the sediment discharge.
  1) Upper-end opening of the pipe that supplies pure water and increases the flow velocity and decreases the slurry density.
  2) Water jet around the suction hole that fluidizes ambient sediment.
  3) Shut and open device by sliding-cover extended the effective length of the suction pipe. In this

experiment, the effective length was extended up to eleven suction holes, i.e. was nearly tripled by applying this shut and open device.

- One-dimensional numerical model was developed for predictions of hydrodynamics of MHSP. The developed numerical model provides predictions of pressure and flow velocity inside the pipe from the input data of water head difference as well as dimensional features of MHSP.

4.3 Observed features of MHSP for silt

The system also discharged the silt collected from Miwa Dam if the consolidation time was short and deposited silt was retained to have relatively high fluidity. As the fluidity of the deposited silt goes down (moisture content of 50% in this experiment), however, MHSP sometimes failed to intake the silt around the suction holes. Compared to the experimental case for quartz sand, average concentration of slurry was higher, ranged from 5 percent to 15 percent. Bed geometry remained plane and the entire bed level was lowered as MHSP discharges the silt.

5. Field Experiments

Through the laboratory experiment, we confirmed the applicability of MHSP especially for the discharge of sand in a laboratory scale. Applicability of MHSP for discharge of silt in an actual scale, however, still remains unclear. In order to investigate this feature, we carried out a field experiment at the east bank of Miwa Dam (Figure-6) from December 2004 until February 2005. Miwa Dam is located in the river system of Tenryu River, Nagano. The experiment used the actual silt deposited in Miwa Dam whose fluidity was relatively high.

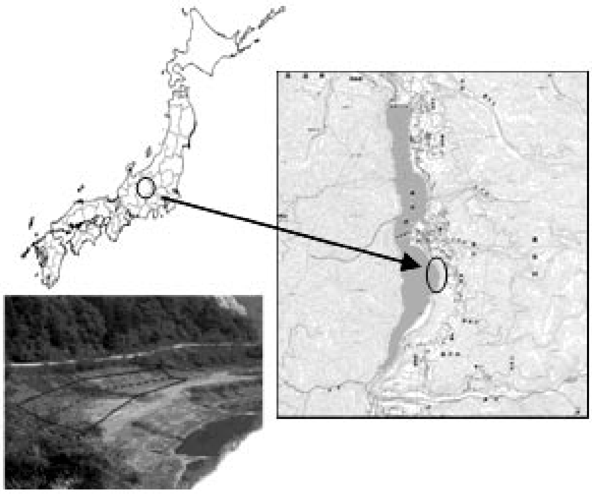


Figure-6 Field study site

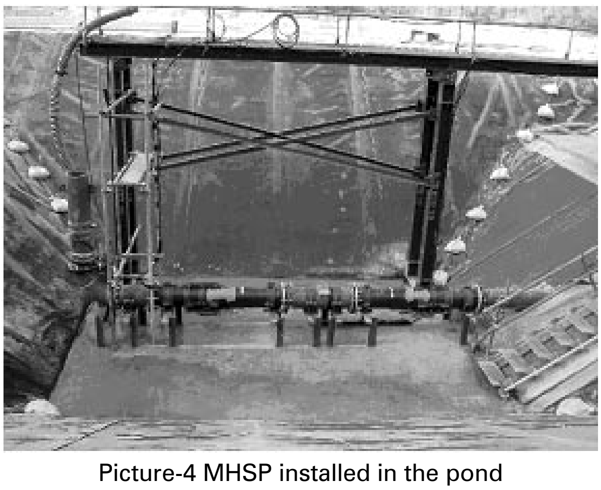
5.1 Outline of the experiment

We first developed a 3.75m-deep basin with horizontal dimensions of 4mx6m near the bottom and 9mx11m near the surface and installed MHSP (Pictures 3 and 4) near the bottom of the basin. Diameters of the main pipe and suction holes were set 300mm and 134mm, respectively. In this basin, we spread the silt collected from the dam reservoir until the surface elevation of the silt reaches to 1m above the bottom of MHSP. Moisture contents of the silt was measured and controlled before the silt was spread in the basin. The basin was then filled with water and the surface water elevation was stabilized by overflow gate. The length of the discharge pipe from the down end suction hole to the discharge opening was 40m and the diameter of this discharge pipe was D=300mm. Elevation of the discharge opening of the pipe was adjusted so that the total water head difference becomes 1, 2, 3, or 4m depending on the experimental cases.

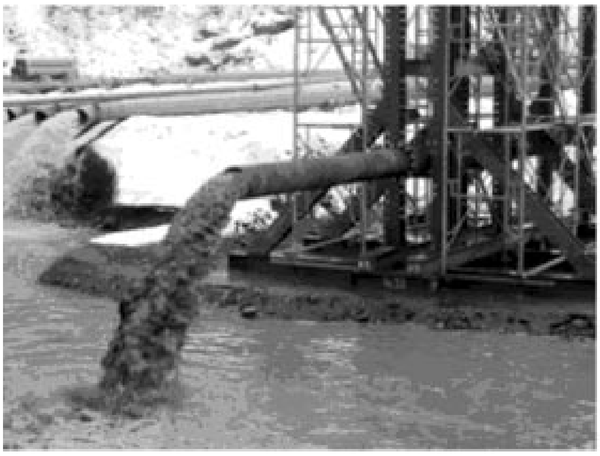
In all experimental cases, we first shut both suction holes and let the pure water flow in the pipe from the upper end opening. After the flow rate was stabilized, we opened one or two suction holes depending on the experimental cases and measured the time-varying velocity, pressure, and the density of the slurry flow. Figure-7 shows the positions of five pressure gages, a γ-ray density meter, and three acoustic Doppler current profilers. Procedures of the experiment are summarized as follows.

1) Collect Miwa silt and control the fluidity of the silt by adjusting the moisture contents of the collected silt.
2) Measure and record the moisture contents of the silt and spread it in the basin.
3) Fill in the water until the surface water reaches to

Picture-3 Overview of field experimental devices



Picture-4 MHSP installed in the pond



Picture-5 Discharge pipe during operation

Picture-6 Geometry change of silt bed after suction

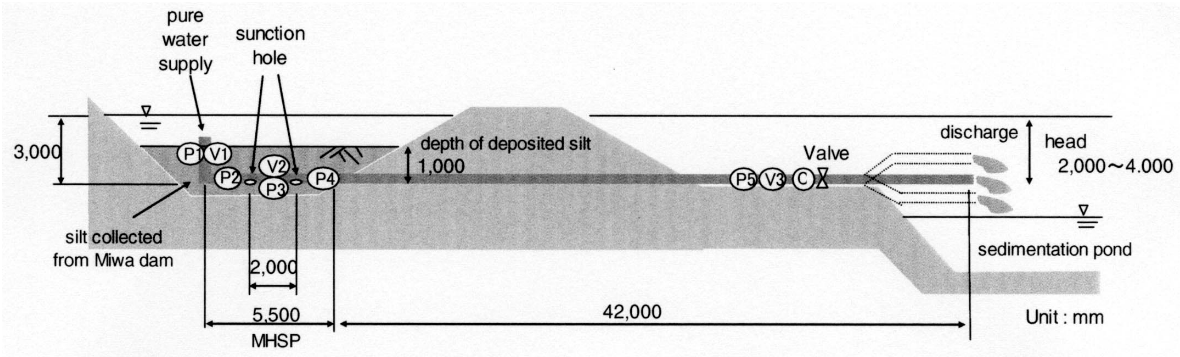


Figure-7 The outline of a field experiment institution

the designed elevation.

4) Open the downstream valve with both suction holes closed.
5) After the water flow was stabilized, open one or two suction holes to begin sediment suction.
6) Measure velocity, pressure, and density of the discharged slurry.

5.2 Observed features of MHSP in the field scale

Through the field experiments, we confirmed that MHSP in the actual field scale can effectively discharge the silt with relatively high fluidity. We should however note that the depth of deposited silt above the bottom of MHSP was kept in 1m in this experiment and further experiments may be necessary to examine the applicability of MHSP when the depth of deposited sand is deeper. Other primary features of MHSP observed in this experiment are as follows:

1) Average density of the discharged slurry was $1.2g/m^3$, which corresponds to 11% of volumetric sediment concentration and $2x10^5$ppm of SS, suspended solid, concentration.
2) Slope of the silt surface after the discharge was around 1:10. This slope may depend on fluidity of the deposited silt.
3) MHSP discharged silt without blockage even when debris such as pieces of wood and plastic bags was mixed in the silt.
4) When the density of the discharged slurry is stable with its peak value (Phase 1 in Figure-8), the

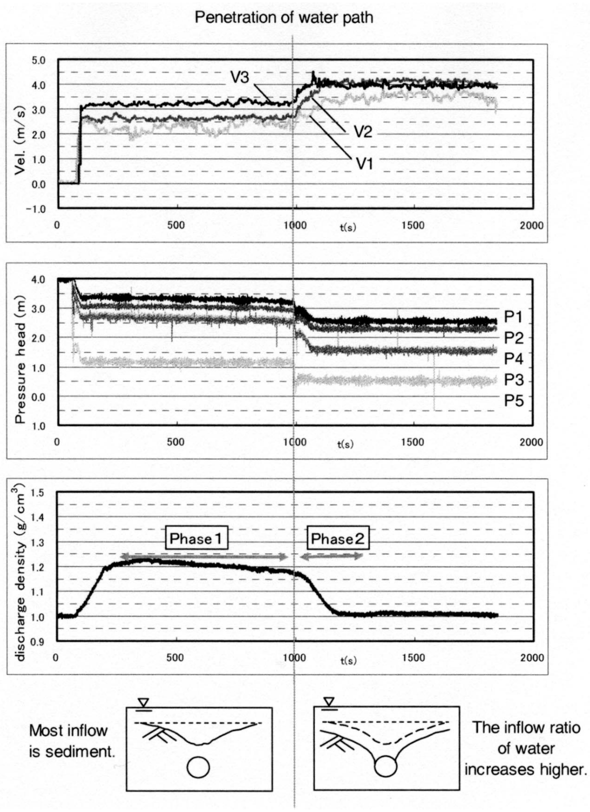


Figure-8 Time varying flow velocity, pressure and slurry density (Head:4m, Deposition thickness:1m, Water ratio:34.6%, 1 suction hole)

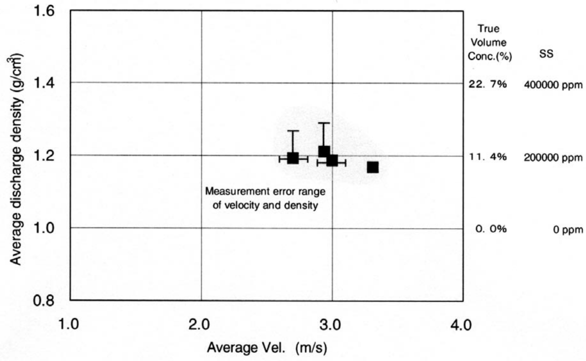
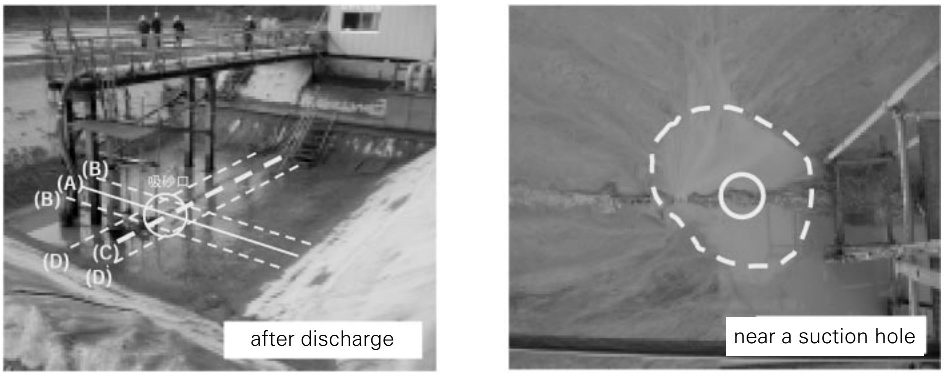
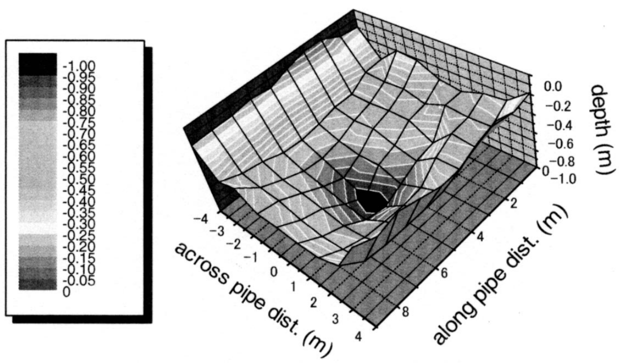


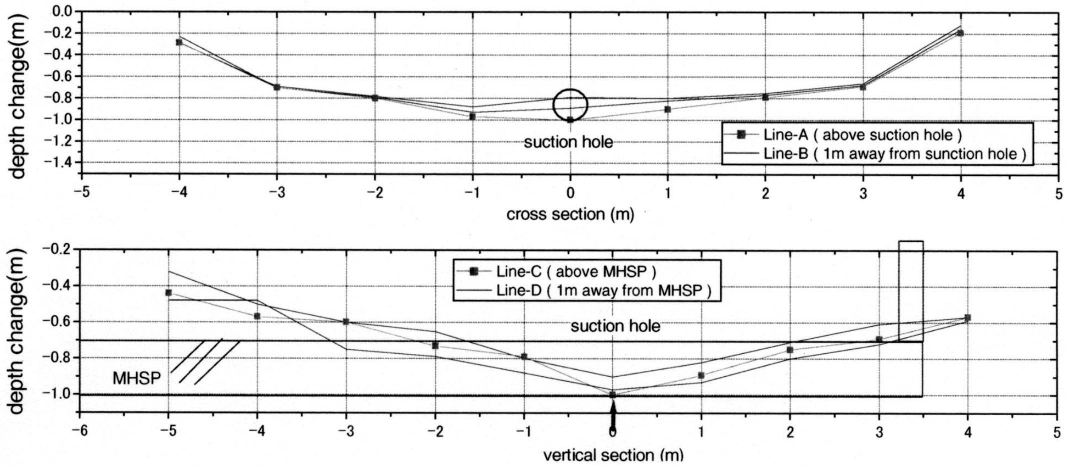
Figure-9 Relationship between average density and average discharge flow velocity



(1) Picture of silt bed after discharge



(2) Bird's-eye view of silt bed after discharge



(3) Slope of silt bed after discharge (cross and vertical section)

Figure-10 Geometry change after discharge (Picture and Survey result) amount of discharged silt, determined as a product of measured flow rate and the measured silt concentration near the discharge opening, was nearly equal to the product of suction flow rate and the concentration of silt deposited around MHSP. Here suction flow rate was determined as a difference of measured flow rates upper and down side of the suction hole. The concentration of deposited silt was simply determined as 1-n with n=0.6, the porosity of the deposited silt. This observed fact indicates that, in this experiment, the ambient silt with high fluidity directly flew into the suction hole without seepage effects.

5) The numerical model, originally developed based on the laboratory experiment, was also applicable to the real scale conditions.

Based on observations listed in 4) and 5), we can estimate the peak concentrations of discharged slurry as well as time-varying flow rates and pressure distributions in MHSP.

6. Practical Application of MHSP to Dam Reservoirs

Applicability of MHSP in the real scale was confirmed. As discussed above, the newly developed numerical model provides us flow rate of slurry as well as its peak density once following conditions were given: water head difference; sediment characteristics; dimensions of MHSP such as diameter and length. All these information is now readily incorporated to the fundamental designs of the entire MHSP system according to target discharge rate of slurry and local conditions of the dam reservoir. Hinokidani(2005) referred to our work in his paper and pointed out the validity and applicability of the numerical model as a tool to design the practical MHSP system.

Especially for the reservoir which has deposited sand with relatively uniform grain diameters, MHSP may be one of suitable systems to remove this deposited sand. Water jet may be effective to fluidize the deposited sand and to promote piping. Occasional pump may also be helpful to reduce the risk of blockage.

Kashiwai (2005) also admit the future applicability of HSRS, such as MHSP, but pointed out following two limitations of the system. The first limit is that the ability of existing HSRS may be limited up to the sediment discharge of about 105m^3/year. The other limit is that HSRS can be used only during the flood and therefore requires certain supplemental system that can be used for everyday basis and collect deposited sand in the area within which HSRS can entrain the sand.

Presenting MHSP may also be more efficient being combined with other existing sand removal systems. Especially to collect the sand (transport deposited sand within reservoir), for example, combination with following systems may be effective:

1) Dredging
2) Sand transportation by pumping system
3) Use of HSRS combined with sedimentation pond whose water elevation is lower than targeted sand.

In item 2), dredging or MHSP may be applied to collect the transported sand. In order to apply MHSP, however, one may require a pump to yield the pressure difference. Among these options, one advantage of Item 1) may be in that debris can be removed when disposing the dredged sand to the target site and the risk of blockage may be reduced. When designing the entire sand-removal system, one needs to account for various conditions, such as characteristics of deposited sand, targeted volume of removed sand, and available water head difference, etc.

7. Future Works

All the experimental and numerical studies performed by authors have clarified the overall performance and applicability of MHSP. All these knowledge are now incorporated to carry on the fundamental design of the system. In order to apply MHSP in the actual sediment removal project, however, there still remains following problems:

1) Control of the sediment concentration of discharged slurry.
2) Removal of debris to avoid blockage of the system.
3) Establishment of solid methodologies for design, construction, maintenance, and operations of the entire system.

All these problems are essential to apply MHSP to the practical sediment removal project and the research on these problems is on going.

Hinokidani(2005) pointed out that, in the future sand removal project, it is important to design the separate systems depending on the characteristics of removed sand. Characteristics of the sediment may be classified in terms of recyclability as well as impact on downstream environment. In this sense, the present MHSP should also specify the preferable sediment characteristics that can be efficiently discharged so that one can clearly see if the system is applicable when certain sediment conditions are specified.

8. Concluding Remarks

Studies on applicability of MHSP were performed through laboratory, field, and numerical experiments. Through these studies, physical mechanisms and overall performance of MHSP have been cleared while several problems still need to be solved. Development of this kind of new system often requires step by step improvements based on practical applications of the system. The development of the system is on going and we are hoping that, with grateful advices from various contributors, MHSP will be flexibly modified and improved corresponding to individual practical project.

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“Hydro Technology” For Discharge of Sediment from Dams and Weirs

Toru Hashimoto
Vice President, Kohwa Engineering Co. Ltd., Japan

Shuji Kumagai
Asunaro Aoki Corporation, Japan

Abstract

With long time operations of dams, reservoirs, silt basins, soils that have flown into such facilities would settle and present problems. The technology we are going to address here is a dredging technique to discharge sediment out of the facilities or to remove them from lake to downstream river. The Hydro Technology utilizes the natural water flows to discharge sediment under given conditions of water head, particle sizes of sediments, and therefore the discharging can be made with minimum energies. That is to say, because of the lower pipe friction losses the longer distance of discharging can be achieved. Moreover, risk of pipe blocking is very low because the Technology takes advantages of natural effect of slurry flows that has self-controlling function of transporting sediments at the best appropriate concentration. Furthermore, because of less chances of causing water turbidity within the reservoir, it is possible to perform the discharge without disturbing the original purpose of the reservoir for hydropower generation, etc.

In this presentation, we introduce the social significance of the Hydro Technology, explanation of its theories as well as displaying of application examples and the results of verification tests in relation to the Technology.

1. Introduction

In recent years, sediments deposited in reservoirs, lowering river bed in downstream regions, scouring cases in bases of structures such as bridges and riverbanks, and shore erosion are presenting serious problems from the viewpoint of river management. In the relation of natural environment and circumstances in which humans live, rivers and soil/sands generate very complicated problems of river management, in relation to environments as well as safety and maintenance. The flows of rivers are basically the slurry flows of water and soil that change characters under various factors including motion characteristics generated from erosion, transportation and deposition of soils, particular characteristics and properties of soil particles that may cause dispersion, coagulation and settlement. In addition, there are changes of natural conditions, passage of time, location as well as artificial works. These natural changes are not necessarily compatible to the human livings, which require consistent and constant conditions of infrastructures and social activities. Such incompatibility presents the problems of today.

In July 1998, the official report submitted by the sub-committee on general management of river titled “Toward integrated management of flowing soil” defined the movements of sands on a river along its total areas of basins from origin to offshore as “flowing sand system.” It called for actual measures based on an integrated management under the concept of this system. In one of the chapters of this report is titled “Development of Integrated Management.” It urged for establishment of effective measures to discharge sediment for any new projects of dam construction, and the establishment of a system to discharge the sediments from existing dams.

While the problem of sediment at dams and other reservoirs have various phases from its generation, sedimentation, dredging and transportation, erosion and finally flow-out, the hydro technology discussed here was developed in Norway as the most economical means for the dredging and transportation phase.

Features of the technology include:

1) Low-cost dredging and transportation is possible by utilizing the water head of the reservoir concerned.
2) The technology causes no turbidity of water within the reservoir, thus the dredging and the discharging of sediment can be performed from time to time without interrupting the original purpose of the dam. Water intake can be made, whether for power generation, irrigation and/or water supply.
3) Not being an operation at a fixed point, the dredging work can be performed at any point of the reservoir.
4) From the viewpoint of the principle characteristics

of this method, it should enable operators to carry out the dredging work of high concentration without risks of pipe blockage.

5) The principle characteristics of this method indicate that the dredging and transportation of the material with minimum energy requirements. In other words, the works can be performed with less pipe abrasions and with longer transportation distances.
6) Operation is simple and does not require a skilled operator

2. Principle Characteristics of Hydro Technology

The principle characteristic of Hydro Technology is quite simple. It just makes use of a natural phenomenon. (Refer Fig. 1) The hydroÅ|pipe, which has continued slots. in shape of intermittent slots in the lower portion, and laid underneath the sediment, is connected to the outlet pipes. The most upstream portion of the hydro-pipe, however, must be protruded from the surface of the sediment.

The operation of the hydro-pipe is performed in two phases as addressed below:

1) Wait until certain volume of sediment materials pile upon the pipes. One of the features of this technology is no blocking of the pipes because the slots are located on the bottom side.
2) When the outlet pipe is opened, water is suctioned into the pipe through the opening at the top of the pipe, protruding from the surface of the sediment materials. At the same time, sediment materials are also suctioned with water and discharged to the downstream. When the entire materials are suctioned to nil, the suction pipe moves toward downstream.

If the concentration of the slurry within the pipe would increase, frictions within the pipe would also increase and the velocity of the slurry drops, subsequently the suction would be decreased. The decrease of suction, then would help decrease of the slurry concentration. As a reverse phenomenon, the drop of the slurry concentration would help increase of the velocity of the slurry inside the pipe and consequently would increase suction and therefore slurry concentration. So, under the given condition in the most appropriate situation (minimum energy), the discharge of the sediment can be achieved in a self-adjustment process. The process should automatically prevent the blocking within the pipe.

The slurry flows theoretically make the soil particles in the saltation flow to floating situation when the velocity is slightly faster than the limit deposit velocity. However, the kinetic energy of soil particles perpendicular to the flow is not high. Therefore, the energy consumed for the particles collisions to the tube wall or among the grains would be minimum, namely the flow can be achieved with minimum friction loss and minimum flow gradient. The flow in this situation is called heterogeneous flow. Incidentally, the faster flow than this is called homogeneous flow. In this case, the concentration in the cross section of perpendicular to the flow will be the same but the friction loss of the flow would become larger. For the planning of installing the Hydro-technology facilities to discharge the sediment materials, it is not ignorable to predict the limit deposit velocity. There are several calculation methods for the limit deposit velocity. They were proposed by Durand, Wilson, Gilie, and others. For engineering purposes "Sedimentation Engineering" edited by Vito A. Vanoni of the ASCE(American society of Civil Engineers) Task Committee says the Durand method is safe enough. There are considerable differences in the calculation results by the respective methods, largely by the mean particle size of sediment and the inner diameter of the pipe. The hydraulic gradient required for discharging the slurry at the limit deposit velocity may be calculated as a value in expression of clear water with the Durand-Condolios methods or others. To perform these calculations, several characteristics of the soil properties including kinematic viscosity, drag coefficient and others are required. Still, variations of particle sizes, shapes of the sediment and the concentration of the slurry would make it difficult to establish the soil properties, although there are some formulations based on various assumptions.

All the requirements may be established through calculations but the results of these calculations are, as mentioned, based on assumptions. Therefore they should be considered as guidelines for the engineering. To obtain appropriate values, it is desirable to establish them through laboratory tests and/or small-scale experiments. We believe it is the basic principle of the engineering.

3. Types of Hydro-Technology

Types of the Hydro-Technology may be classified in two types, depending on the shape of the hydro-pipe, a straight one and a curved to the shape of "J" charac-

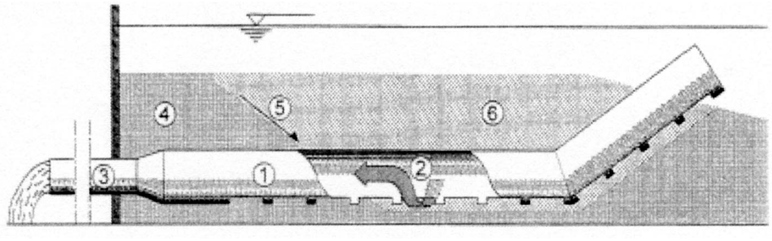


Fig.-1 Basic Composition of Hydro-Technology System
① Hydro-pipe ② Sucking point ③ Outlet Pipe ④ Sediment
⑤ Sediment collapsing toward the sucking point ⑥ Discharged portion

Table -1 Features of Hydro-Pipe and Hydro-J

Type	Conditions for application			
	Installation	Operation	Installation Method	Mobility
Hydro-Pipe	Buried in the sediment but for the portion of outlet pipe.	All operations can be done with closing and opening of valves.	Installation for new dam is easier. For installation to the existing facilities, advance dredging is necessary. Hydro-J is applicable for advance dredging.	Difficult to move once it is installed.
Hydro-J	J-pipe is hung in water. The outlet pipe portion can be positioned either in water, on the surface of in air.	It is hung and lifted by crane either from land or from a dredger. The pipe is movable with a winch. Operations can be done with closing and opening of valves.	Installation is easy.	Movable.

ter. While the principle of the suction dredging is the same, there are some differences on operation and features of the respective types. The former is called Hydro-Pipe and the latter Hydro-J. The Hydro-Pipe is used as a fixed facility for operation of longer periods with minimum operation process. On the other hand, the Hydro-J is mobile and more hands are required for operation. Table-1 below shows the basic features of both types.

For these two types the most desirable method of generating water flow is the utilization of the difference of water levels in and out of the dam. However, we do not necessarily stick to this method. Depending on the method to generate water flows inside the pipe and the composition of the equipment, there are several ways of dredging as described below:

3.1 Methods based on Hydro-pipe

3.1.1 Hydro-PB

This is the operation in the most basic form. Install the slotted Hydro-Pipe on the bottom of a dam or a shaft in opposite direction of the sedimentation to discharge the sediment through outlet pipes to outside the lake. The difference of water levels between outside and the inside of the reservoir, or similar difference of water levels inside the reservoir and the shaft should generate the water flow.

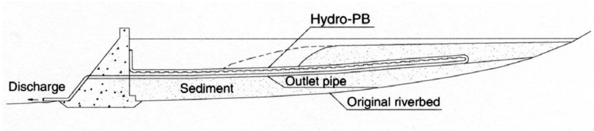


Fig.-2 Hydro PB

3.1.2 Hydro-PS

This is a method to discharge sediment materials into the shaft or hopper by differentiating the water levels of the lake and inside the shaft or hopper. The water level within the shaft or hopper is lowered by pumping or promoting natural drainage. It is possible to remove the sediment inside the shaft or hopper to outside the lake with a hydro-pipe installed on the bottom.

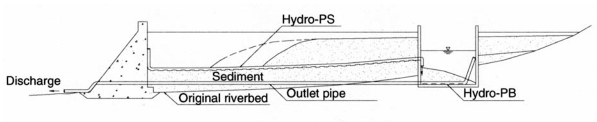


Fig.-3 Hydro PS

3.1.3 Hydro PP

Where water level differentiation is not applicable to generate water flows, connection of pumping equipment is used to generate flows for carrying out suction dredging. Because any mechanical driving system is not involved in the Hydro-pipe system itself, possibility of pipe blockages by debris or large particles is rather low. However, pumps generally have mechanical driving system like impeller, which are vulnerable to such debris and large particles. Therefore, it is recommended to select appropriate type of pumping equipment that has not mechanical driving system, such as a jet pump. Also, from the viewpoint of energy saving, it is desirable to establish the level of the sand discharge point as low as possible.

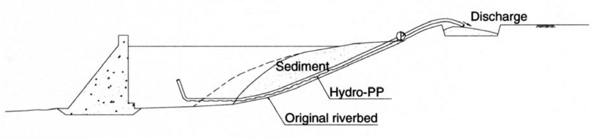


Fig.-4 Hydro-PP

3.2 Methods based on Hydro-J

3.2.1 Hydro-JB

A dredger or a working barge lifts a "J" pipe that dredges sediment materials and discharge through outlet pipes to outside the lake. Water level difference in and out of the lake generates the water flow.

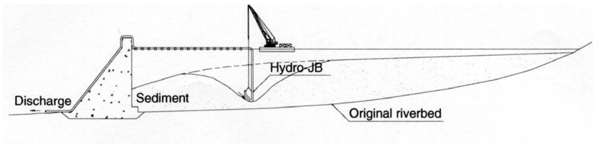


Fig.-5 Hydro-JB

3.2.2 Hydro-JS

A dredger or a working barge lifts a "J" pipe that dredges and transports sediment materials to a shaft with differentiated water level.

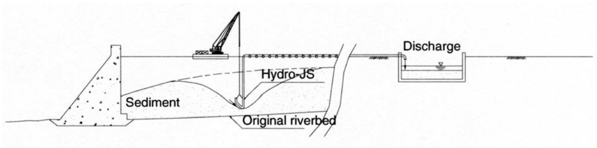


Fig.-6 Hydro-JS

3.2.3 Hydro-JT

A dredger or a working barge lifts a "J" pipe that dredges and transports the sediment materials to a tank having lower water level or into a hopper barge. Flow is generated by differentiated water levels of the dam and the tank or the shaft of which the water levels are lowered by pumping processes. The bottom-hopper type barge would be preferably used to transport the sediment.

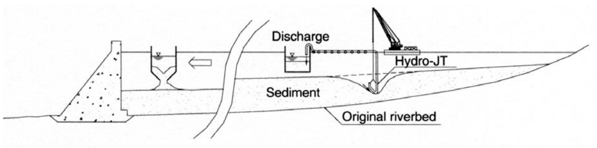


Fig.-7 Hydro-JT

3.2.4 Hydro-JP

Where the water level of the discharge point is higher than that of the lake, a pumping system is used in midway of the outlet pipe in order to transport the sediment. Similarly to the requirements for Hydro PP, selection of the pump type and making the level of the discharge point as low as possible are desirable requisites.

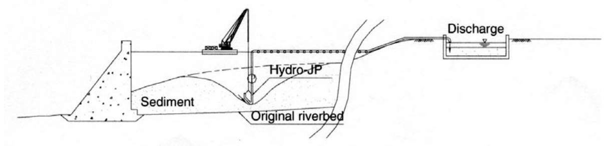


Fig.-8 Hydro-JP

3.3 Combination

A typical example of combined use of the above technologies is illustrated in Fig. 9 below.

In the upstream area of the dam or reservoir, dredging is performed with Hydro-PP to move the sediment to the downstream area. In midstream area the dredging work is carried out by Hydro-JT and transport the dredged material with bottom hopper type floating tanks to the downstream area. The sediment materials so collected in the downstream area are finally discharged by Hydro-PB method. Sizes of the downstream basin mentioned here vary from a few meters to several kilometers by the particle sizes, thickness of the sediments and applicable water head differentials. The range is limited in a range where the Hydro-PB is applicable. The midstream area must be deep enough to allow accesses of dredging or working barges. The area of course may be expanded toward upstream area with the progress of the dredging works. There are some difficulties in the upstream area to secure water level differentials.

4. Suitability Tests of Hydro-J Method

Following is the results of suitability tests on Hydro-JT technology performed at Sakuma Dam Reservoir belonging to Dengen Kaihatsu (Electric Power Development Co. Ltd. known as J-Power)

Testing Facilities:

1) A working barge of 30 x 10.5 meters loaded with a 35-ton crawler crane to suspend Hydro-J pipe.
2) A 2,5meter dia. floating tank fixed to the working barge
3) A 250mm dia. Hydro-J pipe and a 60-meter long outlet pipe with 200mm diameter.
4) A γ-ray densitometer and an electromagnetic discharge meter
5) A sand pump of 200mm, portable power generator, etc.

Testing method:

1) The Hydro-J pipe is lifted down to the bottom of the reservoir of about 20 meters deep.
2) Operate the sand pump inside the floating tank, for lowering the water level in the tank (3-5 meters).
3) With the lowering of the water level in the tank, dredged sediment materials began to flow into the tank from the Hydro-J pipe.

Mud and sands subjected to the test mainly composed of silty fine sands with the grain size below 2mm. (some gravel stone of minimum size of 140mm

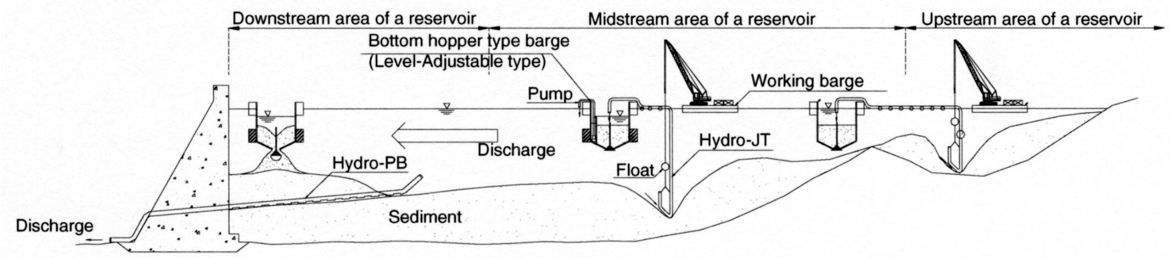


Fig.-9 Combination of Hydro Technologies as well as sunken tree branches of 20mm dia., 60mm long were also included.)

Findings from the results of the tests included:

1) The absence of mechanical elements within the Hydro-J pipe and outlet pipe showed ability to dredge out considerably large objects mentioned above, namely, gravel stones and sunken wood branches. In comparing the fact that there were considerable residues in the floating tank of gravels and other foreign objects which sand pump system could not take, the Hydro-J system was quite effective to dredge out miscellaneous objects.
2) Supposing the increasing ratio of gradient of slurry flow to that of clear water flow on various losses of intake, valve, bending, and discharge are equal to the increasing ratio on pipe friction loss, we tried to express the results in the formula to obtain utilizable dimensionless excess head loss (ϕ) and calculated following value:

   $\phi = (i_m - i)/(i^*C_v) = 6.78$

   where:

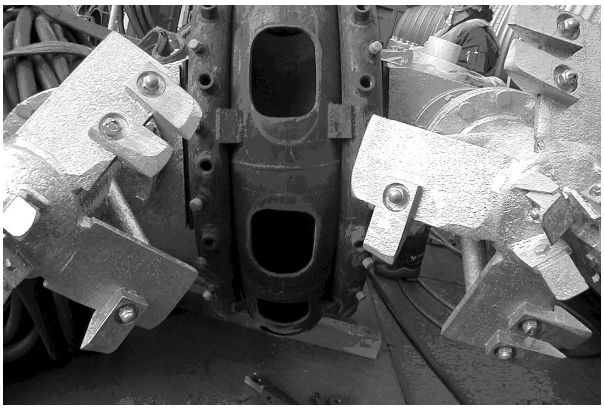
   i_m = hydraulic gradient of slurry as expression of clear water gradient

   i = hydraulic gradient of clear water

   C_v = concentration by volume of slurry.

   The value almost agreed with the calculated value.
3) The density of the discharged sand applied to the maximum water head difference of 5 meters was about 9% in terms of volumetric concentration. Taking into account of losses of in-take, discharge, bending and valve, for the slurry at this maximum point, we can assume discharge of the tested subjects could be achieved with a hydraulic gradient of about 1.8%. This may be converted to ability to transport and discharge for a distance of more than eight kilometers at the water head difference of 150 meters.
4) From 2) and 3) above, we could confirm that this method being practical with capability of dredging and discharge dense sediment materials of considerably long distance.

Photograph-1 Working barge with a crane to hang Hydro-J pipe



Photograph-3 Slots of Hydro-J Pipe

Photograph-2 Hydro-J Pipe

Photograph-4 A group of measurement instruments

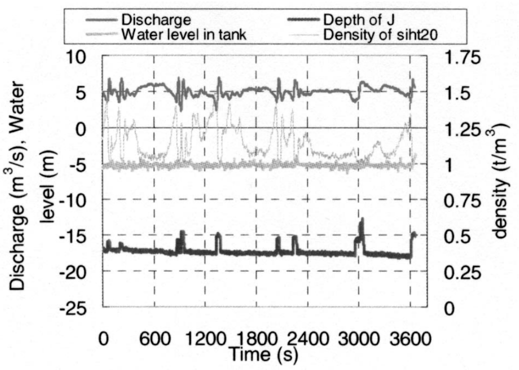


Fig.-10 Excavation Log

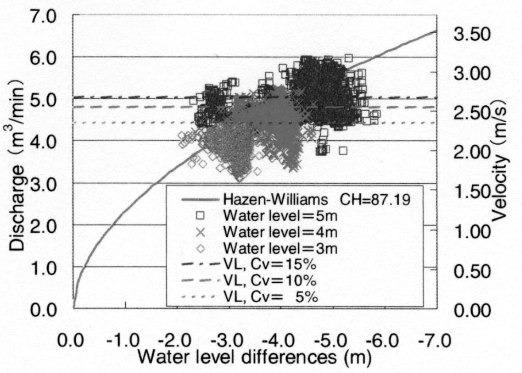


Fig.-11 Relation between the water level differences and discharge volume

5. Laboratory testing of Hydro-pipe method

Following is a summary of laboratory tests on Hydro-pipe method held at the laboratory attached to Asunaro Aoki Construction Co. Ltd.

Testing facilities:

1) A water tank of 1meter wide x 10 meters long x 1.2 meters depth.
2) A Hydro pipe, 100mm dia., 10meter long and an outlet pipe 75mm dia., 7meter long.
3) Slot size of Hydro-J pipe: 60mm wide with 180mm long
4) Difference of water levels from upstream side and outlet at downstream side was 1.3 meters.
5) Measurement instruments included an ultra sonic distance meter and load meter to measure discharge and density and measurement of pressures inside the pipe, using manometer and pressure transmitter.

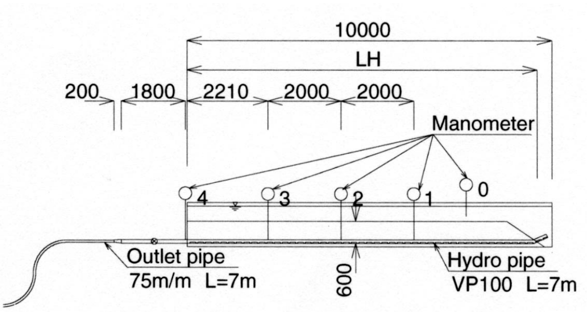


Fig.-12 General Outline of Testing Facilities

Material subjected to test:

Mountain sand with main component of medium to fine grains under particle size of 4milimeters taken from Shimodate Region, Ibaraki Prefecture, containing about 2% of silt under 75μm. d_{50}=0.7~0.8mm.

Test Method:

1) Fill the water tank with water and put test material to the height of 600mm from the bottom of Hydro-pipe and let it piling up...
2) Keep overflowing of the tank with fresh water in order to maintain consistent water level.
3) Open the main valve to operate the Hydro-pipe.

Table 2 shows the results of the measurement. Figures 13~16 are the results expressed in graphs.

t = time passed since opening the valve

L_H = Hydro Pipe's length covered with sediment material

V_H =Velocity of slurry inside the Hydro Pipe

C_v =Volumetric concentration of Slurry

I_m =Hydraulic gradient of slurry as expression of clear water

H_d =Pressure differentials between the pressure inside the pipe at the downstream end of the pipe and the static water pressure (water head as expression of clear water). It indicates the difference of manometer reading of 0-4 in Fig. 4.

V_s =Discharged volume of sediment materials (true volume)

Table-2 Results of Laboratory Tests

t (s)	L_H (m)	V_H (m/s)	C_v	i_m (mH$_2$O/m)	H_d (mH$_2$O)
1200	9.0	0.92	0.025	0.070	0.75
4200	7.0	0.95	0.045	0.089	0.68
5700	5.0	0.87	0.050	0.108	0.65
7200	2.5	1.08	0.080	0.148	0.43
8340	1.3	0.76	0.045	0.128	0.28

Note: When L_H =1.3 meters, the valve was closed to half-opening to avoid possibility of excessive velocity.

From these results, we could find the followings:

1) When the LH decreases, the concentration of the slurry proportionately increases (Fig.-13). This fact forms the basis of similar trend in the relation between the LH and the hydraulic gradient. (Fig. -14)
2) The time and the coverage length represent the functional relations on upward convex (Fig.-15). It shows the discharge is accelerated with the passage of time.
3) H_d is the pressure differences of inside and outside of the outlet. This is one of the elements affecting the continuity of discharge operation and the stabil-

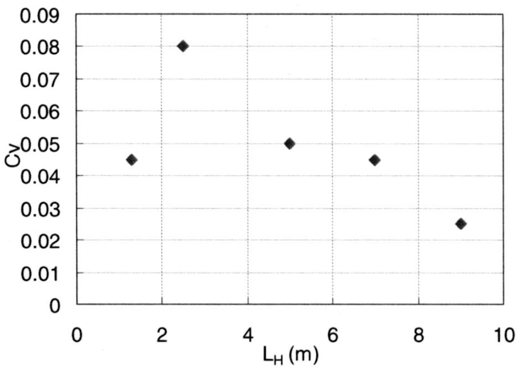


Fig.-13 Relations of L_H and C_v

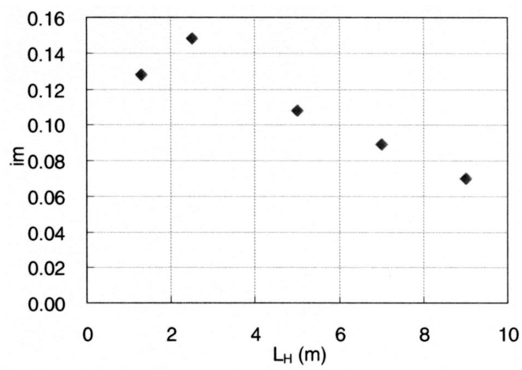


Fig.-14 Relations of L_H and hydraulic gradient

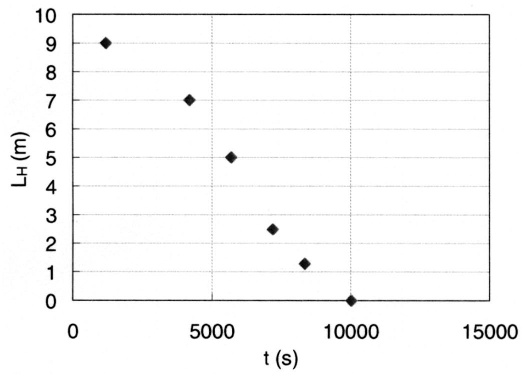


Fig.-15 Relations of time and L_H

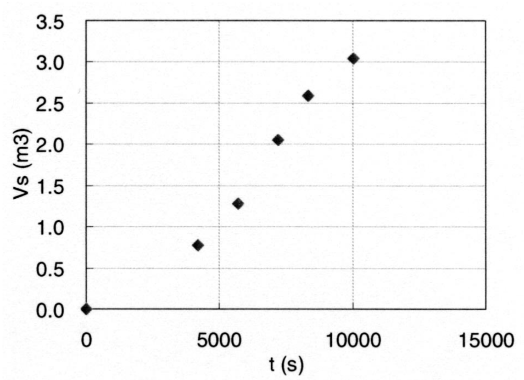


Fig.-16 Relations of time and Aggregated volume of discharges

ity of the sediment at the bottom of Hydro-pipe. The test results showed that Hd was maximum at the start of the operation but it decreased with the passage of time. (Table 2)

These test results tell that if the continuity of the operation could be maintained, then the work efficiency would be appreciated in the hydro-pipe technology and, therefore, it is very important to find the relation of pressure difference and of sediment at the bottom of pipe.

While these tests were performed under single and simple condition (pipe diameter and the material subjected to the tests), they were helpful for us to predict the qualitative character of the technology and that the hydro-pipe technology is an indicative method for displacing sediment in considerably wide area with simple facilities and operations.

Further research would be necessary to pursuit the discharging capability and maintaining continuity of the operation in the future. The quantitative aspects of this technology should be introduced from the actual work results under various conditions and circumstances and researches and analysis in the actual works and achievements.

Conclusion

Hydro-Technology which has many epoch-making features enables to discharge sediments more effectively and economically with optimum combination with various other facilities. And this technology has been confirmed on its effectiveness and practicability of dredging and discharging in experiments both on site and in laboratory. Therefore, Hydro-Technology would be mighty available method to solve the problems of sediment at reservoirs that have been growing serious year by year.

Current Conditions of Reservoir Sedimentation and Turbidity in Irrigation Dams in Japan

Hiroyuki Taruya
Department of Hydraulic Engineering,
National Institute of Rural Engineering, Japan

Abstract

This paper describes results and analysis of some investigations on reservoir sedimentation and turbidity problems of irrigation dams in Japan. In general, the catchment area of the irrigation dam is smaller than that of the dam for electric power generation when the storage capacity is identical. As a result irrigation dams have unique characteristics on sediment management as follows; 1) C/I ratio and specific sediment rate of irrigation dams are comparatively larger than those of other ones. 2) As turnover rate of an irrigation dam is smaller and the water level fluctuation is larger than those of other dams, sediment movement is very active in a reservoir. These facts can be made sure by echo sounding exploration that is also introduced as a case study. Sedimentation problems and turbidity problems are in a contradictory relationship. In a case where it is judged to be necessary to discharge turbid water, discharging a specified quantity during a flood is recommended as a countermeasure.

1. Introduction

In order to predict future sediment trends including a study of the appropriateness of the initial plan, it is essential to clarify sedimentation and hydrological history, and catchment basin development trends to analyze trends in sedimentation in a relatively long time span extending from the opening of the dam to the present time. And to clarify the present state of sedimentation to select appropriate countermeasures backed up by progress in observation and exploration technologies, it is essential to carry out an emergency concentrated investigation of sediment routing. By the way, there are some distinguishing characteristics in reservoir sedimentation and turbidity problems of irrigation dams.

An irrigation dam provides multiple functions, but its basic function is to store water. The deposition of sediment in a dam effects its storage function, but the speed that this occurs varies according to the characteristics of the catchment basin and the reservoir management[1]. In this paper, appropriate sediment management for irrigation dams is studied. Current condition of reservoir sedimentation and turbidity are studied and introduced for the purpose of applying appropriate countermeasures in irrigation dams in the future. And a case study of sediment routing of an irrigation dam is also introduced.

2. Sedimentation characteristics of irrigation dams

In this chapter, the sedimentation characteristics of the irrigation dam reservoirs are discussed based on the investigation data.

2.1 C/I ratio

The ratio of C to I (C/I) is designated as C/I ratio in this paper, where C (m^3) is the storage capacity of the reservoir and I (m^3) is the annual inflow of water into the reservoir. C is one of the factors related to the reservoir factors, and I is one of the watershed factors. That is, the C/I ratio is a dimensionless value related to both factors. Moreover, the C/I ratio is a reciprocal number of the annual revolving rate (I/C) of the reservoir. It is obvious that the annual revolving rate of the reservoir markedly affects the reservoir operation or hydraulic conditions of the reservoir, the tributaries, the watershed, etc. Therefore, it is assumed that the C/I ratio is closely correlated with the reservoir sedimentation process. The analysis of reservoir sedimentation based on the C/I ratio is important to clarify the sedimentation factors and the sedimentation characteristics in the irrigation dams.

2.2 Catchment area and C/I ratio of irrigation dam

In general, water storage in an irrigation dam is designed in considering the annual revolving rate of the reservoir. As a result, suitable catchment area which can supply a sufficient volume of water as that planned, is selected. On the other hand, in a dam for electric power generation both the volume of water and the water level must be secured. The catchment area of the irrigation dam is smaller than that of the dam for electric power generation when the storage capacity is identical.

Fig.1 shows the relationship between the water storage capacity and watershed area of each dam. Sedimentation data of the irrigation dam were originally collected in this study and the previous data, mainly

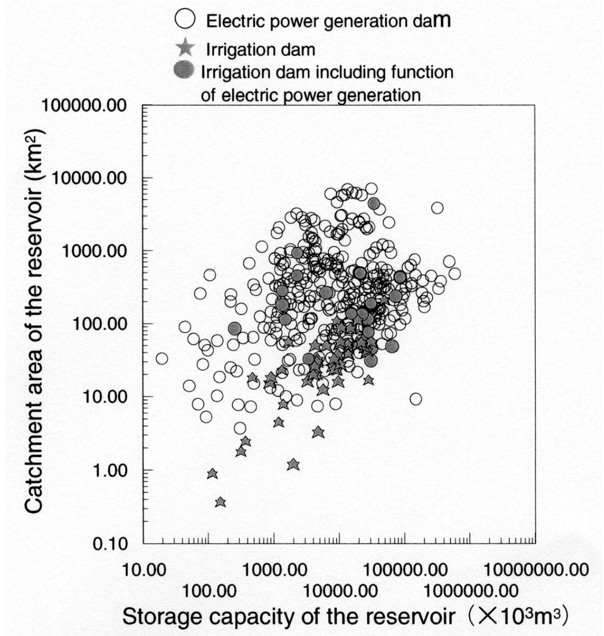


Fig. 1 Relationship between storage capacity and catchment area

data for electric power generation[4], were used in the analysis. The symbols in Fig. 1 indicate the kind of dams, either for irrigation or for other purposes. Fig. 1 shows that the dams for electric power generation have large catchment areas. Assuming that all the dams sampled had the same storage capacity, the watershed area F and the annual water inflow I of the irrigation dams would be smaller than those of the power generation dams. In fact, based on the same assumption, the C/I ratio of the irrigation dams is considered to be larger than that of dams for electric power generation.

2.3 C/I ratio and annual sediment deposit rate

Fig. 2 shows the relationship between the C/I ratio and annual sediment deposit rate of the dams[1,3]. The purpose of the reservoirs is indicated by the symbols. The annual sediment deposit rate is defined as the average percentage of storage capacity lost by sedimentation in a year (% year). As stated in the previous paragraph, the values of the C/I ratio of an irrigation dam are mainly included in the range of the value zone indicated in Fig. 1. Moreover, their distribution is in agreement with the general tendency observed in Fig. 2. Furthermore the plots of their distribution show a smaller value for the following reason. The annual sediment deposit is expressed as Q/Y/C, where Q is the total volume of accumulated sediment (m^3), Y is the number of years after dam construction, and C is the storage capacity (m^3). Assuming that A and B dams have the same degree of storage capacity C, if the annual revolving rate (I/C) of A dam is larger (C/I ratio is smaller) than that of B dam, the annual sediment deposit (Q/Y) of A dam is considered to be larger in general. It is generally assumed that the values of Q/Y/C of irrigation dams are relatively smaller than those of other dams, In other words, it is considered that in the irrigation dams the sedimentation stages are longer than in the power generation dams. Therefore, it is assumed that the irrigation dams are located in suitable sites for avoiding the sedimentation problems.

2.4 C/I ratio and specific sediment rate

The results can also be interpreted by paying attention to Q/Y which corresponds to the amount of annual sediments deposited in the reservoir. At first, it is assumed that all the dams sampled show the same values of F and Qin, where Qin refers to the annual sediment volume transported into the reservoir. As the irrigation dams tend to trap sediments effectively and Q/Y/F of an irrigation dam shows relatively larger values (Fig. 3).

Fig. 3 shows the relationship between the C/I ratio and specific sediment rate[1,3]. The purpose of the reservoir is indicated by the symbols. The specific sediment rate is defined as the total amount of accumulated sediments deposited in the reservoir per unit catchment area, averaged in a year ($m^3/km^2/year$). The distribution of the specific sediment rate of irrigation dams shows a similar pattern to that of other dams as depicted in Fig. 3 and larger values are recorded, especially when the C/I ratio is close to 0.1. The macroscopic tenden-

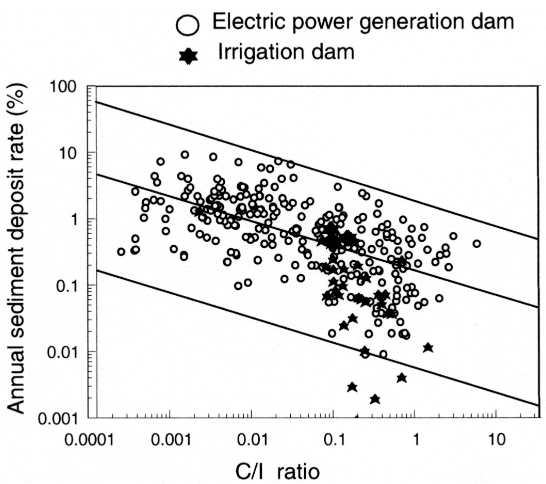


Fig. 2 Relationship between C/I ratio and annual sediment deposit rate

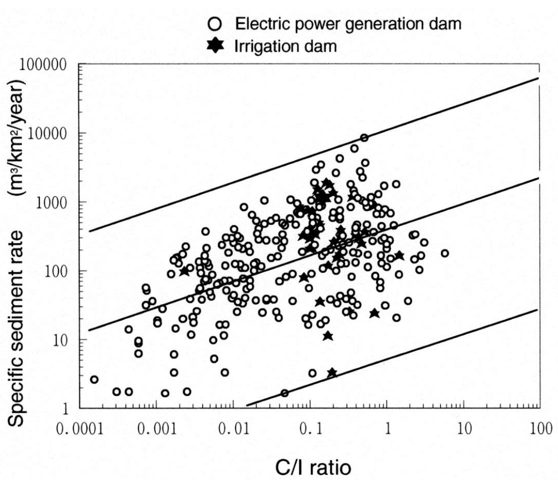


Fig. 3. Relationship between C/I ratio and specific sediment rate

cies mentioned above can be explained as follows. The specific sediment rate is expressed as Q/Y/F, where F is the watershed area (km^2). First of all, attention should be paid to the value of F and it should be assumed that these dams have the same storage capacity C. It was already indicated in Fig. 1 that the watershed area F of irrigation dams tends to be smaller than that of the power generation dams. Ashida et al.[2] observed that the specific sediment discharge rate shows a reverse correlation with the watershed area F. The difference between the specific sediment rate and the specific sediment discharge rate was disregarded here for simplicity, since the specific sediment rate (Q/Y/F) of the irrigation dams is considered to be larger than that of other dams. These observations are in agreement with the results indicated in Fig. 3.

3. Turbidity Problems in Irrigation Dams

This chapter describes the organization and analyses of the results of the investigation that were done to show trends in turbidity problems at irrigation dams and at the same time to consider appropriate future countermeasures and ways to properly manage dams to deal with turbidity problems at the drainage basin level and reports on the results of these studies.

3.1 Outline of the investigation of the state of turbidity at irrigation dams

3.1.1 Method

The questionnaire forms were distributed to dam managers through agricultural administration offices and regional development bureaus by the Construction Division of the Ministry of Agriculture, Forestry, and Fisheries. The dam managers were asked to answer the questions in the questionnaire. The forms were distributed in March 1995 and the answers were collected in about one month.

3.1.2 Respondents

The respondents were managers of organizations that actually manage dams in the field (national government, prefectural, land improvement district, etc.). The questions included some regarding the present state and causes of the problem of turbidity, but it is assumed that the answers reflect each respondentÅfs personal understanding and suppositions.

3.1.3 Districts investigated

Agricultural administrative bureaus selected major irrigation dams that were constructed as part of nation-

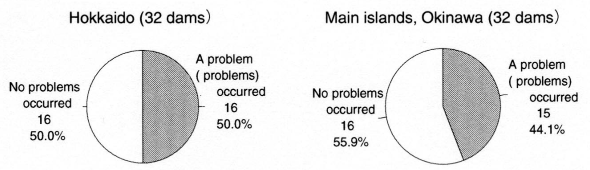


Fig. 4 Whether or not Turbidity Problems Occurred in the Districts that were Investigated

al land improvement projects and are still in operation. Dams in a total of 66 districts (35 districts by the Hokkaido Development Bureau and 4 districts by each of the Regional Development Bureaus) in Japan were investigated. In the remainder of this report, the dams will be represented by the numbers 1 to 66, with dams 32 to 63 located in Hokkaido. All are dams used for irrigation. Of the 66 dams, those with other purposes in addition to irrigation (electric power generation, disaster prevention, etc.) were located in 13 districts (19.7%) (below, these are categorized as agricultural use dams "that also have electrical power and disaster prevention functions"). They also include 21 districts where the dam has not been in operation for 10 years (32%).

3.2 Investigation results

3.2.1 Occurrence/non-occurrence of turbidity problems

"Turbidity problems" in this report refers to problems caused by a change or an increase in turbidity in the flowing water revealed by a comparison of the turbidity in the so-called natural river before construction of the irrigation dam with the turbidity after the construction. Therefore, it is predicted that places and dams where turbidity is considered to be a problem and its causes will be mixed in each case. Fig. 4 shows the breakdown of the results of the answers by district. It reveals that nationwide, turbidity problems have appeared in about half the districts.

3.2.2 Parties complaining of turbidity problems

If the parties complaining that turbidity presumably caused by a dam is a problem are clarified based on the results in Table-1 are obtained. Most of the sources of the complaints are fisheries associations, but they include farmers on irrigated land, residents, and nature conservation groups.

3.2.3 Principal geology of the drainage basins

Fig. 5 shows the relationship of the results of answers to question 4 concerning whether or not tur-

Table-1 Source of the Problem Causing the Turbidity and Details

Individuals or organizations that have made inquiries (or submitted complaints) regarding turbidity	Description of the inquiries (or complaints)	Number of districts investigated where this has occurred (multiple answers)
Fishing association related	Turbidity	11
Farmers	Turbidity, odors, blocking of irrigation systems, reduction of product quality	6
Water supply service related	Turbidity	2
Residents and nature conservation bodies etc.	Turbidity, odors	6

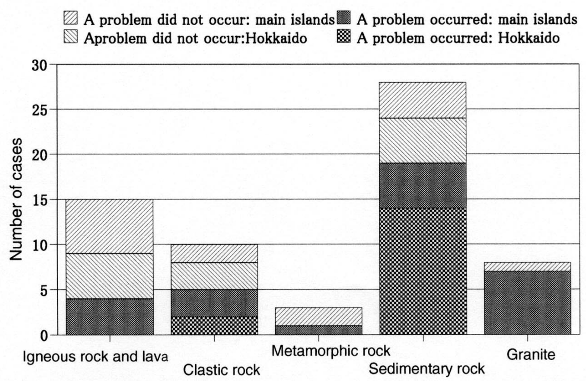


Fig. 5 Occurrence/Non-occurrence of Turbidity-Drainage Basin Geology Relationship

bidity problems have occurred with the principal geology of each dam's drainage basin. The results show a high rate of occurrence of turbidity problems in districts that include sedimentary rock geology and in districts including granite and clastic rocks.

3.2.4 Dam intake and discharge methods

Table-2 shows the relationship of the answer to question, whether or not problems have occurred, with the intake and discharge methods (whether or not ponded water is drained) of the dams. Here "draining ponded water" refers to artificially lowering the water level in the reservoir at the end of the irrigation period. The intake method was judged from sketches of the intake facility that respondents were requested to include with the completed questionnaire. Because many of the dams that were included in this survey use the floating type intake method (particularly in Hokkaido), many problems occurred at such dams, but no significant differences between the rate of occurrence of problems was found according to the intake method.

3.2.5 Factors causing turbidity (factors hypothesized based on respondents' answers)

Written responses concerning causes of turbidity hypothesized by the managers are categorized from A to I in Table-3 Legend. The turbid substances that originally caused the turbidity are sediment and the skele-

Table -2 Turbidity – Dam Intake Method and Ponded Water Drainage Relationship

Ponded water drainage after irrigation period / Intake method	Ponded water is drained		Ponded water is not drained	
	A problem occurred	A problem did not occur	A problem occurred	A problem did not occur
Intake tower, surface intake gate method	2	3	5	0
Intake tower, floating method	11	9	1	3
Selective intake gates inside the dam body	1	1	4	5
Inclined drain, bottom drain	1	3	4	1
Other	0	0	2	1

Table- 3 Period and Frequency of Turbidity and Causal Factors based on Hypotheses of Dam Managers

Duration of each occurrence of turbidity / Annual frequency of turbidity	1 day~1 week	1 week~2 weeks	2 weeks~1 month	1 month or more
Once	**42**(A,H) 65(A,B)			1(A), (A,B) 16(A)*, 19(C) 24(C), 28(B,D) **36**(E), **63**(A,B)
Twice	**51**(A,B,G)	**40**(G,H)	13(A,B,D,I)	12(A,F), **52**(A,B) **53**(A,B)
Three times	20(A,B,D)	**48**(A,B,G)		18(A,B,D)*, 66(A,B)
Four times or more	29(A,B)	**61**(A,D) **62**(A,D)		

Legend of factors hypothesized by dam managers to cause turbidity
A By rainfall (flooding) in the dam drainage basin
B By the geology or vegetation in the dam drainage basin
C By high temperatures or continuous dry weather in the summer
D By the fall of the reservoir level
E By work upstream from the dam
F By sedimentation inside the dam reservoir
G By the discharge of ponded water
H By the formation of humus or algae by fallen leaves
I By the agitation of the irrigation water
Note 1) 25 (A,B,D) indicates that it is dam No. 25 and the factors causing the occurrence of turbidity are A, B, and D.
Note 2) Numbers printed in bold are dams under the jurisdiction of the Hokkaido Development Bureau.
Note 3) The symbol "*" indicates dams with purposes other than irrigation (electric power generation, disaster prevention).

tons of living organisms produced inside the drainage basin and the reservoir. Their behavior is governed by the movement of the water that was the medium for their transport. There is a method of hydraulically and hydrologically analyzing the deposition and flow in a reservoir of sediment produced in the dam drainage basin as a “dam sedimentation mechanism.” Because a turbidity problem is a problem concerning the transport of turbid substances including sediment, it is possible to hypothesize the causes of turbidity problems based on primary causes of the problem of dam sedimentation. Primary causes of dam sedimentation can be generally categorized as drainage basin primary causes and as reservoir primary causes[1]. Among the reservoir primary causes, primary causes that are results of the characteristics of management that are particularly important in the case of an irrigation dam are considered, and part of the reservoir primary causes are categorized as management primary causes. Based on these concepts, the factors hypothesized by dam managers in Table-3 are categorized as (1) drainage basin primary causes A, B, C, and E, and as (2) management primary causes D, G, and I, and as (3) other reservoir primary causes F and H.

In Table-3, causal factors hypothesized by dam managers were categorized according to the annual occurrence frequency of turbidity phenomena, or in other words “frequency”, and the duration of each turbidity phenomenon that has occurred. Frequency presumably differs according to each respondent’s impression and feelings throughout the year. But the overall survey has shown that without doubt, most phenomena occurred with frequency of once or twice a year and continued for a long duration of more than a month. In addition, the results of this investigation have shown that the times of the occurrence of turbidity phenomena are mainly concentrated from July to September.

3.2.6 State of implementation of countermeasures in the field

Table-4 organizes the state of implementation of countermeasures taken to deal with turbidity problems by the district where turbidity problems occurred. Countermeasures have been taken (or are planned) in 16 districts, that is about half of all the 31 districts where problems occurred. No responses were received regarding many of the other districts, but in some cases, measures have not been taken (or cannot be taken) for a variety of reasons: there is no countermeasure method, the cost of countermeasures is too high, or the problem does not harm irrigation.

Countermeasure methods are broadly categorized into three types (1) measures to prevent turbidity in the drainage basin that is the source of the production of turbid substances, (2) measures to prevent turbid water from flowing into the reservoir or to remove turbid substances that have flowed in, and (3) measures to prevent the intake or discharge of turbid water by surface intake or by discharge etc. (study of intake and discharge operation methods as non-physical measures and improvement of systems as physical measures). These measures are taken in response to the hypothetical causal factors in Table-3, and are the grounds for the categorization of turbidity problems by phenomenon and by cause, but these are discussed below.

3.3 Considering the survey results

Based on the above section, the following predictions regarding turbidity phenomena have been made.

1) Because the total discharge from an irrigation dam is generally lower than at other types of dams, it is hypothesized that the absolute quantity of turbid water is small. It can also be stated that the quantity of water required to dilute turbid water is low, because it is in a water environment where the turbid water density rises easily.

Table 4 Description of the Implementation of the Measure

Category of countermeasure method	Specific method	Dam No.
Change of the intake or discharge methods	Surface intake/discharge	1,28*.29,**48**, **51,52,53**,65
	Bottom water discharge	1
	Change of the intake opening	**63,**66
	Advance drainage of ponded water before a flood	**61,62**
Improvement (reconstruction) of the intake or discharge system	Installation of a pipe filter	65
	Installation of a sludge outlet valve	64
	Enlargement of the discharge pipe diameter	16*
Purification inside the reservoir	Dredging work	2,**36**
	Installation of water purification system	24
	Replacement of the water	**63**
	Ensuring an alternate route for inflowing water	18*
River basin countermeasures	Construction of a sabo dike	**61**
	Adjustment of the work period	**33**

Note 1) Numbers printed in bold are dams under the jurisdiction of the Hokkaido Development Bureau.
Note 2) The symbol “*” indicates dams with purposes other than irrigation.

2) It is assumed that eutrophication occurs readily in a reservoir with a long retention time. At the same time, reservoirs of this kind generally stratify easily, resulting in a danger of a dense flow of muddy water causing high density turbidity.
3) As shown by A, B, in the Legend in Table-3, there are assumed to be close relationships between the geological and rainfall conditions of the drainage basin and the occurrence of turbidity. The impact of geology is extremely clear as already shown by Fig. 5. In the estimation equation of the specific discharge of flooding in drainage basins, as the area of the drainage basin increases, the specific discharge declines in proportion to the square of its reciprocal[5]. This suggests that because an irrigation dam has a smaller drainage basin area than that of other types of dams, the specific discharge of the drainage basin of an irrigation dam may be relatively larger than that of other types of dams. In this case, there is danger of suspended load or wash load that are the principal constituents of turbid substances being produced easily[1].
4) As shown by comparing an irrigation dam with an electric power generation dam, the turnover rate of an irrigation dam is small and the water level fluctuation is large. This means that there are many cases when water intake is also required when the water level is low. Normally, it is extremely dangerous to take in high concentrations of turbid water and turbid substances that have been left by surface intake and retained or deposited in the bottom layer. This is probably conspicuous during droughts.

4. Investigation of Sediment Routing in an Irrigation Dam Reservoir

In this chapter, a study conducted using sample cases to answer two questions is described. To what degree can information be obtained by observations at fixed intervals of several years of present sediment routing done as part of efforts to quantitatively predict future sedimentation and how can this information be applied?

4.1 Investigation methods

4.1.1 Investigation location

Dam A, an irrigation use rock fill dam with a catchment basin of 55km^2, has been an important source of water irrigating 7,118ha of land since it was completed in 1983. Its upstream basin is steep and its valleys narrow, it includes volcanic sediments produced by mud flows and pyroclastic flows, and sediments flow easily through its topographical and geological environment, and judging from the results of continuous observations of the quantity of sediment made in recent years, there is a danger that continued sedimentation will reduce the storage capacity of its reservoir.

4.1.2 Investigation method

The progress of sedimentation of a dam is governed by the quantity of sediment produced in its catchment basin and its transport mechanisms. The annual advance is generally slow, but in the case of Dam A that was investigated, intensive rainfall in the year before the investigation (two day rainfall in the entire river basin including the dam catchment basin of 220mm) deposited unstable sediment in the upper and middle parts of the reservoir, and later this flowed continuously into the reservoir.

The investigation was continued according to this timing. From the following year of 1996 until 1998, topographical measurements of the sediment were performed during the bank full stage (May and June) that is an effective time for sounding in order to clarify minor fluctuations of the sedimentation over a wide area from year to year. Specifically sounding of the reservoir bed (3 times), lateral measurements on land (2 times), and exploration of the bottom sediments with an acoustical sounder (1 time) were done. The reservoir bed topography was measured within the same range as once a year for a total of 3 times. The sounding was done using the Small Investigation Boat Sounding System Combining DGPS and Acoustical Explorer[6] .

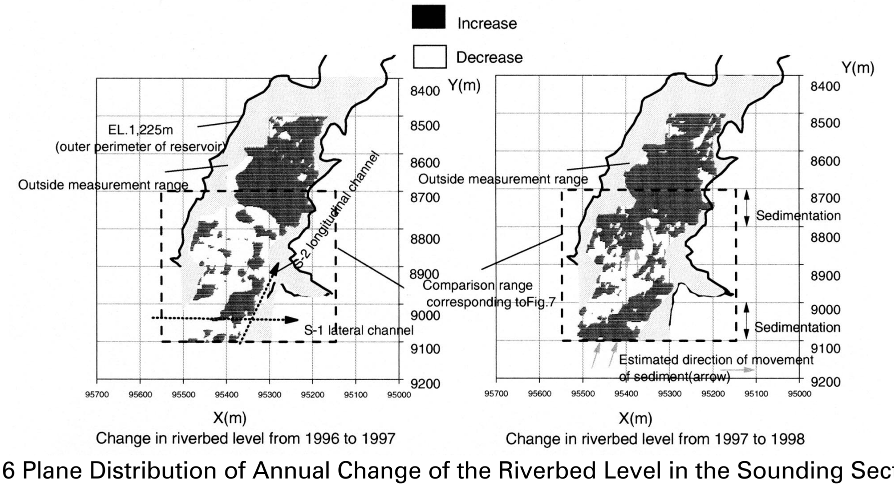


Fig. 6 Plane Distribution of Annual Change of the Riverbed Level in the Sounding Section

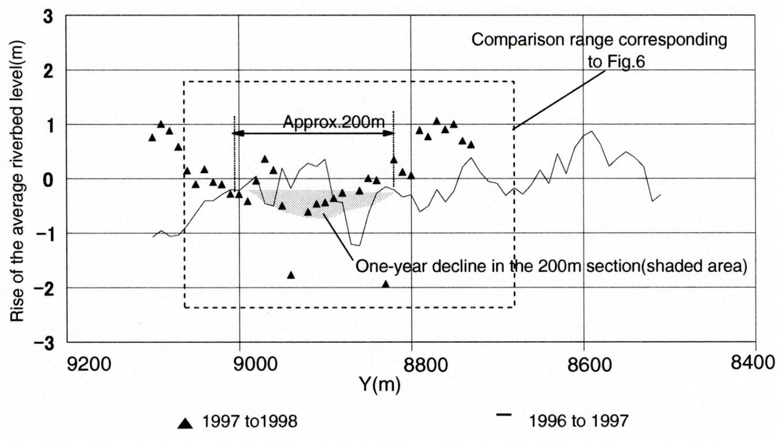


Fig. 7 Annual Change of the Average Riverbed Level of Each Section

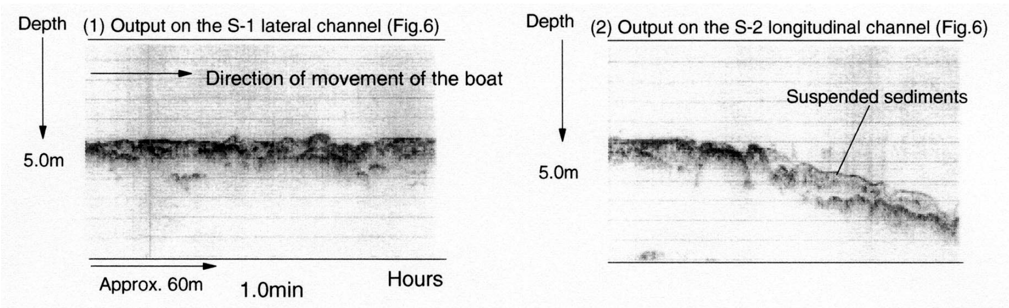


Fig. 8 Output of the Bottom Sediments Exploration

4.1.3 Lateral measurements on land

These were done twice, once a year in 1997 and 1998, on the land where sounding is impossible. The survey section was set at intervals of about 100m to perform lateral measurements using an optical distance measuring device.

4.1.4 Exploration of bottom sediments

This was done once in 1996. An acoustical explorer with output of 5 to 10 kHz (Senbon Denki Co., Ltd., SH-20) and a GPS transceiver were used for exploration in the same range as the reservoir bed sounding.

4.2 Considering the sediment routing

The data obtained by the three year exploration were data for year-to-year change equal to three times at the most but, characteristically, it is spatially dense information.

4.2.1 Direction of movement of the sediment

Fig. 6 shows the results of calculating the change of the riverbed level at points in each mesh based on the results of the three years of sounding. Downstream from Y = 8,700m (upward on the figure) is the range of one-way sedimentation on the reservoir bed throughout the year. So upstream from there, the direction of plane movement of the sediment is considered focussing on the range until Y = 9,100m (part enclosed by the dotted lines in the figure).

The difference between the way the riverbed level changed between 1996 and 1997 and between 1997 and 1998, can be categorized in four patterns: (1) two years continuous sedimentation, (2) change to sedimentation in the second year, (3) change to erosion in the second year, and (4) two years of continuous erosion. Pattern (1) corresponds to the filling range and pattern (4) corresponds to a range where a large erosion channel formed. The area at the site where large scale relatively stable erosion channels are mixed with multiple constantly moving small erosion channels almost corresponds to (3) and (4). It is, therefore, assumed that sediment flowed into areas (1) and (2) (sedimentation range) from areas (3) and (4) (erosion range), so it is possible to draw arrows showing the direction of the movement of sediment as in the right side of Fig. 6.

4.2.2 Quantity of movement of sediment

In order to consider this issue in greater detail, the quantity of movement of sediment is considered. Fig. 7 shows the degree of change each year of the average riverbed level at each lateral riverbed section from Y = 8,500 to 9,100 m. Because the locations of change of the sediment within the part of Fig. 7 corresponding to Fig. 6 (part of the figure enclosed by the broken line) are originally based on the same riverbed fluctuation data, the trend is the same as that shown by the results of considering Fig. 6. It is assumed that the quantity of decline of sediment in the part where the riverbed fell between 1997 and 1998 (approximately 200m section near Y = 8,900 ~ 9,100) is equal to the quantity that moved downstream to calculate the quantity of movement of sediment during a single year. If the area of the shaded part of Fig. 7 that corresponds to the above-mentioned approximately 200m part is assumed to be approximately a triangle, it is 220m X 0.5m X 1/2 = 50m^2. If the lateral width of the riverbed in this part is assumed to be approximately 200m, ultimately, it is possible to estimate that the annual quantity of sediment that moves is the value of approximately 50m^2 X 200m = 10,000m^2.

4.2.3 Considering the results of the bottom sediment exploration

Fig. 8 shows the results obtained from the bottom sediment exploration on the two channels S-1 and S-2 shown in Fig. 6. Near S-1, the area downstream from sediment deposited by the intensive rainfall of 1995 assumed to be the area where great longitudinal and lateral erosion occurred. In the lower layer of the section, a striped pattern thought to be traces of an old channel is observed. It is assumed that after coarse grain materials were transported and deposited, relatively fine grain sediment was deposited on the old channel. And the relative flatness of the riverbed section is also thought to have been caused by continued lateral erosion accompanying the constant change of the channel and by the endless supply of sediment from upstream.

In S-2, it appears that as the riverbed gradient changes abruptly, the quality of the sediment changes. The relationship of these changes in the sedimentation and erosion environments with the existence of points of change on the riverbed and other output from the exploration is a future challenge.

5. Conclusion

Generally speaking, sedimentation problems and turbidity problems are in a contradictory relationship. For example, aggressively discharging turbid water that is one cause of sedimentation is disadvantageous from the standpoint of turbidity, but restricting the increase in the production of sediment is beneficial from the standpoint of sedimentation problems. It is, therefore, assumed that in order to resolve these problems at the drainage basin level in the future, as a general theory, restoring the quantity and quality of the water in a river, the quantity of sediment it transports and other natural conditions in the river to their original states in order to discharge a suitable quantity of sediment at an appropriate concentration is considered. But at dams where there is little danger of the quantity of sediment increasing during the service lifetime of the dam, it is more appropriate to either maintain the reservoir level at a low level without draining ponded water (low water management) or to restrict the production of turbid water by surface discharge. In a case where it is judged to be necessary to discharge turbid water, discharging a specified quantity during a flood is recommended. Reasons for selecting "during a flood" include three facts: it is possible to discharge in a short time, it can bear the role of the dilutant that tends to be short supply, and the discharge does not abruptly change the turbidity of the river water. The problem at this time is setting the appropriate quantity of turbid water to be discharged, but this is a yardstick for the quantity of turbid material retained in the reservoir over years. The quantity of water necessary to discharge sediment from an irrigation dam is ideally and rationally provided from water overflowing the spillway or from discharged ponded water. If this is done, it is possible to safely reduce the factors causing turbid water and sedimentation a little each year without greatly altering the river environment in order to maintain the sediment hydraulic environment of the reservoir and the drainage basin. It is necessary to develop related economical physical technologies, but equipment that is too large considering the required quantity of sediment discharged is inappropriate.

Acknowledgements

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A Study on Sedimentation Processes in Takase Dam Basin

Shigeru Tsuruta
Vice Manager, Civil Engineering Administration Group
Transmission Department, Power Network Division
Tokyo Electric Power Company, Japan

Junji Kobayashi
Manager, Civil Engineering Administration Group
Transmission Department, Power Network Division
Tokyo Electric Power Company, Japan

Yoshihisa Uchita
Construction Engineering Center
Tokyo Electric Power Company, Japan

Abstract

Since sediment management is one of the most important issues for dams in operation, it is essential for considering countermeasures against sedimentation flowing into a reservoir to investigate characteristics of sediment production, transportation and deposition processes in a mountain river basin, which has steep topography and a severe erosion problem.

The study area, Takase Dam basin is surrounded by mountains higher than 2500m in elevation. Characterized by steep topography and weak geology consisting of deteriorated granites, the area is one of the major sediment runoff areas in Japan.

Focusing on the Nigorisawa-Fudosawa basin which is a part of Takase Dam basin and main source of sedimentation in the reservoir, this paper discusses erosion characteristics of slope failure sites, erosion and deposition processes in streams, and the overall situation of sediment balance of the basin, based on aerial photographs, slope erosion data obtained from measuring sticks and seasonal changes of water turbidity in rivers.

Key Words: *Takase dam, mountain river basin, sediment production, transportation, deposition*

1. INTRODUCTION

The Takase dam basin (131 km^2, Fig. 1) is located in the uppermost part of the Takase River in the Shinano River System. It is one of the major sediment runoff areas in Japan, characterized by steep sloped topography and weak geologic configuration.

Takase dam was completed in 1978 as upper dam of the Shintakase Pumped Storage Hydropower Plant with maximum output of 1280 MW. During 1978 to 2004, however, a total of about 16 200 000 m^3 of sediment was deposited in Takase dam reservoir and caused a reduction of about 20% (below High Water Level) of the total storage capacity, i.e. 76 200 000 m^3.

Of the 16 200 000 m^3 sediment, 4 800 000 m^3 came from the Takase River, 8 900 000 m^3 from the Nigorisawa-Fudosawa rivers and the remainder from other streams (Fig. 2). The specific sedimentation rate, which can be obtained by dividing the total volume of sediment deposited by catchment area of 131 km^2 and then by 26 years elapsed, is 4 750 m^3/km^2/year.

This value is far greater than the average value for all mountain areas in Japan, i.e. 300 m^3/km^2/year. It indicates the severity of sediment runoff in the Takase Dam basin. The specific sedimentation rate for the Nigorisawa-Fudosawa basin, which is 12.8 km^2 in area, is quite high of 26 970 m^3/km^2/year.

It is essential to investigate the present state of sediment runoff and prediction of future conditions, in order to take appropriate actions for maintaining the reservoir. Moreover, in a large reservoir, loss of storage capacity due to sedimentation is not the only problem caused by sediment runoff; it also causes the problem of reservoir water turbidity by fine soil particles in large quantities transported during flood. Prolonged turbidity of water downstream is causing serious environmental consequences in Japan.

This paper focuses on the Nigorisawa-Fudosawa basin, an active sediment production area in the Takase Dam basin, and discusses erosion characteristics of slope failure sites, sedimentation and erosion

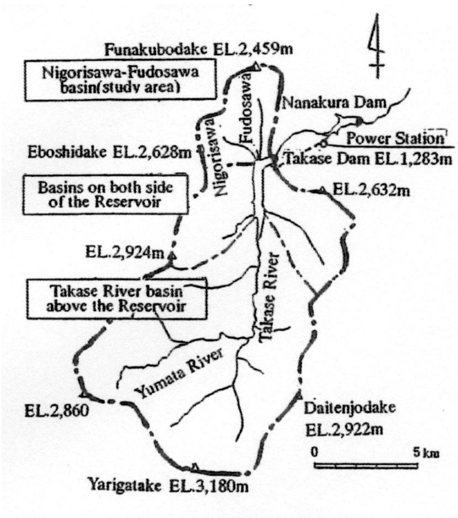

Fig. 1 Takase Dam basin

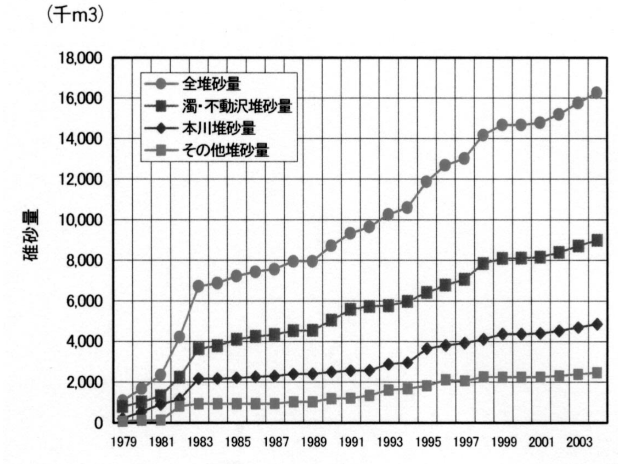

Fig. 2 The volumes of sediment deposited in Takase reservoir

processes in the river system, and the sediment balance of the river basin based on field measurement data.

2. CHARACTERISTICS OF SEDIMENT PRODUCTION

2.1 Slope Failure Sites in the Nigorisawa-Fudosawa Basin

The Takase dam basin is located in a mountainous region and is surrounded by mountains higher than 2800 m in elevation. The boundary of the basin lies above the forest line. The area consists primarily of bare slopes, grasslands, and alpine shrubs. In winter, the entire basin is covered with snow, and snow remains even in summer in some sections of the basin. Average annual precipitation and temperatures at the dam site are around 2100 mm and -16 to 30°C, respectively.

Most of the slopes in the river basin are steeper than 30°. Geologically, more than 90% area of the basin consists of Cretaceous to Paleogene granites. Partially, there are volcanic rocks, alluvial sand and gravel deposits. Slope failure sites, which are a major source of sediment, are highly concentrated in the Yumata River (slope failure area ratio=15%) and the Nigorisawa(25%)-Fudosawa(20%) area, where there are hydrothermally altered and fragile granites.

The distribution of slope failure sites in the Nigorisawa-Fudosawa basin, based on aerial photograph interpretation, is shown in Fig. 3. The rocks at these sites are hydrothermally altered and fractured under the influence of volcanic rock intrusion and have developed fine cracks. The cracks contain clay minerals and the rocks have been weathered to considerable depths. Rainwater infiltration and freeze-and-thaw action in winter have accelerated the weathering of these slopes, causing considerable decreases in strength.

Fig. 4 shows changes in the area of slope failure over the years. As shown in Fig.4, the area of slope

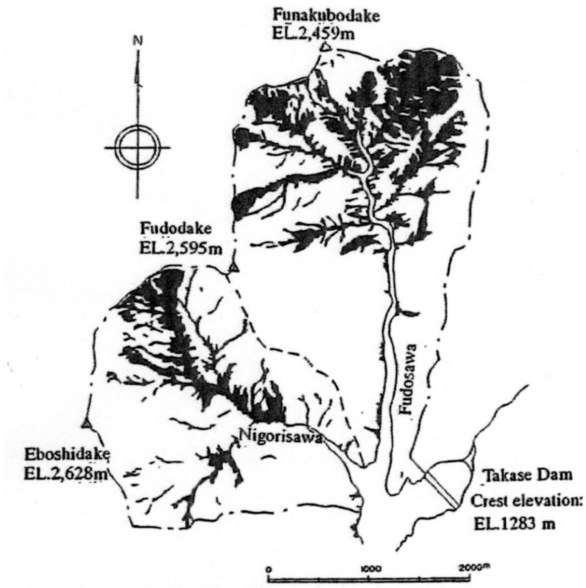

Fig. 3 Distribution of slope failure sites in the Nigorisawa-Fudosawa basin (1983)

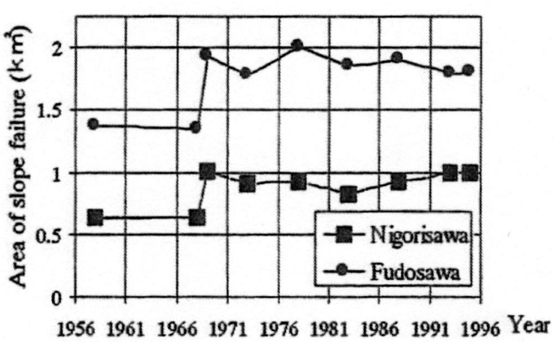

Fig. 4 Changes over the years in the area of slope failure in the Nigorisawa-Fudosawa basin

failure increased due to the expansion of preexisting slope failure areas caused by extremely heavy rain in August 1969. After 1969, aerial photograph interpretation was done at five-year intervals to investigate changes in the area of slope failure. The result indicates that variations in the area of slope failure ranges from only ±1 to 4% per year.

2.2 Characteristics of Erosion of Bare Failed Slopes Based on Measuring Stick Data

While the ratio of new and expanded slope failure areas was as small as 1 to 4% per year, the volume of sediment produced as a consequence of expanded slope erosion was about 17% of the average volume of sediment deposited per year in the reservoir.

In 1984, measuring sticks (steel bars) were installed at three locations on a rock slope of the Fudosawa basin to observe the state of erosion and deposition situations at the bare failed slope. The measurements were carried out from June (rainy season) to October (before snowy season).

The measurement results are shown in Fig. 5 and Fig. 6. The results have shown that there are differences in the rate of erosion or sedimentation along the locations of the measuring sticks on the slope. Characteristics of slope failure can be summarized as follows:

(1) At the upper part of the slope, erosion is in progress (F1-4). The rate of erosion is higher in the winter-snowmelt season (November to the beginning of June) than in the rainy season (the latter part of June to October). The average erosion rates in the rainy season and the winter-snowmelt season in 1986 to 1990 are 1.7 cm/year and 8.6 cm/year, respectively.

(2) At the middle part of the slope, deposition and erosion cycles are occurring. In the winter- snowmelt season, sediment eroded from the upper part of the slope is deposited at the middle part of the slope. Intense rains wash the deposits away to river during early days of the rainy season. Subsequent rains caused erosion and deposition. Maximum depth of erosion up to about 15 cm has occurred in rainy season. The large deposition data (F1-3, F3-1) were obtained from locations subject to concentration of surface water and sediment because of the spoon-shaped topography. The long-term erosion tendency data (F1-1, F2-2) were obtained from hollow-topography areas.

(3) The above mentioned results indicate the following: Deterioration of the surface layer caused by freeze-and-thaw cycles, rainwater infiltration and others are in progress on a bare slope consisting of softened materials. Factors such as rains, snow movement and separation by self-weight during the snowmelt season cause erosion of the upper part of the slope. The eroded sediment moves down slope and is deposited. Intense rainfalls in and after June, when snow on the ground in the ravine area below the slope disappears, cause further down slope movement of sediment deposits on the slope and erosion of the slope. As a result, the eroded sediment mows down onto the riverbed.

3. CHARACTERISTICS OF TRANSPORTATION OF SEDIMENT ON MOUNTAIN RIVER BASIN

3.1 Deposition and Erosion of Sediment in River Basin

Changes in elevations of the slope failure sites and the river beds in the Nigorisawa-Fudosawa basin were examined by using aerial photographs. The 1:15 000-scale aerial photographs were taken in October 1983, 1993, and 1995. The 1983 photographs were taken immediately after a large amount of sediment flowed into the reservoir (Fig. 7). The 1993 photographs were taken after two relatively dry years when the amount of sediment that flowed into the reservoir was small. The 1995 photographs were taken immediately after the intense rainfall (Fig. 7).

Fig. 8 shows the deposition and erosion patterns during two periods, one before 1993 and the other after 1993. Fig. 9 shows changes in the elevation of the beds of the two rivers plotted on a longitudinal profile of the riverbed

From the obtained results, characteristics of the changes of the failed slopes and the riverbed in the Nigorisawa-Fudosawa basin can be summarized as follows:

(1) Erosion occurred on almost all failure slopes throughout the observation period. Temporary deposi-

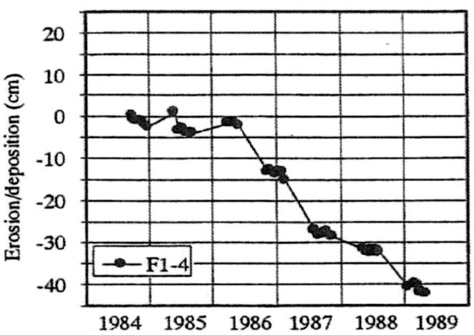

Fig. 5 Results of measuring stick measurements carried out in the upper part of the failed slope

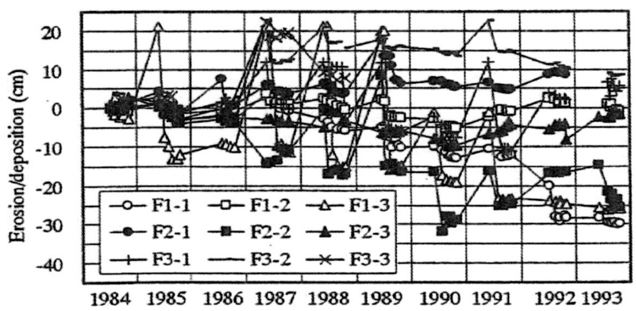

Fig. 6 Results of measuring stick measurements carried out at the middle part of the failed slope

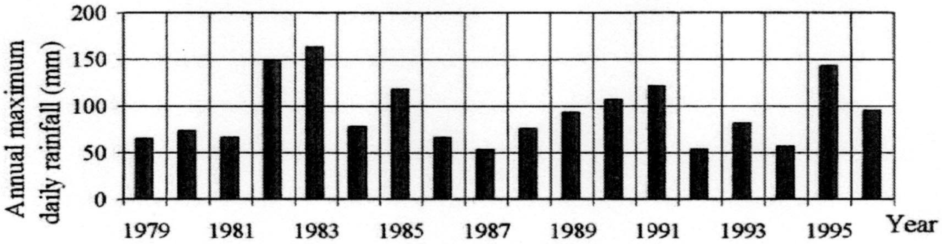

Fig.7 Changes over the years of annual maximum daily rainfall at Eboshidake station

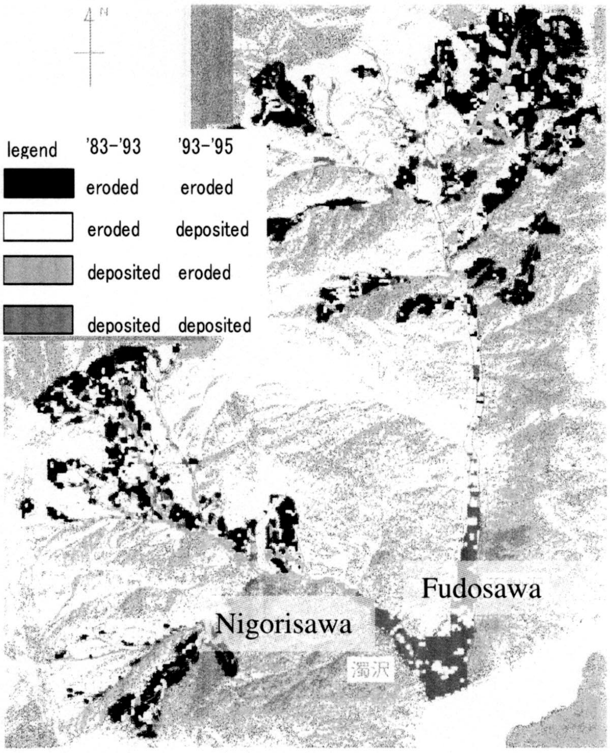

Fig. 8 Deposition and erosion patterns (1983-1993-1995)

tion of sediment occurred at the places subjected to concentration of eroded sediment, such as hollow-shaped areas and locations on gentle slopes where the slope angle changes.

(2) During the period from 1983 to 1993, the bed of the Nigorisawa river in an upstream section (distance from the reservoir; TD more than 1,000 m) and the bed of the Fudosawa river in an upstream section (TD more than 3800 m) rose considerably. Later, the trend was reversed and erosion became predominant and resulted in wash-outs of a major part of the deposits. Since all of these river sections adjoining large-scale failure sites, the supply of sediment to the riverbed is prominent. In addition, usual discharges in rivers over these sections are low because of the smaller catchment area. The bed slopes of rivers are greater than 15°. The riverbeds are narrow, and the duration of sunshine is short. According to field observation results, sediment is dammed by huge boulders and the river beds are covered with snow and ice. Heavy rainfall triggers runoff of deposited sediments together with ice and snow as debris flow. The volumes of sediment deposited in the above-mentioned sections of the Nigorisawa and the Fudosawa rivers during 1983 to 1993 were about 220 000 m^3 and 60 000 m^3, respectively. The volumes of sediment eroded during the period from 1993 to 1995 were about 260 000 m^3 and 100 000 m^3, respectively (Table 1).

(3) On the bed of the Fudosawa river from TD3900 m to 1700 (bed slope: 7 to 15°), riverbed gradation or degradation occurred near the junction with the tributary and near the locations where bed slope or river width changes. Throughout two observation periods, however, the magnitude of change in elevation was not very large.

(4) The Nigorisawa river section from TD1000 m downstream and the Fudosawa river section from TD1700 m down to the reservoir (bed slope: 2 to 7°) form a compound alluvial fan. In these sections, deposition remained predominant throughout the two observation periods.

Changes in the pattern of deposition at the entrance to the reservoir are shown in Fig. 10.

Results indicate that the riverbeds upstream of the reservoir have a tendency to rise under the influence of the reservoir.

Table 1 compares the volumes of sediment inflow into the Takase reservoir obtained from the sediment balance of the Nigorisawa-Fudosawa basin based on aerial photogrammetry with the measurement results

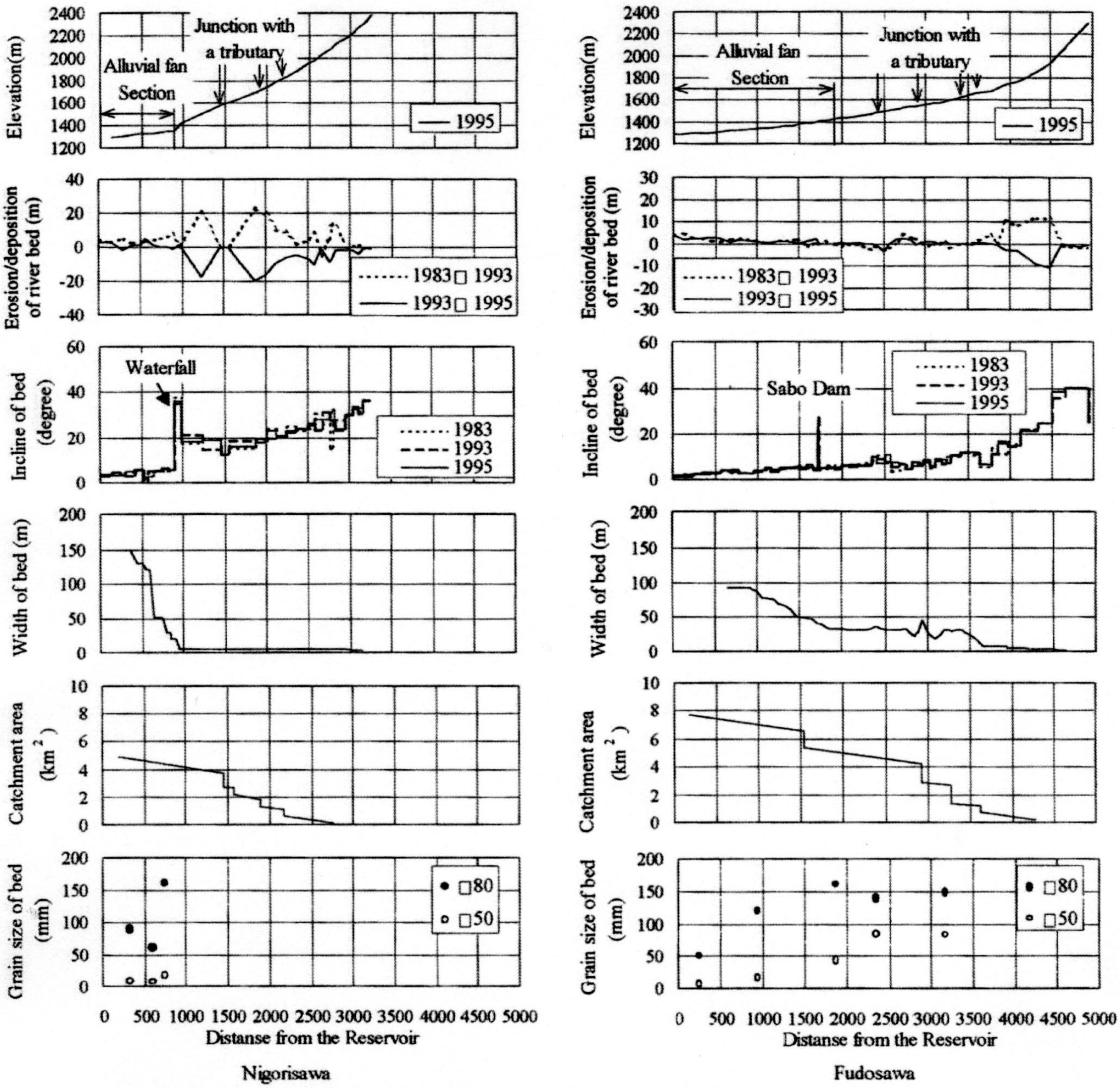

Fig. 9 River bed erosion/deposition profiles

Table 1 Sediment balance of the Nigorisawa-Fudosawa basin based on aerial photogrammetry

【$\times 10^3 m^3$ +Deposition -Erosion】

Name of Stream	Location	1983~1993	1993~1995	1983~1995	Remarks
Nigorisawa	Failed slope	-1,143	-464	-1,561	Values obtained by multiplying with the volume expansion factor of 1.3
	River bed($\theta \geqq 15^\circ$)	222	-258	-36	
	River bed($\theta < 15^\circ$)	280	289	568	Mostly in alluvial fan areas of $\theta < 7^\circ$
	Subtotal	-641	-433	-1,029	
Fudousawa	Failed slope	-1,563	-853	-2,303	Values obtained by multiplying with the volume expansion factor of 1.3
	River bed($\theta \geqq 15^\circ$)	64	-104	-33	
	River bed($\theta < 15^\circ$)	147	439	586	Mostly in alluvial fan areas of $\theta < 7^\circ$
	Subtotal	-1,352	-518	-1,750	
Total		-1,993	-951	-2,779	Corresponding to sediment inflow into the reservoir (below HWL)
Measured sediment inflow into the reservoir (below HWL)		2,176	651	2,827	

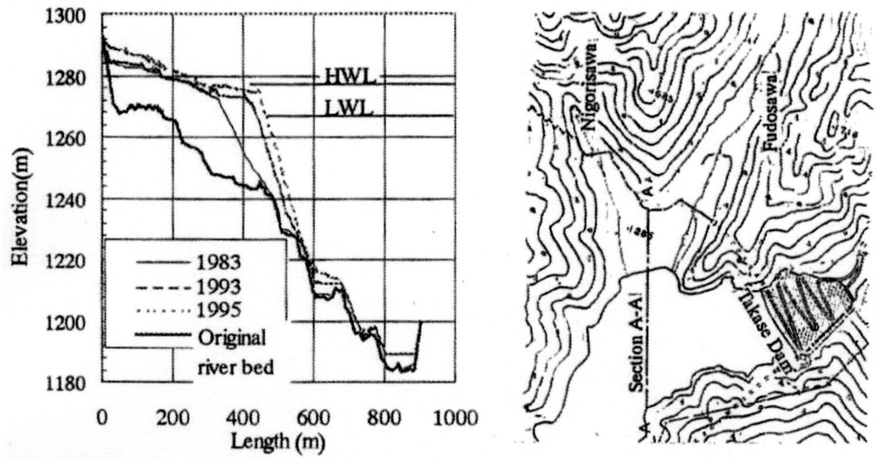

Fig.10 Changes in pattern of deposition at entrance to reservoir (section A-A)

Table 2 Average annual erosion depths at slope failure sites based on aerial photogrammetry

cm/year

Name of Stream	1983～1993	1993～1995	1983～1995
Nigorisawa	8.8	17.8	10.0
Fudosawa	6.7	18.2	8.2
Average	7.4	18.1	8.8
	(6.4)	(13.1)	(7.6)

* The numbers in parentheses were calculated from reservoir sedimentation

obtained in the reservoir. As shown, the two sets of values show fair agreement.

Most of the sources of sediment are the failure slopes and sediment yield varies depending on rainfall history. Part of the sediment eroded from the slope is retained temporarily on the slope or in the upstream river sections, and is washed away during intense rains. During 1983 to 1995, most of the sediment washed away from the slope was deposited in the reservoir or on the alluvial fan at the entrance to the reservoir.

The amounts of erosion (Table 1) on the failure slopes were determined by aerial photogrammetry for the period of 1983 to 1995. The results show an erosion rate of 8.8 cm/year. This value is slightly greater than the value of 7.6 cm/year calculated from the volume of sediment deposited in the reservoir. (Table 2) This difference is close to the volume of sediment deposited on the riverbed in the alluvial fan and other areas (about 670 000 m^3).

3.2 Seasonal Changes in Sediment Transport

In order to investigate the seasonal changes in sediment inflow into the reservoir, seasonal changes in the turbidity of river in the Nigorisawa and the Fudosawa were examined.

The results show the relationship between turbidity of river water and discharges (Fig. 11). However, turbidity of water tends to increase in and after June in cases where snow coverage is lower than 20%. The average change in the snow coverage ratio in the river basin is shown in Fig. 12. Upstream main source of sediment are covered almost completely at the beginning of June.

Fig. 13 shows average changes in the elevation of the riverbed in the alluvial fan section of the Fudosawa during the period from December 1997 to August 1998.

According to Fig.13, during the winter-snowmelt season from December to June, the riverbed became lower, because of scouring due to supply of less sediment loaded flow from upstream. Later, in the summer month of July, the trend was reversed and the riverbed began to rise.

These facts indicate the following: Sediment runoff is closely related to the snow conditions on the sediment production areas along the upper reaches in the basin and the runoff of deposits and slope sediment is not severe under snow cover. As the snow melts under influence of air temperatures and precipitation, sediment begins to flow into the river and results in deposition of large volumes of sediment in the reservoir.

4. CONCLUSIONS

Sediment production and transport processes in the Nigorisawa-Fudosawa basin, a steep and high-sediment-yielding mountain basin that flows into the Takase reservoir, were investigated. The conclusions drawn from the study are summarized as follows:

(1) The Nigorisawa-Fudosawa basin comprises 30-degree-or-steeper slopes made up of granite. Slope failure sites on the basin are concentrated in areas where rocks are affected by hydrothermal alteration.

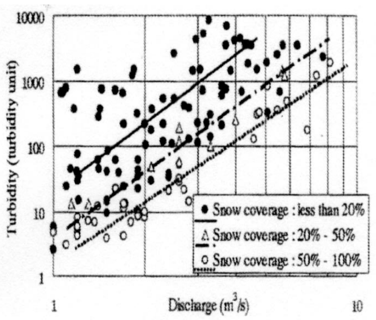

Fig. 11 Relationship between river discharge and turbidity

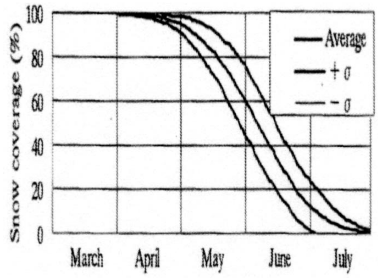

Fig. 12 Monthly changes in the snow coverage in the river basin

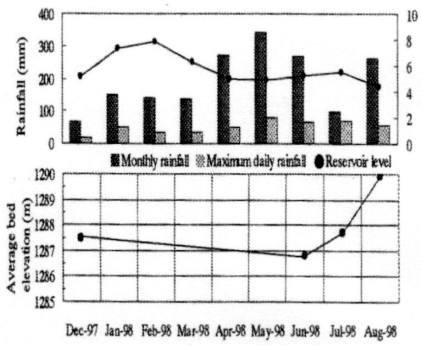

Fig. 13 Seasonal changes in river bed elevation in Fudosawa alluvial fan

The area of slope failure portions of the basin increases or decreases by about 1 to 4%. Thus, changes in the area of slope failure are small. During very intense and long-lasting rainfall, however, the area of slope failure increases considerably.

(2) In existing slope failure areas, erosion occurs during both winter-snowmelt season and rainy season. The upper part of the slope is dominated by erosion mainly due to freeze-and-thaw action. Sediment is deposited mainly on the middle part of the slope. In the latter season, when snow on the ground disappears, intense rains trigger sediment runoff and cause erosion.The sediment is transported and deposited on the river bed and gently sloping areas.

(3) In steep-sloped (more than 15°) ravines where failure slopes are concentrated, large-scale deposition occurs. Possible causes of such deposition include the yielding of large volumes of sediment, low river discharges, narrow river channel that can be easily dammed by huge boulders, a short duration of sunshine and ice and snow remaining on the river bed even in summer. Most of the sediment deposited on the lower part of the slope and on the river banks is washed away downriver during intense rains.

(4) Changes in riverbed elevation on the middle part of the basin, where the bed slope ranges from 7 to 15°, are small. The alluvial fan downstream, which has a slope of less than 7°, tends to undergo deposition under the influence of the reservoir. As sediment runoff is directly related to the snow conditions on the sediment production area along the upper reaches, the riverbed become lower during the winter-snowmelt season.

(5) The sediment balance obtained from aerial photogrammetry of the slope failure site and in the riverbed shows fair agreement with the measured volume of sedimentation in the reservoir.

Sediment production sources are mostly located in the failure slope and the average annual depth of erosion of the failure slope was found to be about 8.8 cm/year from the past 12 years' data. Sediment eroded from the failure slope surface was retained temporarily on the middle and lower parts of the slope or on banks. During 1983 to 1995, most of the sediment eroded from the slope was deposited in the reservoir or on the alluvial fan.

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Research on Sedimentation of Tingzikou Reservoir on the Jialingjiang River

Wan Jianrong Lu Jinyou Gong Ping
Changjiang River Scientific Research Institute, Wuhan, China

Abstract

Based on 1D imbalance sediment transport theory, Csediment deposition and erosion in Tingzikou reservoir on the Jialingjiang River was analysed by calculation. Results show that after 100 years' operation the sediment deposition amount in the reservoir would come to 21.808 $10^8 m^3$. And after 20 years' operation, due to the impact of sediment deposition, the water levels on the river section between 41.9 km below Zhaohua and 10 km above Zhaohua would rise 1.05~2.53 m when floods of different frequencies occur.
Key words: *Freservoir sediment; mathematical model; sediment calculation*

1. Preface

Jialingjiang River is the largest left bank tributary of the upper Changjiang River. The catchment area is 159,800km^2 and the length of main channel is 1,120km. The upper reach of Jialingjiang River above Guangyuan (380km long) is a mountain river, the upper part of the middle reach between Guangyuan to Cangxi (175km long) is a hill-stream, the lower part of the middle reach between Cangxi to Hechuan (470km long) is a wide valley (400~2000m wide) and the lower reach below Hechuan (95km long) is an alpine gorge.

Jialingjiang River has abundant water, at the same time, it is one of the most important sand producing rivers of the upper Changjiang River and its sediment is the main source of Three Gorges Reservoir. Beibei hydrometric station, the control station of Jialingjiang River, controls 156142 km^2, occuping 97.7% of the catchment area. The mean annual runoff, the mean annual amount of suspended load and pebble bed load of Beibei station are 65.5 billion m^3, 12000 10^4t and 7 10^4t respectively. The runoff of Beibei station amounts to 19% of that at Cuntan hydrometric station and 15% of that at Yichang hydrometrci station on the main stem Changjiang River and the amount of suspended load takes up 27% and 24% of that at both stations respectively.

The Tingzikou Water Project, known a key control project on Jialingjiang River, lies on the middle canyon stretch between Guangyuan and Cangxi. The dam site, 15 km above Cangxi and 147 km below Guangyuan, controls a catchment area of 62550 km^2. The reservoir area contains a main stem of the river from GuangYuan to dam site and tributary estuary sections of Bailongjiang River and Qingshuihe River. The average bed slope from Guangyuan to Zhaohua (about 26 km long) is about 0.8%, from Zhaohua to Daqinggou (about 36km long) is 0.67% and from Daqinggou to dam site (about 97km long) is 0.67%. Figure 1 is the plane schematic diagram of Tingzikou Reservoir area.

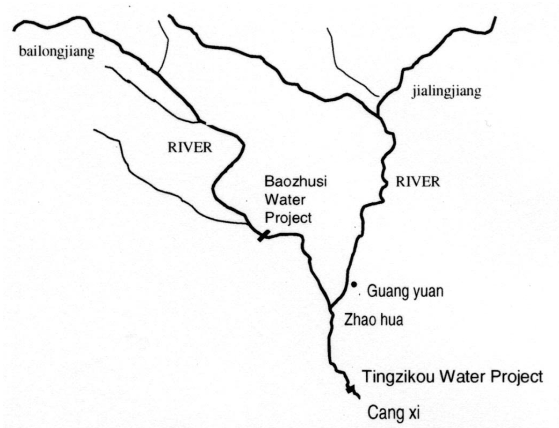

Fig. 1.The plane schematic diagram of Tingzikou Reservoir area

Tingzikou hydrometric station is the representative station of the Tingzikou Water Project. The mean annual runoff is 203 $10^8 m^3$, the mean annual sediment load is 6100 104t and the mean annual bed load is 14.9 10^4t.

In this paper researched is the sediment deposition in the reservoir through 1D mathematical model of total load.

2. Conditions and Schemes of Calculation

2.1 Conditions

Conditions of Hydrology and Sediment

Through the statistical analysis of long-series hydrological and sediment data of Xindianzi hydrometric station, the primary control station of the main stem above dam site, hydrological series of 10 years (1974~1983) were selected as the typical series. The mean annual water volume and suspended load amount of the typical series are 64.4 $10^8 m^3$ and 3490 10^4t respectively, very close to the long-time average annual value of 64.33 $10^8 m^3$ and 366010^{4}t.

Time Interval and River Reach of Calculation

One year was divided into 60 time intervals, the duration of which is short in flood season and long in low flwo period, and then ten years were divided into

600 time intervals in all.

The topographical map used in calculation was surveyed in 1992. Thirty-two cross sections were arranged from dam site to Guangyuan (163km long) on the main stem and six cross sections 17.8 km upstream from the junction of Bailongjiang River were arranged.

2.2 Schemes

This calculation scheme considered normal pool level of 458m, flood control limit level of 458m and dead water level of 438m.

3. Mathematical Model

Calculation used software "HELIU-2", developed by Changjiang River Scientific Research Institute based on 1D imbalance sediment transport theory. The following is the simplified basic equation used in "HELIU-2".

(1) water surface profile formula

$$Z = z_0 + \frac{n^2 Q^2 \Delta x}{2}\left(\frac{B^{4/3}}{A^{10/3}} + \frac{B_0^{4/3}}{A_0^{10/3}}\right) + \frac{U_0^2 - U^2}{2g}$$

(2) suspended sediment concentration variety equation

$$S_i = S_{*i} + (S_{oi} - S_{*oi})e^{-Y} + (S_{*oi} - S_{*i})Y^{-1}(1 - e^{-Y}) \qquad (i = 1,2,\Lambda\ ,8)$$

where,

$$Y = \frac{\alpha \omega_i \Delta x}{q}$$

$$S_{*i} = K_i S_{*m}$$

$$S_{*m} = k\left(\frac{U^3}{gh\omega_m}\right)^m$$

if m=0.92, k/g^m=0.0175, then

$$S_{*m} = 0.0175\frac{Q^{2.76}B^{0.92}}{A^{3.68}\omega_m^{\ 0.92}}$$

$$\omega_m^{\ 0.92} = \sum_{i=1}^{8} P_i \omega_i^{\ 0.92}$$

K_i, grouped sediment-carrying capacity of flow (Douguoren formula);

$$K_i = \frac{(P_i/\omega_i)^\beta}{\sum_{i=1}^{8}(P_i/\omega_i)^\beta}$$

P_i, suspended load gradation

$$P_i = \begin{cases} P_{oi} & \textit{balance} \\ \dfrac{G_{soi} - \Delta G_{si}}{\sum (G_{soi} - \Delta G_{si})} & \textit{imbalance} \end{cases}$$

(3) river-bed variation caused by suspended load

$$\Delta Z_1 = \sum_{i=1}^{8} \frac{(Q_0 S_{0i} - QS_i)\Delta t}{\gamma'_{si} B \Delta x}$$

(4) bed load discharge

Suspended sediment discharge was obtained based on empirical curve of suspended sediment discharge presented by Changjiang River Scientific Research Institute. The relationship of the empirical curve is

$$\frac{V_d}{\sqrt{gd}} \sim \frac{q_s}{d\sqrt{gd}}$$

(4)

where,

$$V_d = \frac{m+1}{m} / \left(\frac{h}{d}\right)^{-\frac{1}{m}} U$$

$$m = 4.7\left(\frac{h}{d_{50}}\right)^{0.06}$$

(5) competent velocity formula (Zhangruijin formula)

$$U_c = \left(\frac{h}{d}\right)^{0.14}\sqrt{17.6\frac{\rho_s - \rho}{\rho}d + 0.000000605\frac{10+h}{d^{0.72}}}$$

(5)

(6) river-bed variation caused by bed load

$$\Delta Z_2 = \sum_{i=9}^{16} \frac{(G_{boi} - G_{bi})\Delta t}{\gamma'_{si} B \Delta x}$$

where,

Δt, time interval;

Δx distance between two cross sections;

S_i, $S*_i$, grouped sediment concentration and sediment-carrying capacity of flow;

S_*m, total sediment-carrying capacity at a cross-section;

Q, discharge per unit width;

ω_m, mean settling velocity of heterogeneous sediment;

k, m, coefficient and exponent of sediment-carrying capacity;

β, exponent (1/6);

U_d, flow velocity near bed;

Uc , competent velocity of bed material;

D, grain diameter;

H, water depth;

Qb, sediment discharge of bed load per unit width;

Gb, total sediment discharge of bed load;

Gs, suspended sediment discharge at a cross-section;

subscript "0", known cross sections.

The above software "HELIU-2" was used to calculate sediment deposition of Three Gorges Reservoir, Danjiangkou Reservoir etc and the results were all accepted by experts.

4. Analysis on Sediment Deposition Laws of Reservoir

4.1 Sediment Storage Calculation and Analysis of Baozhusi Reservoir

There is a tributary, Bailongjiang River, joining in the backwater fluctuation zone of Tingzikou Reservoir. Baozhusi Reservoir with storage of 25.5 10^8m^3 is a large reservoir on the upper Bailongjiang River. The water and sediment storage of Baozhusi Reservoir obviously influenced the inflow and sediment hydrograph of Tingzikou Reservoir, so silt storage calculation of Baozhusi Reservoir is needed.

Table 1 Results of Sediment Storage calculations of Baozhusi Reservoir

operation term (year)	reservoir sediment deposition amount (10^8m^3)	sediment flushing rate (%)	Flushing rate of Sand size D<0.01mm (%)
10	1.62	6.2	98
20	3.19	9.4	93
30	4.69	12.8	85
40	6.12	17.2	74
50	7.49	21.1	66
60	8.76	26.2	58
70	9.95	31.2	52
80	11.05	36.4	46
90	12.05	42.0	41
100	12.95	47.9	37
110	13.75	53.9	33
120	14.45	59.4	31
130	15.09	63.2	28
140	15.57	71.8	26
150	16.05	72.5	25.4

(1) Sediment Deposition Amount of Baozhusi Reservoir

Baozhusi Reservoir has a big storage and much sediment, Cso Sediment deposition amount is abundant accordingly. Calculated results show that after 50 years' operation of Baozhu Reservoir, the amount of sediment deposition will be 7.49 10^8m^3 and after 100 years' operation will come to 1.295 billion m^3. Details were shown in Table 1.

(2) Analysis on Sediment Deposition of Tingzikou Reservoir Influenced by the Sediment Storage of Baozhusi Reservoir

The control area and inflow of Baozhusi Reservoir are approximately 50% of Tingzikou Reservoir and mean annual sediment discharge is about 30% of Tingzikou Reservoir. During the earlier operation of Tingzikou Reservoir, sediment flushing rate of Baozhusi Reservoir is small that markedly reduces the sediment inflow and deposition amount of Tingzikou Reservoir. But as the runtime of Baozhusi Reservoir and Tingzikou Reservoir increase, the sediment outflow of Baozhusi Reservoir will increase, the sediment deposition in Tingzikou Reservoir will slowly attenuate and the deposition course will prolong.

4.2 Calculated Results on Sediment Deposition of Tingzikou Reservoir

After construction of Tingzikou Reservoir, the water depth and velocity of the reservoir will deepen and slow, and the most sediment will silt in the reservoir. Calculated results of Sediment (including bed load) deposition in reservoir were shown in Table 2.

Tabble 2. Results of Tingzikou Reservoir sediment deposition calculation

operation term (year)	deposition amount: from dam site to Zhaohua (10^8m^3)	deposition amount: above Zhaohua (10^8m^3)	reservoir sediment deposition amount (10^8m^3)	sediment flushing rate (%)	preserved storage: flood control storage (%)	preserved storage: regulating storage (%)
10	2.440	0.125	2.565	26.4	96.7	94.3
20	4.949	0.131	5.080	28.8	94.9	89.6
30	7.347	0.143	7.490	32.0	92.8	85.0
40	9.730	0.154	9.884	35.1	90.7	81.0
50	12.043	0.203	12.246	38.5	88.2	76.3
60	14.241	0.242	14.483	42.9	85.9	72.4
70	16.333	0.261	16.594	47.1	83.3	68.1
80	18.206	0.273	18.479	53.9	80.9	65.1
90	20.008	0.295	20.313	56.6	78.1	61.5
100	21.502	0.306	21.808	65.0	75.5	58.3

4.2.1 Reservoir Sediment Deposition Amount, Distribution and Pattern

(1) Amount of sediment deposition

When Tingzikou reservoir have operated 20 years, 50 years and 100 years, Cthe sediment deposition amount of the reservoir area will come to 5.080 10^8m^3,12.246 10^8m^3, 21.808 10^8m^3, respectively.

Sediment deposition velocity of Tingzikou Reservoir is relative even. During the earlier 30 years of operation, Csediment silt quickly, Clater deposition velocity will slowly attenuate and the deposition course will prolong, and the term of reservoir sedimentation balance will exceed 100 years.

(2) Sedimentation Distribution

A) Sediment Deposition Amount in Perennial Backwater Zone

Because of deep water depth and slow flow velocity, inflowing reservoir sediment mostly silted in the perennial backwater zone. At the same time, because of the low stage before dam and large reservoir inflow in flood period, sediment-carrying capacity of flow is strong, and then sediment silted in reservoir head and that comes from upstream were also brought to this zone. After 20 years' operation of the reservoir, the amount of sediment deposition will be 4.949 10^8m^3 and after 100 years the amount will come to 21.502 10^8m^3. Based on the analysis of calculated results, the amount of sediment deposition in this zone is about 98% of the total amount of the reservoir area.

B) Sediment Deposition Amount in Fluctuating Backwater Zone

Since backwater just has a little effect on fluctuant backwater zone, the flow velocity is swift and the sediment transport capacity of flow is strong, sediment deposition in the zone is lesser. After 20 years' operation of the reservoir, the amount of sediment deposition will be 0.131 10^8m^3 and after 100 years' operation the amount will come to 0.306 10^8m^3. The amount of sediment deposition in this zone is about 2% of the total amount of the reservoir area.

(3) Sediment Deposition Pattern

The deposition pattern in Tingzikou reservoir is of a delta type. With the runtime of reservoir increasing, the delta deposition is pushed to dam and stretch upwards, the surface of delta is also driving up. The distance between vertex of delta deposition and dam is about 80 km after 20 years, and it is about 60km after 50 years. After 80 years, the speed of sediment deposition will slow down and the monomial sediment deposition pattern is replaced by an alternate deposition and erosion process, so the sediment deposition will arrive at a balance stage. The longitudinal section view after 20 years is shown in Figure 2.

4.2.2 Variation of reservoir sediment flushing rate

There is more Sediment deposition at the initial stages of reservoir operation, but small sediment flushing rate. The sediment flushing rate is 28.8% after

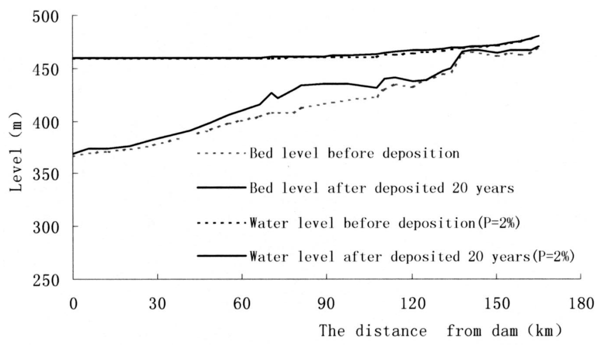

Fig. 2. The longitudinal section view after 20 years

reservoir operated 20 years. With the runtime of reservoir increasing, the volume of sediment deposition is increasing, the sediment-carrying capacity of flow is increasing and the sediment flushing rate is increasing too. After reservoir operated 50 years, the sediment flushing rate is 38.5%, and after 100 years, it is 65.0%.

4.2.3 Reserved storage after sediment deposition

After construction of Tingzikou reservoir, the reserved storage is reducing because of sediment deposition. The calculation indicated that the reserved storage will maintain 95% and regulating storage will maintain 90% after reservoir operated 20 years. After 50 years the reserved storage will be 88% and regulating storage will be 76%. After 100 years reserved storage will remain 75% and regulating storage will remain 60%.

4.2.4 Analysis of impact of reservoir sediment deposition on flood level

Based on the calculation of sediment deposition, this article calculated the backwater area at the 5% and 20% frequency floods after reservoir operated 20 years. The results of calculation indicate that the flood levels with different frequencies are higher than that before sediment deposition. The water levels are especially higher on the river section between about 41.9km below Zhaohua and 10 km above Zhaohua. The comparison of flood levels after filling and before filling is as follows.

The flood level on the river section about 41.9km below Zhaohua will rise 1.54m to 2.53m when the 5% frequency flood occurs. When the flood frequency is 20%, it will rise 1.05m to 2.40m. When the flood frequency is 5% and 20%, the levels at Zhaohua are 469.37m and 466.25m respectively, rising 2.24m and 2.17m compared to the levels before sediment deposition.

The water level of the main stem above Zhaohua with the same flood frequency rises about 1m to 2m compared with the level before sediment deposition when the reservoir operated 20 years.

After reservoir operated 20 years, the upper end of the backwater area of the main stem is still located between cross-sections 26 and 27 below Guangyuan, and the flood level on rive section near Guangyuan is the same as that before silting. The backwater has little effect on river section near Guangyuan.

5. Conclusions

Tingzikou reservoir located in a high sand load river, the reservoir sand inflow is abundant. The amount of sediment deposition after reservoir operated 20 years, 50 years and 100 years is 5.08 10^8m^3, 12.46 10^8m^3 and 21.88 10^8m^3 respectively. Sediment deposition has a small effect on the comprehensive benefit of Tingzikou project from a long time point of view.

The sediment deposition of Tingzikou Reservoir mainly stays in perennial backwater area and the deposition amount in this area takes up 98 % of the total deposition amount in the reservoir area.

The sediment deposition configuration is in the shape of triangle. The distance between vertex of sediment deposition triangle and dam is about 80km after the reservoir operated 20 years, and it is about 60km after 50 years.

Sediment deposition of Tingzikou Reservoir has no effect on Guangyuan and Bailongjiang Bridge. The flood level of the old town area of Zhaohua will rise about 2m compared to that before silting at the same flood frequency.

Model Test on Sediment of Right Bank Underground Power Station in TGP

Li Dazhi Guo Wei Chen Yiwu
Changjiang River Scientific Research Institute, Wuhan, China

Abstract

The right bank underground power station of the Three Gorges Project (TGP) is far away from the main channel, so the water is diverted from branch channel for power station, after impoundment of reservoir, a slack water area and backflow area occurs in front of power station. In order to study the problems of sediment accumulation, the method of combination of mathematic model and material model has been used to carry out the test on sediment flushing effect of sediment flushing outlet and engineering measures for improving underground power station.

Keywords: *underground power station of TGP; material undistorted model; sediment accumulation test of serial year; sediment flushing effect*

1. Preface

The underground power station of the Three Gorges Project (TGP) located under the ridge of Baiyanjian on the right bank and adjacent to the right bank power station is a part of the TGP power station, with the installation of 6 sets and the total installed capacity of 4200000kW. The gate opening of power plant intake is 9.5m×15m, with bottom elevation of 116m. After impoundment of TGP, the front of right bank underground power station is in the slack water area and backflow area, and sediment accumulation happens. With long-term operation of the TGP, the accumulation layer thickens and the flow pattern becomes complicated, which will threaten the long-term safe operation of underground power station. Therefore, during the period of design and construction of TGP, the sedimentation test of long serial-year (70+6 years) has been carried out for many times with 1/150 undistorted model[1] to research the sediment flushing effect and the engineering measures of communicating canal between the right bank power station and underground power station to improve power station inflow. The simulating river reach is 31.5km long with 17.5km long river reach upstream of dam site.

2. Test conditions[2]

(1) For the hydrographs of flow discharge and sediment concentration, the mathematic model calculation results of hydrologic serial year of 1961～1970 is used. However, the calculation results do not consider the reservoirs newly built on the main stem and tributaries in upper reach of TGP reservoir and the sediment reducing effect of water and soil conservation project, so the silting amount is larger.

(2) Operating schemes of water level in front of dam and structures include: the limiting water level of dam area in flood season is 135m from the 1st to the 10th year, and 145m from the 11th year; the water level in the downstream of dam is controlled at 66m according the level in front of dam in Gezhouba Project.

(3) Considering that the water level in front of dam will be controlled above 145m for a long time in high flow and heavily silt-carrying years like 1954, in order to research the influence of high flow year on sediment scouring & silting of dam area, the typical hydrologic years 1954 and 1955 were inserted at the end of 30 years, 50 years and 70 years of the TGP operation, namely, the high flow year like 1954 will occur for 3 time in 76 years when TGP operating. In fact, the flood of 1954 is 80-year-frequency flood at Yichang station, with the maximum sediment concentration measured since 1950, therefore, the serial years used in the test are on the safe side.

(4) Considering that the underground powerhouse is arranged in depositional area of original Maopingxi, in order to guarantee the normal operation of underground powerhouse without influence of sedimentation, the priority is given to underground powerhouse in operation in principle, namely, 6 units will operate when flow discharge is above 20000m^3/s in flood season, and at least 2 units in low-flow season.

3. Flow & sediment movement in front of underground power station

(1) After operation of TGP, the upstream river regime will become slightly sinuous and smooth gradually (Fig. 1). The main flow just faces the overflow section in front of dam, and two backflows with opposite direction occurs in front of left and right power station section; while the underground power station located in Maopingxi at the right bank is separated

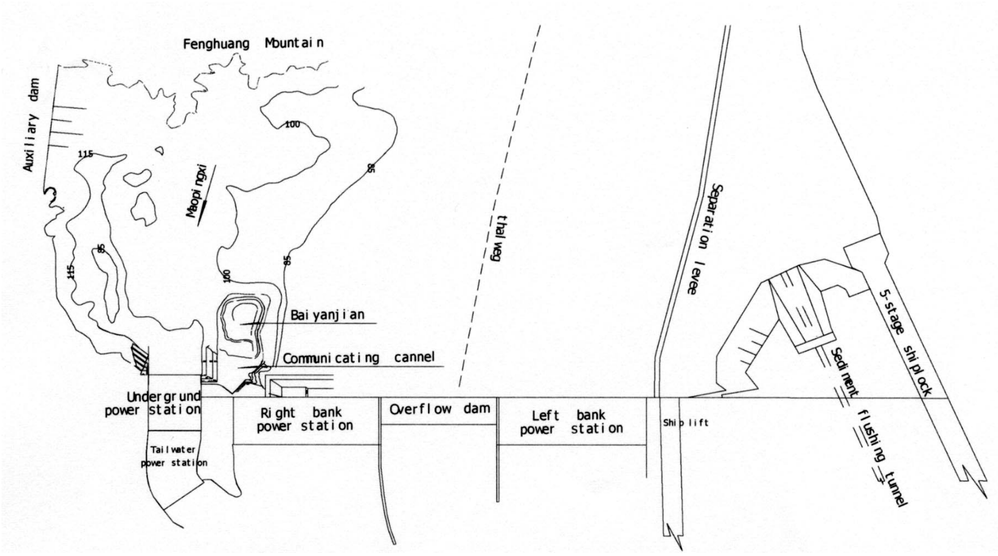

Fig. 1. Layout oer Connection between underground power station and right bank power station

from main channel by Baiyunjian mountain, when power station running, part of water flows into underground power station from the mouth of Maopingxi, namely diversion from branch channel, and another part of water flows into underground power station from communicating cannel.

(2) The method of "storing clear water and discharging muddy water" is used in operation of TGP, the water level in front of dam is controlled at 145m in flood season generally. A great quantity of sediment is discharged from deep outlet of overflow section to the downstream of dam with flood, while underground power station is far away from the main channel, the water is diverted from branch channel, therefore the grain size of sediment flowing into underground power station is fine relatively.

(3) In front of underground powerhouse, the flow exchanges in vertical violently, the distribution of sediment concentration is uniform in vertical. Due to the limiting water level of 145m in flood season and 175m in non-flood season, after running of underground powerhouse, the water flows into lower reach through 29~59m deep power station intake and hydraulic turbine, resulting in strong flow exchange in vertical and uniform distribution of sediment concentration and sediment grain size in vertical in front of underground powerhouse.

(4) The mouth of Maopingxi located at 590m upstream of underground powerhouse belongs to accumulative depositional area originally, when underground power station running, the mouth is not be silted up completely due to flow discharge passing through here, with maximum discharge of 4800m^3/s and minimum discharge of 1600m^3/s. When the reservoir running for 76 years, namely after the reservoir is in the state of equilibrium of sediment scouring and silting, the mouth is about 350m wide and the sediment deposition elevation is below 140m at water level of 145m.

4. Sediment accumulation in front of underground power station and in tailrace

4.1 Sediment accumulation in front of underground power station

Under the conditions of all sediment flushing outlets closed, at the end of 30 years of TGP operation, the maximum elevation of sediment accumulation at the front 30m away from underground power station is 119.4m, the general accumulation elevation is 115.6~119.4m, then the accumulation elevation at the front of 4 units has exceeded the intake bottom elevation of 116m. At the end of 50+4 years, the maximum elevation of sediment accumulation at the front 30m away from underground power station is 119.8m, the accumulation elevation at the front 30m away from the whole units has exceeded the intake bottom elevation of 116m, which does not effect the normal water diversion of power station. At the end of 70+6 years, the deposition elevation at the front 30m away from the underground power station exceeds the intake bottom elevation, with the maximum of 120.3m, which does not influence normal diversion of underground powerhouse.

The underground power station is far away from the main channel, the water diversion is from branch channel, therefore, the sediment grain size in front of underground powerhouse is finer than that in the right power station of main powerhouse. The median size of sediment d50 is 0.015mm at the end of 30 years, 0.015~0.020mm at the end of 54 years, and 0.022~0.028mm at the end of 76 years.

4.2 Sediment accumulation in tailrace of underground power station

The tailwater of underground powerhouse discharges through 3 drainage tunnels into the tailrace of right power station directly. Due to the small intersection angle of about 15° between the tailwater of underground powerhouse and that of right power station, the influence on tailwater discharge of right power station is small, only backflow happens locally as a result of flow dissociation after 2 streams of water joining, but no obvious backwater occurs. At the same time, 4800m^3/s flow discharges into underground powerhouse, the flow turbulence intensity in tailrace of right bank power station is strong further, the flow velocity in tailrace increases, especially the velocity of right side increases obviously, which makes the sediment accumulation in the tailrace of right bank power station less compared to that without underground powerhouse.

Because the boundary conditions of the lower tailwater area of right bank power station is stable, the flow turbulence is strong, and the discharge of right power station and underground powerhouse is stable relatively, the flow pattern and sedimentation conditions in the tailwater area is similar during different service period after TGP running.

The model test shows that, at the end of 76 years of TGP operation, within the range of 300m in the downstream tailrace of right bank power station and underground powerhouse, the accumulative sedimentation will be 142000 m^3, reducing by 17% compared to that before the construction of underground powerhouse.

5. Sediment passing through generating units

After normal operation of the TGP for 76 years, the bed load has not be transported to the front of dam, the sediment transported to dam area is still suspended load. The transverse distribution of sediment concentration and grain size for suspended load in front of dam is higher in the middle and smaller in the left and right side of main channel, and due to the long distance from main channel, for underground powerhouse, the sediment concentration is far lower than that of main channel and right & left power stations, and the sediment grain size is finer than that of them. And there is no coarse sediment passing through generating units. During the middle period of operation, when flood discharge is 55900m^3/s in flood season, the sediment concentration of flow passing through generating units is 2.52kg/m^3, and the median size d_{50} of suspended load is 0.018~0.020mm with 8% of them more than 0.05mm. During the later stage of operation, when flood discharge is 55900m^3/s in flood season, the sediment concentration of flow passing through generating units is 3.92~4.01kg/m^3, and the median size d_{50} of suspended load is 0.025~0.027mm with 11% of them more than 0.05mm.

6. Effect of sediment flushing outlets in underground power station

The transverse slope (perpendicular to main flow direction) and longitudinal slope (parallel to main flow direction) of scouring funnels of sediment flushing outlets closely relate to reservoir operation method, sediment inflow and water inflow conditions, topography in front of dam, discharge of sediment flushing outlets, and the thickness, deposition duration, density degree and grain size of sediment in front of underground power station. The sediment tunnels are opened 3 times respectively at the end of 30 years, 54 years and 76 years of TGP operating, under the conditions of sediment discharge of 360m^3/s and sediment flushing duration of 35h, the comparative tests have been carried out for concentrated sediment flushing scheme and separated sediment flushing scheme. For concentrated sediment flushing scheme, only 1 sediment tunnel is arranged for 6 generating units, while for separated sediment flushing scheme, 3 sediment flushing holes are arranged transversely. The test results show that, the sediment flushing effect of separated scheme is better than that of concentrated scheme. For concentrated scheme, part of sedimentation elevation in front of power station is higher than intake bottom elevation after sediment scouring, for separated scheme, the sedimentation elevation after scouring is lower than intake bottom elevation of 116m and the stable scouring funnels forms. The transverse slope and longitudinal slope of scouring funnels are 1:2.9~1:3.9 and 1:6.8~1:11.7 respectively, the sediment flushing effect is remarkable, without sedimentation in front of power station, which can ensure the normal water diversion of underground powerhouse for a long time.

7. Inflow conditions of communicating cannel scheme

In the later period of TGP operation, with sedimentation near the mouth of Maopingxi, the inflow conditions of power station becomes worse. In order to

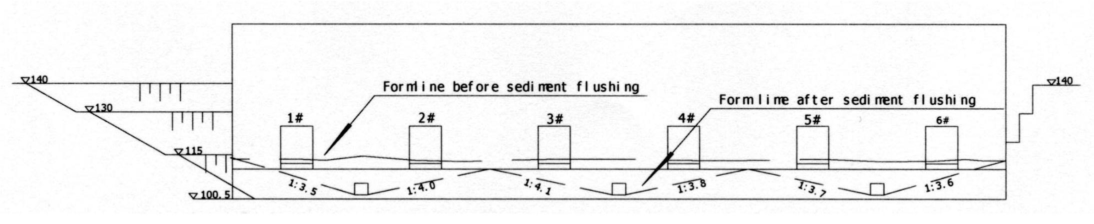

Fig. 2. Topographic Changes before and after opening of sediment tunnels in underground power station at the end of 30+2 years opration of TGP

improve the inflow conditions and ensure the long-term safe operation of underground power station, the communicating cannel for compensating water from lateral is excavated in Baiyanjian Mountain between underground power station and right bank power station during construction period. However, after running of underground power station, the flow from communicating cannel and the flow from Maopingxi mouth join together to form eddy with vertical axis, and with the long-term operation of reservoir, the strength of eddy increases, the flow pattern in front of power station becomes complicated.

Because a large amount of rock is needed for TGP construction, if the top of mountain and part of mountain on right side upstream of Baiyanjian are excavated, the water can be diverted from the main channel of Changjiang river upstream of underground power station to Maopingxi, the inflow mouth of underground power station can be widened, and the flow pattern will be smooth, which can meet the demand of inflow into underground power station during later period. And at the same time, the communicating cannel will be blocked.

8. Conclusions

(1) After impoundment of the TGP, if water diversion is performed over a long period of time for underground power station, the mouth of Maopingxi will not be silted up to influence inflow. At the end of 76 years of operation, the normal water diversion still can be ensured basically.

(2) In the earlier stage of operation, there is no coarse sediment passing through generating units, while in the later stage, when flood discharge is $55900m^3/s$ in flood season, the sediment concentration of flow passing through generating units is $3.92\sim4.01kg/m^3$, and the median size d50 of suspended load is 0.025~0.027mm with 11% of them more than 0.05mm.

(3) When sediment tunnel is opened, deposited silt in front of power station is scoured to form stable scouring funnels with transverse slope and longitudinal slope of 1:2.9~1:3.9 and 1:6.8~1:11.7 respectively. For separated scheme, the accumulation elevation in front of power station will be lower than that of intake during the earlier, middle and later stages of operation, and the front of power station can be kept clear to ensure the normal water diversion of underground powerhouse for a long time. The sediment flushing effect of separated scheme is better.

(4) Due to small intersection angle between the tailwater of underground powerhouse and that of right power station, when they join together there is no obvious backwater occurring. The discharge of tailwater of underground power station is favorable to the reduction of sedimentation in the tailrace of power station tailrace.

(5) After implementation of communicating cannel scheme, the flow pattern in front of underground power station becomes complicated, it is recommended to excavate the top of mountain upstream of Baiyanjian to enlarge the inflow in upper reach of power station and to block communicating cannel, thus the long-term inflow requirements of underground power station can be satisfied.

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(The work was carried out under the instruction of Prof. Lu Jinyou and Prof. Wei Guoyuan of Changjiang River Scientific Research Institute)

Sediment Management by Satellite Remote Sensing Technique

D.M. More
Director General,
Maharashtra Engineering Research Institute & Engineering Staff College,
Nashik, Maharashtra, India

S.K. Kalvit
Assistant Research Officer,
Maharashtra Engineering Research Institute, Nashik,
Maharashtra, India

Synopsis

Annual rainfall, the only source of water is acutely seasonal and erratic over the large part of the country. It therefore became inevitable for the planners to build large number of reservoirs by undertaking a massive programme of construction of dams during the last few decades. Maharashtra one of the prominent states of India, industrially well developed, cannot be an exception to this dam building activity. So far 1813 dams (17 major, 173 medium & 1623 minor) have been completed and construction of another 1004 dams (65 major, 26 medium and 813 minor) is going on.

The water holding capacity of reservoir is affected by sedimentation. The erosion of soil cover in the catchment and the subsequent transportation and deposition of the silt over the bed of the reservoir is a natural phenomenon. The rate of sediment deposition could however be minimized by soil conservation treatment on a sustained basis. The main factors responsible for soil erosion are topography, climate, hydrology, and land use pattern. The first three are controlled by nature and under normal circumstances they are subjected to no changes by human activities. But the land use pattern can be changed suitably to arrest the erosive action. The process of sedimentation of the reservoirs can be brought under control by undertaking catchment area treatment programmes on a long term basis. These measures can be of engineering devices, agronomic treatment, a forestation and grass cover.

Activities such as identification of areas prone to erosion, catchment area prioritisation to estimate the quantum of silt going in to reservoirs, periodical capacity surveys, effect of sediments on quality of water, monitoring the growth of aquatic vegetation viz. water hyacinth and water weeds in the reservoirs are carried out under sediment management. A High Tech approach of 'Satellite Remote Sensing Technique' is being adopted for the quantitative and qualitative estimation of silt in the reservoir. The work has already been initiated and so far capacity resurveys of almost 25 major reservoirs have been completed. From the findings of the study completed so far it is seen that the rate of sedimentation in reservoirs situated in the Western Ghat i.e. hilly area having better biomass cover is low as compared to that for reservoirs having larger catchment covered with more land under cultivation. The useful capacity of reservoirs located in steep valleys does not get affected in the initial period of 25+ years, was also observed. There is no development of aquatic weeds / water hyacinth in the reservoirs situated in the western ghat area was also seen. But the reservoirs in the plain and flat basin have started showing signs of faster sedimentation along with the undesired spread of harmful vegetation in the submergence.

Study Area

Maharashtra State situated on the West Coast of India is the third largest state in India both in respect of population and area. It has the maximum number of large dams in the country. Most of the dams have been constructed in the post independence era. These dams have played an important role in economic development of the state, by increasing food production as well as augmenting power supply by generating hydropower. About 1813 dams (17 major, 173 medium & 1623 minor) have been completed and another 1004 dams (65 major, 26 medium and 813 minor) are under construction. The State can be divided physically in to coastal strip of Konkan and Deccan plateau. The dividing range commonly called Sahyadri runs almost parallel to Arabian Sea at a distance varying from 25 to 50 kms.

A large number of rivers originate from Sahyadri

Fig. - 1, Satellite image of Maharashta State

range and flow eastward / southeast direction. The river Krishna, Koyna, Bhima, and Godavari are main rivers among them. Due to its peculiar topography and high rainfall, the western Ghat area houses large number of dams, which are relatively close to one another. The rainfall in the catchment area of these projects, which falls in the Sahyadri range, is in the range of 1500 - 3500 mm, and contributes to high runoff in the rivers during monsoon. The Deccan plateau is relatively flatter one and has a major part of the land use as a cultivable land. Sugarcane is the main cash crop in the western part whereas cotton is taken on large area in the Middle East part of the state.

Sedimentation phenomenon

Sedimentation is a natural phenomenon. It is a process in which the top soil cover of the land gets eroded, transported and deposited over the reservoir bed at different elevations due to physiographical, climatological and hydrographical factors. The main factors are:

1) Nature & type of soil in the catchment area
2) Topography of catchment area
3) Vegetation cover
4) Rainfall intensity
5) Physical and chemical weathering
6) Discharging capability of the streams etc

Periodic earthquakes may also sometimes make the soil cover loose. If the rock in the catchment happens to be fragile & withered due to repeated earthquakes it gets loosened & landslides occur & consequently, the eroded material is carried away by the drain water and the silt load increases considerably in the post seismic period. The process of soil erosion is further accelerated by various faulty-land use practices like deforestation in the upper parts of the catchment, cultivation on hill slopes, unrestricted grazing of grassy lands, shifting cultivation, and cultivation of crops & vegetation unsuitable to field conditions. This eroded material gets deposited over reservoir bed. The smaller size of material is deposited beyond delta front. If the stream carries appreciable fine silt, much of this material remains in suspension for long time and may settle down over the entire length of the reservoir. Thus the storage capacity of the reservoirs goes on reducing progressively at all elevations affecting their useful life.

Necessity of sediment management

The sedimentation poses threat to the useful capacity of reservoirs by rapidly eating away the storage potential created behind the dams. The silt in suspension is very much prone to settlement in the reservoir owing to the dampening effect on the natural draining capability of the rivers and streams across which the massive barriers in the form of dams are created. The sedimentation has a direct effect on the annual water use planning for various purposes like irrigation, industry, drinking water supply and so on. The sedimentation in reservoir can also cause operational as well as ecological problems due to turbidity and due to gradual delta formation at the upstream end of the reservoir. It also promotes the growth of water hyacinth and other water weeds in the reservoirs. These aquatic weeds obstruct the flow courses and encourage silt deposition. Thus sedimentation process and growth of water weeds run in a cyclic manner, thereby causing progressive reduction in storage capacity of reservoir. The true capacity of the reservoir helps in scheduling the water use in a more rational way. The systematic & unbiased evaluation of sedimentation on a continual basis therefore becomes utmost necessary.

The correct assessment of silt load in the reservoir, in the post dam situation is complex one. It is time consuming also if decided to be done by conventional method. The large number of small and big reservoirs created over the last few decades and their capacity assessment with respect to sedimentation has been a difficult task for the managers of the water supply schemes.

The reduction in water storage hampers the entire irrigation and domestic water planning. In India every year about 5334 M Tonnes of soil (about 16.35 t/ha) is eroded annually out of which about 10 % gets deposited in the reservoirs causing reduction in their storage capacity by 1% to 2% annually. Reduced storage of water on one side and increased demand on other side puts the management authorities on edge in making equitable distribution of water. Heavy floods every year cause more sedimentation in the reservoirs. Periodical assessment of capacity and study of sedimentation of reservoir has therefore attained utmost importance in all water resources development projects.

Measures for controlling sedimentation

As discussed above the soil erosion ultimately results in sedimentation either in rivers or in reservoirs depending upon the silt carrying capacity of the rivers. Hence the very first thing in minimizing the sedimentation is to prevent soil erosion. The soil conservation measures like nalla bunding, check dams, land terracing, etc help reduce soil erosion. The methods of soil conservation can be broadly classified into three categories viz. (1) Agronomic (2) Aforestation, and (3) Engineering.

1. Agronomic Measures include Crop rotation, Strip cropping, Contour farming, and development of grazing land etc.
2. Aforestation includes planting new trees, maintaining natural vegetation etc. The natural vegetation cover destroyed partially or fully needs to be re-established. This also includes checking of deforestation, seeding of pastures, planting the perennials on eroded and gullied areas etc.
3. Engineering measures include Nalla bunding, check dams etc.

The selection of each type of measure can be location specific.

Monitoring the sedimentation through Remote sensing technique

Remote sensing satellite gives a synoptic coverage of the land use pattern on the surface of the earth. This gives the information about the vegetal cover on the surface (actual agricultural land, forest land, area under natural vegetation etc) the extent of barren land, habitated land etc. Land use pattern of the inaccessible area can also be effectively mapped after studying the satellite images. Information about soil type, slopes, land use pattern derived from the images of remote sensing satellites in conjunction with the hydrological factors like rainfall intensity, runoff can be effectively used to know the silt load from the catchment and probable silt volume likely to be dumped in to the reservoirs. In some cases it is possible simply by visual observation of the satellite image to locate the erosion prone part of the catchment / sub catchment The silt in suspension and the shallow depth due to sedimentation at the mouth of river / nallas draining into reservoir is clearly reflected in the satellite image in bluish or cyan colour. The land use /land cover information derived from satellite images clubbed together with the physiographic information such as slope, relief, and hydrological parameters like rainfall can help predict soil loss with the help of universal soil loss equation. After identifying such erosion prone areas they can be treated with proper measures. The sedimentation in the reservoirs can be monitored at regular intervals by conducting periodic sedimentation surveys. Satellite remote sensing technique is found to be useful, for periodic repeat surveys, which provide a faster and cost effective solution to this problem. Satellite remote sensing technique requires the satellite data and some field data like lake levels, original water spread area and storage capacity and a computer system with image analysis software for the study. It has been experienced that for reservoirs having large water spread area; remote sensing technique is faster and cheaper as compared to hydrographic survey method. This method is faster than previous two methods and requires no field job but only laboratory work. Manpower required is also very less. Just one or two trained and experienced engineers/scientists are enough for this work.

The satellite based sediment management

Maharashtra Engineering Research Institute, (M.E.R.I.) Nashik, a supporting organ of the State Water Resources Department, has done considerable work in the field of reservoir sedimentation and capacity assessment studies using both satellite remote sensing technique as well as hydrographic survey method. The Institute has so far conducted capacity assessment studies by remote sensing technique for about 20 major reservoirs including Jayakwadi, Ujjani and Koyna reservoirs which are the largest reservoirs in the state both in terms of water spread and storage capacity. These three reservoirs are having storage capacity of more than 3000 Mm^3. The studies were found to be economical. Manpower requirement for remote sensing based study is also less. Another advantage of this technique is that one image has large ground coverage, which can accommodate number of adjacent reservoirs in one scene. The satellite data selection for the capacity assessment study of all the reservoirs covered in one image, if carefully done with due consideration to cover maximum fluctuation in the water levels in all these reservoirs, helps reduce the cost of capacity assessment work per reservoir substantially.

In the earlier period there was a practice to fix the dead storage level of the reservoir at a level, which has a capacity to accommodate the total silt estimated over its designed life and likely to be deposited in the reservoir based on some rational or empirical formula. It was assumed that the entire silt would be deposited below this dead storage level and there would not be any encroachment over the live storage part and the reservoir would continue to serve its planned benefits throughout its life span. But in practice, it is observed that the sedimentation takes place in the reservoir throughout its full range of depth at different rates, encroaching live storage which results in reduction of the useful storage capacity, causing an adverse impact on the planned irrigation potential, power generation and other uses.

In estimating the sedimentation in reservoirs by remote sensing technique, multidate satellite imageries of reservoirs at different water levels are obtained and these are analysed using digital image analysis technique with the help of standard image analysis software. The images are classified into different classes such as water, land, vegetation etc and water spread area is measured. Maximum likelihood Classifier is normally adopted for classification. Capacity between successive levels is worked out based on satellite measured water areas using prismoidal formula, as below.

V = H / 3*(A1 + A2 + SQ.RT.(A1*A2))

Where V is the volume between two levels.

H is difference between two levels.

A1 & A2 are areas at successive levels.

This capacity is then compared with the designed capacity and reduction in capacity is attributed to sedimentation. This method is cost effective, requires lesser manpower and time and has better level of accuracy. With the availability of high resolution satellite data, the accuracy in the results can be improved further. Considering the importance of reservoir capacity evaluation and sedimentation survey and the cost effectiveness of remote sensing technique, Maharashtra Engineering Research Institute, Nashik has planned this work in a phased manner. In its first phase survey of 25 reservoirs has been completed. As mentioned earlier, hydrographic surveys are costly and can not be carried out repeatedly at regular interval. Application of remote sensing technique has therefore a first priority

Field Data

The data of water levels in the reservoir on different dates of satellite pass, their respective water spreads and the volumes as per the project report are required for the study, which can be obtained from the field authorities.

Satellite Data

IRS 1C /1D or P6 (Resourcesat) Satellite data of LISS III sensor having a spatial resolution of 23.5 m is used for the study of reservoirs in Maharashtra.

Methodology

These digital images are rectified with corresponding Survey of India toposheets of 1:50,000 scale. These are the subsetted covering an area of interest. Then the subsetted images are classified into different classes such as land, water, vegetation etc by taking appropriate training samples of the respective classes. For classification of the images, Supervised Classification Technique with Maximum Likelihood Classifier is generally adopted. After classification, areas of different classes are measured from histogram of pixels belonging to respective classes, by multiplying pixel counts of each class by the size of the pixel. Water spread areas of the reservoir in all satellite sub images are thus measured. From the water-spread area obtained as above for different levels, the storage capacity between successive levels under study is computed using standard prismoidal formula. The volume of water i.e. the storage capacity calculated at various levels is then compared with the designed capacity, i.e. storage capacity between the same levels, as per the project report. Any change, either increase or decrease is broadly attributed to erosion or siltation respectively. So far, about 25 major reservoirs have been studied by satellite remote sensing technique at M.E.R.I., Nashik, in last 3 years. On an average about 75 % to 80 % live storage of the reservoirs has been covered under study. The area-capacity curves and the tables have been updated for these reservoirs.

Monitoring growth of aquatic weeds and water hyacinth

While studying the sedimentation in reservoirs through satellite images it was observed that in submergence area of many reservoirs, large growth of aquatic vegetation like Besharam and other aquatic weeds like Pan-kanis has occurred especially in the reservoirs in the flat areas. This was clearly reflected in the satellite images. Ujjani, Jayakadi, Manjara, Upper Wardha, Lower Terna, and Lower Wunna reservoirs are some of the major reservoirs where such growth is predominant. In other reservoirs, growth of such vegetation has started developing. Such vegetation grows in the water and when the water level depletes it is exposed to land surface. During hot weather leaf shading of this vegetation occurs. During this period such patches of aquatic weeds appear whitish yellow in colour. When water level in the reservoirs increases in the next rainy season, these again turn dense because of large leaf area and appear bright red in satellite image. These changes in vegetation cover can be clearly observed in the images. This type of vegetation is not even edible for animals. They are not suitable for green mannuaring also. Their spread is very fast and within few years they encroach over hectares of submergence area. Sometimes the height of such vegetation can be that of a man and the density is such that it is difficult for a man to walk through. Such vegetation develops first in the delta formation area which is a result of sedimentation in the upper reaches of the reservoir. It then traps the silt coming into the reservoirs and allows it to settle there which forms a platform for the vegetation to spread further. Thus sedimentation and growth of vegetation are complementary to each other. It has also been observed that wherever the domestic and the industrial waste is carried by natural drainages like rivers and streams in to the reservoirs, growth of these water hyacinth and other aquatic weeds is more predominant. In case of the reservoirs located in the western ghat area covering hillocks and forest which are not contaminated, harmful vegetation growth has not been observed. Koyna, Dhom, Kanher, Warasgaon, Panshet and Bhandardara are some reservoirs where the growth of aquatic weeds was not observed. As stated earlier such vegetation grows and spreads profusely in the water, consumes the dissolved oxygen in the water and prevents sunlight from reaching the water surface. Such areas create suitable environment for mosquitoes breeding. It is therefore necessary to eradicate totally such vegetation

Work of sediment management for some major reservoirs in the state.

1) Shivaji Sagar Reservoir (Koyna Hydroelectric Project) Dist. Satara
2) Bhandardara Hydro electric project Dist. Ahamadnagar
3) Vaitarna Hydropower Project Dist. Nashik
4) Pench Hydroelectric Project Dist. Nagpur

5) Jayakwadi Project (Nathsagar Reservoir) Dist. Aurangabad
6) Ujjani Reservoir Dist. Solapur
7) Upper Wardha Reservoir Dist. Amravati
8) Manjra Project Dist. Beed.
9) Lower Terna Project Dist. Osmanabad
10) Bor Project Dist. Wardha

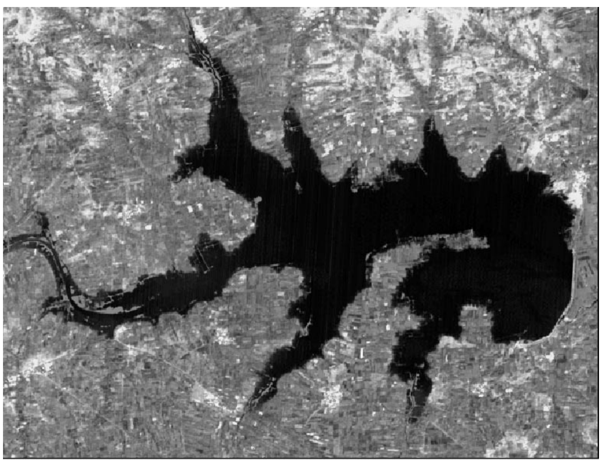
Findings of the work

Even after a span of 40 years no siltation has occurred in the operating zone of Koyna reservoir. On the contrary, due to erosion of the steep banks all around the fringe of the reservoir, the soil crust has appeared to be slipped down exposing large boulders, slightly widening the upper zone. Whatever sedimentation has occurred due to erosion of the banks it is deposited around MDDL and below. There was no growth of aquatic weeds / water hyacinth. The entire catchment area is covered with forest. The land under cultivation is meager. Due to better biomass cover the soil erosion is not significant.

Bhandardara project is almost 80 years old and still functioning effectively. Remote Sensing based study was conducted for this reservoir. The study revealed no reduction in the storage capacity in the normal operating zone of the reservoir. The survey did not give any indication of sedimentation in the normal operating zone. Neither there was any ill effect of aquatic weeds in the reservoirs.

Vaitarana project. This reservoir is located in the Sahyadri range i.e. western ghat of Maharashtra. The river Vaitarna has a natural fall of 274 m, which is used for hydropower generation. Sediment assessment study of this reservoir was taken up in the year 2004-05 and was completed recently. The findings of the study revealed that like Bhandardara and Koyna Hydroelectric project, this reservoir also did not suffer a loss in storage due to siltation in its normal operating zone

Pench hydroelectric project is a major hydroelectric project in the eastern part of the state. Almost 88% of the live storage zone of the reservoir was studied for estimating the up to date storage capacity and it was revealed that the storage capacity in this zone was reduced by 4.22% in a span of 25 years. Since the catchment area is mainly in the forest and does not cover any industrial township nearby, the water quality was seen good. The growth of water hyacinth or other aquatic weeds was also not observed.



Red portion within the submergence area shows the spread of aquatic weeds

Study for Manjara reservoir located in Beed District of Maharashtra revealed that because of flat basin, and more cultivable land in the catchment, the sedimentation in the reservoir was higher than the one which was predicted for the design. Hence it was felt necessary to measure the actual sedimentation and estimate the reduction in storage capacity of reservoir. The remote sensing based sedimentation study was conducted. The study showed that sedimentation has caused reduction in storage capacity of the order of 27.03 Mm^3, which is about 13.16 %

A large growth of aquatic weeds like *Beshram* (a local name) was seen within the reservoir at various levels. In the satellite images, dense vegetative growth was seen in the water body especially near F.R.L. and below and along the reservoir periphery at few locations.

Sedimentation study of Jayakwadi reservoir by Remote Sensing Technique was done in year 1999. Results revealed that there is a reduction in the storage capacity of the order of 6.98 %, in a span of about 20 years. The study showed reduction per year in storage capacity, to the extent of 0.35 %. From the study of few levels below M.D.D.L, it appeared that the rate of siltation in dead storage zone is more than that in live storage zone. The growth of water weeds, water hyacinths at few locations around the reservoir, especially at the confluence of drainage with the reservoir (at southern part) was clearly seen in the satellite images. Development of such water weeds, water hyacinths, needs to be monitored periodically to prevent its spread inside the water body and also at its periphery which may also contribute to sedimentation in reservoir in future.

Comparison of Results with Hydrographic survey

The results of sedimentation study by remote sensing method are compared with those obtained by hydrographic survey in respect of Jayakwadi reservoir. Comparison was as below

	Remote Sensing	Hydrographic
Reduction in capacity	6.98 %	5.86 %

From the above, it was seen that remote sensing results fairly matched with the results of hydrographic survey.

Yashwant Sagar reservoir (Ujjani Project), The reservoir basin is very flat and its water spread is almost 125 km.

The capacity evaluation and sedimentation study of Ujjani reservoir by Remote Sensing technique, using IRS 1C/1D LISS III satellite images (23.5 m. spatial resolution) was done in the year 2001-02 for assessing the

Dense vegetation growth observed in the submergence area of Ujjani reservoir

up to date storage capacity. The study revealed that the storage capacity of Ujjani reservoir was reduced by 10.55%. Due to large water spread and flat slope, the sedimentation in the live storage zone of the reservoir was seen higher than usual.

It was further seen that the storage capacity of reservoir was affected considerably due to the combined effect of sedimentation and growth of dense vegetation. The vegetation growth was also observed within the submergence area, and around the edges of the reservoir. This reservoir was seen with large area occupied with aquatic weeds.

The growth of vegetation deteriorates the water quality and also creates the favourable conditions for mosquito breeding causing environmental degradation.

The information received from the satellites is unbiased and therefore this technique has become a versatile tool for periodical monitoring of sedimentation and vegetative growth in the reservoirs.

Effect of Small Dam on Sediment Control in Different Treated Catchment of Shiwalik Region of India

I.P Sharma and Rohitashw Kumar
Department of Soil Science and Water Management, India

Dr. Y.S Parmar
University of Horticulture and Forestry, Nauni, Solan, India

Abstract

Shiwalik region of India comprise 2.14 m ha, which is most fragile and having degraded area. The average annual rainfall is up to 1000 mm out of which 80 per cent rainfall goes as runoff. The 80 per cent of the rainfall is received during monsoon period. The area is having very scanty vegetation. A study was conducted to minimize the degradation by treating the catchment through vegetative and mechanical measures. The study conducted in forest land, naked land and cultivated land for minimize the sediment which occurs due to soil erosion. A current estimates show that about 60 per cent of the land suffer from soil erosion. The selected watersheds have an area of 530 ha. For the study 29.20 ha area was selected having 2.25 Km length and 50 to 300 m width. In the catchment forest, small check dams were treated with plants of different species. Gabion spurs and check dams were constructed. For channelization of stream flow, selected suitable mechanical measures with vegetation reinforcement. In this connection Spurs and gabion wall were constructed to direct water towards the central line. About 43 ha of torrent bed were claimed.

Among the mechanical measures, gabion spurs and check dams were constructed for control soil erosion and stabilizing the banks. The gabion spurs are attracting and deflecting type. The protected length of bank varies linearly with spur length. In case of attracting spurs the protected length has been 3.5 times the length of spurs. The deposited silt volume varied positively with the angle of spurs. The sediments deposited in catchment under forest land constructing chech dam varies 0.1-20 tons, in cultivated land it varies 2.0-30.0 tons and in naked land sediment deposition is very high i.e 10.0-81.0 tons in one season. Therefore it was observed that the mechanical as well as vegetative measures helped in the channelization of water cause towards the central line with tune of 10-100 m and the sediment was highest in the naked land and minimum sediment was found in forest grassed land.

Key words: *Spurs, gabion structure, sediment and torrent*

INTRODUCTION

Indian land resources are under immense pressure as reflected in the fact that it share only 2 per cent of the world geographical area. A current estimate show that about 60 per cent of the land suffer from soil erosion. The study undertaken at the lower shiwalik of Himachal Pradesh of India in KJS watershed. The area is under torrent affected and tremendously affected by erosion problem. The torrent are ephemeral mountain stream emanating from the outer Himalayas and Shiwalik and empty into major rivers and cause extensive damage of life and properties in valley areas by scouring their beds, undermine and erode their banks. Torrent has been mostly taken as small mountain stream, rushing down the slope with flashy floods and often loaded with sediments. Area under torrent in Shiwalik in the state of Punjab, Uttar Pradesh, Haryana, Uttaranchal and Chandigarh was assessed through remote sensing techniques and GIS analysis. The area under torrent in Himachal Pradesh is 399.08 sq. km. The selected watershed has an area of 530 ha. For the study 29.20 ha area was selected having 2.25 km length and 50 to 300 m width. In the catchment area, forest land, torrent affected and cultivated land were treated.

Due to large scale denudation in Shiwalik hills, area once covered with thick vegetation replete with wild life has been converted into bush land of little economic importance. The gentle streams got converted into ferocious torrents laden with sediment. Consequently the area affected by the torrent has been increased over the years. Besides, geographical situation, climatic factors and above all biotic pressure and over-exploitation are major causes of land degradation. An area of about 186.7 m ha (57 %) of total geographical area of the country is suffering from various degradation hazards, of which a dominant fraction (45 %) is under water erosion.

In one Himalayan foothills and Shiwaliks mis-management of fragile steep slopes and marginal lands, overgrazing and illicit cutting of forest for fuel and fodder needs, faulty road construction and unscientific mining have caused extensive land sliding and mass erosion on the hill slopes. The resultant excessive debris load carried into hill torrent down below. The coarser fraction of the debris get deposited in the form of gravel. Isolated island once the stream reaches the milder slope in the valley. Reduced stream slopes cause considerable reduction in the carrying capacity of the streams. Flow tends to migrate laterally overstepping or undermining the banks which are often low and ill defined. This leads to encroachment of the streams into the adjacent agricultural, forest and horticulture lands. To control the erosion problem different treatments were used and a study undertaken the sediment load by constructing different size (small) dams under the forest, naked and cultivated lands.

METHODOLOGY

A study undertaken in the area of torrent affected of KJS watershed. The total area of watershed is 530 ha and considered a stretch of 1.5 – 2.0 km length. The treatment of small dam were conducted in three different type of land i.e. naked land, forest land and cultivated land. The gabion spur and small dam were constructed. For channelization of stream flow, selected suitable mechanical measures with vegetation reinforcement. In this connection, gabion spurs and loose wall were constructed to direct water towards the centre line. Using gabion structure treated with vegetative spur of different combination to protect the stream banks to further extension. To channelize the flow and measuring the sediment load carried by water the small dam were constructed The gabion structure are two i.e. attracting and deflecting type. The deposited debris volume varied positively with the angle of spurs. The protected length has been 3.5 times of the length of spurs. It is assumed that the life span of small dam is 10 – 15 year that it will be fully deposited with sediment volume and area reclaimed in given time of span, but it is observed that the check dam constructed in naked land has been full with sediment in 2-3 years and received less amount of sediment in forest grassed land.

RESULT AND DISCUSSION

In the study area, small check dam were constructed in three type of land, forest, torrent and cultivated land. The gabion structure erected by wire. The length of gabion structure and small is an average 7.0 m and height 2.0 with width 1.5 m. The sediment deposited in up stream and down stream was recorded. For channelization of stream flow, selected suitable measures with vegetation reinforcement. In this connection, gabion spurs and wall direct the flow of water towards the centre line with tune 10 – 100 m. About 43 ha of torrent bet were reclaimed. Two type of gabion spurs were constructed i.e. attracting and deflecting. The protected length of bank varies linearly with spur length. In case of attracting spurs the protected length has been 3.5 times the length of spurs. The sediment deposited by constructind small dam in forest land varies 0.1 – 20.0 tons and cultivated land 2.0 – 30.0 tons and naked land it is maximum i.e. 10.0 – 81.0 tons in one season. Therefore, maximum sediment load volume was found in catchment in naked land.

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Legislation and Measure to Support Dam Reservoir Area Development

Young-hoon Choi
General Manager, Water Resources Business Sec.,
Water Resources Management Dept.
KOWACO, Dajeon, Korea

Hee-bok Ahn
General Manager, Dam Maintenance Sec. III,
Water Resources Management Dept.
KOWACO, Dajeon, Korea

Sung-tae Hwang
Assistant Manager, Water Resources Business Sec.,
Water Resources Management Dept.
KOWACO, Dajeon, Korea

Tae-wan Moon
Director, Water Resources Management Dept.
KOWACO, Dajeon, Korea

Abstract

Large scale Multi-purpose Dams provide water supply, flood control and power generation to downstream areas, but they also can have a negative effect on the surrounding areas. Therefore, Dam facilities are constructed in an environment friendly manner starting with project inception and continuously during operations. As well, improvement projects are conducted that will help improve standard of living in dam area for residents and visitors. Additionally, after completing the construction, a regular portion of the income from dam operation is used for support projects that are conducted to increase the income level of local residents and to promote social welfare improvements in the area surrounding the dam. Moreover, as KOWACO conducts many projects aiding the development of the dam area and carried out proper maintenance, effort is also made to decrease any negative effects that may be a result of dam operations. This paper introduces the scope of support and improvement projects conducted through revision of related law and regulation. As well, it considers the all aspects concerned with Korean dam areas using instances of construction of environmentally friendly design. We also examine environmental improvement projects at existing facilities and cases of improving dam operations and damage mitigation.

1. Introduction

If rivers flowed regularly and provided abundant water, it would be unnecessary to worry about floods and we could obtain necessary water from waterways or water- intake banks.

Unfortunately, however, the reality of water flow in natural rivers is very different from this. There has been a proverb that says "the one who manages water resources well deserves to rule the people". It is originated an ancient Chinese legend that tell us of King 'Yu(禹)' of the 'Xia(夏)' Dynasty who succeeded in flood control and became the Emperor of the Dynasty. In Korean history, flood control efforts were actively promoted during the Joseon Dynasty. As a result, total number of small dams equaled to 3,378 by the sixth year of King Jeongjo (A.D 1782)

In recent years, the world has repeatedly suffered from disasters like flooding and drought due to global warming. Korea also has suffered from a three or four year cycle of floods origin disasters owing to unpredictable weather changes. Floods in 1984, 1990 and 1995 in the Han river area and in 1996, 1998 and 1999 in the Imjin river area plus typhoons "Rusa" in 2002 and "Maemi" in 2003 are few example among other incidents. As well, nationwide drought occured in 1994~1995 and regional drought in the winter of 1996. Meanwhile, a severe drought occured in June 2001 which was considered the worst drought in 90 years. The Flooding July of the same year was also recorded as the worst disaster in 37 years.

Nevertheless, the problem is that such phenomena are not an end but rather a beginning. As well, those disasters repeat very frequently in these days. Such worldwide disasters could cause serious damages to human-being's life and properties. Realistically, natural phenomena themselves can hardly be prevented, however, we are able to protect ourselves by preparing for them. One of the most effective countermeasures we are considering is dam construction. Especially multi-purpose dams play important roles in controlling floods during the monsoon seasons and aid in overcoming drought during dry seasons.

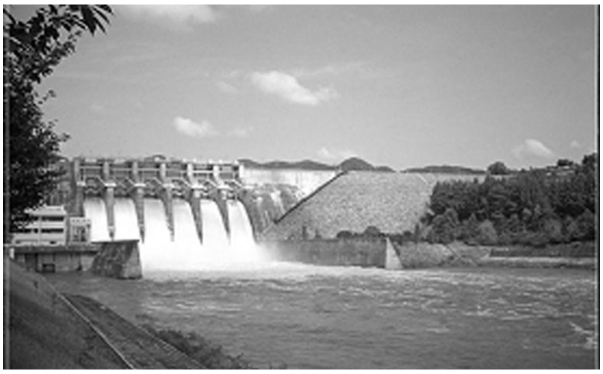


Figure 1 Flood control of dam

In spite of dam's positive roles, there are also many emotional and material resistances from the people who should emigrate from the submerging area. Traffic inconvenience and various kinds of restrictions implemented to protect the surrounding natural environment and to maintain water quality at a dam's reservoir are other reasons of negative responses from the residents. As a result, the living standard of people living adjacent to areas tends to decreasing. In the same manner, Korea also has been experienced such problems caused by dam construction, and as a result, residents' voices demanding land compensation, moving and living countermeasures, road and bridge construction, agricultural damage compensation, local environment preservation measures and other kinds of support have increased since the late 1980's. And recently, one of the most important pending matters dam construction facing is to find solutions to settle all the various demands requested by residents, local governments and NGOs.

Therefore, this article will explorer the systematical context of support and improvement projects for dam areas in Korea and introduce real cases of environmentally-friendly maintenance for dam areas.

2. The outline of support measures for dam area

Support for area adjacent to dams in Korea started with a support fund collected from electrical power plants. The surrounding area was benefited from this support program for the first time after the Act on Assistance to Electric Power Plants-Neighboring Areas was enacted in June 1989. This supporting project was conducted by regional governments or the power plant enterpriser for the area within 5km radius from power plants and was used to increase income and quality of education, to provide public facilities and to improve public relations.

Meanwhile, support for multi-purpose dam areas began in 1995 as 'The Specific Multipurpose Dam Act' was legislated in December 10, 1993. Afterwards, enactment of the 'Act on Construction of Dams and Support for Dam Area' (hereinafter referred to the "Dam Act") in September 7 1999, 'The Specific Multipurpose Dam Act' was abolished and regional support projects in the 'The Specific Multipurpose Dam Act' were enlarged and classified into support and improvement projects in the "Dam Act" and this has continued to date. The contents of the Support and Improvement projects of the "Dam Act" are as follows: Dams with its reservoir area is more than 2million m^2 or total reservoir capacity is more than 20million m^3 are falling under the category. The supported area will be within 5km from dam's planned flood level and within 2km from dam's power plant and other areas where dam manager considers it is necessary. Support projects consist of plans for regional support, residents' support and other support. Improvement projects consist of plans for increasing income, improving welfare, cultural facilities and public facilities. The amount of financial resources is, in the case of support projects, from 6% of the earnings from sales of electrici-

Table 1 History of support project for dam area

Legislation	Financial resources	Period
Establishment of the 『Acton Assistance to Electric Power Plants-Neighboring Areas』 (1989.6.16)	- Within 0.3% of earnings from sales of electricity 2years prior to the year	1991~1994
Revision of the 『The Specific Multipurpose Dam Act』 (1993.12.10)	- Within 1% of earnings from sales of electricity 2years prior to the year - Within 5% of earnings from sales of dam water 2years prior to the year	1995~2000
Establishment of the 『Act on Construction of Dams and Support for Dam Area』 (1999.9.7)	- Within 2% of earnings from sales of electricity 2years prior to the year - Within 10% of earnings from sales of dam water 2years prior to the year	2001~2002
Revision of the 『Act on Construction of Dams and Support for Dam Area』 (2001.12.31)	- Within 3% of earnings from sales of electricity 2years prior to the year - Within 10% of earnings from sales of dam water 2years prior to the year (Water supply-only dam : 15%)	2003~2004
Revision of the 『Act on Construction of Dams and Support for Dam Area』 (2004.1.29)	- Within 6% of earnings from sales of electricity 2years prior to the year - Within 20% of earnings from sales of dam water 2years prior to the year	2005~

Table 2 History of improvement projects for dam areas

Legislation	Financial resources (per dam)	Period
Establishment of the 『Act on Construction of Dams and Support for Dam Area』 (1999.9.7)	- Dams under construction : KRW 20~30 billion	2001~2002
Revision of the 『Act on Construction of Dams and Support for Dam Area』 (2001.12.31)	- Dams under construction or scheduled: KRW 20~50 billion - Constructed dam(before 2000. 3. 7): KRW 20~30 billion	2003~2004
Revision of the 『Act on Construction of Dams and Support for Dam Area』 (2004.1.29)	- Improving projects for flood control only if dam is newly established	2005~

ty and 20% of earnings from sales of dam water (total KRW 42.5 billion (USD 42.5 million) in 2005). For improvement projects, it is about KRW 20~50 billion (USD 20~50 million).

3. Support projects for dam areas

Based on the "Dam Act", support projects for dam areas are being carried out every year after completion of dam. According to the "Dam Act", the details are as follows. Whenever a Dam is completed, the dam management organization (i.e. the Ministry of Construction and Traffic) or the public trustee for dam management such as Korea Water Resources Cooperation must conduct support projects for dam areas to improve the residents' welfare, according to a Presidential Decree.

Dams with its reservoir area is more than 2million m^2 or total reservoir capacity is more than 20million m^3 are falling under the category. The supported area will be within 5km from dam's planned flood level and within 2km from dam's power plant and other areas where

Table 3 Details of support projects for dam areas

Classification		The details of works
1. Regional Support	a. Increasing income	(1) Agriculture, forestry, fishery related works (ex. collective farming facility, repair facility for farm machinery, collective storage of the crops, farm roads, irrigation channels, pumping facilities for farm and etc) (2) Livestock industry related works (ex. collective purchase of water control materials, collective composting facility and etc) (3) Work related to environmentally-friendly farming (ex. machinery of environmentally-friendly farming and distribution system of production) (4) The other works which conference of support project approve for income increase
	b. Promoting living standard	(1) Work related to promoting medical care (ex. purchase of medical instruments and ambulances) (2) Work related to improving public foundations (ex. an nursing home, a village hall, a street light, playground for children, school bus, bus station and etc) (3) Other work which conference of support project approve for improvement of public foundations
2. Residents Support	a. Enhancing residents' living quality	(1) Work related to support for residents' life (ex. medical examinations, medical insurance aid, transportation expenses support to isolated residents, heating and communication expenses support, assistance of electricity duty, support to environmental farming in flood control area and etc) (2) Other work which conference of support project approve for improvement of public welfare
	b. Education	(1) Work related to support for education (ex. purchase of education materials and books, provision of scholarships and school expenses, meal service facility and expenses support and etc) (2) Other work which conference of support project approve for education
3.Other projects	a. Public relations	(1) Support for local cultural events and the study of the environmental conservation of dam areas, farm machinery repairs, ship operation for isolated area's residents, clearing campaign for dam area and etc (2) Publication of local press, a social gathering, inspection of dam, work for effective operation of support project

'Conference for support project of dam area' and dam manager consider it is necessary. Support project plans must include a clear goal and an outline of the project, quarterly finance plan, an investment plan, a set time period, an contractor, contents of project per individual project, etc.Support project consists of works for regional support, residents support and the other support and these are also divided into increasing income, promoting living standard, enhancing residents' living quality, education and public relations.

Operators differ from a type of project. Increasing income, promoting living standard are conducted by regional government. Enhancing residents' living quality, education, public relations are conducted by the dam management organization or the public trustee of dam management.

The details for support projects follow in Table 3.

As described above, support projects had been conducted through the 'The Specific Multipurpose Dam Act' and the 'Act on Assistance to Electric Power Plants-Neighboring Areas' and these merged into current system after enacting the "Dam Act" in 1999.

The total amount of financial resource is KRW 117,387 million(USD 117 million) in 18 dams from 1990 until now, KRW 42,472 million (USD 42 million) in 2005.

4. Improvement projects for dam areas

At the time when the "Dam Act" was legislated, improvement projects were originally applied only to dams constructed after legislation. However, the government wanted dam construction to be progressed smoothly without any problems, including opposition by regional residents. After enacting the act, residents of dams already completed were very unsatisfied. Due to this dissatisfaction, dams already completed before March 7, 2000 were also benefactors through a revision of the "Dam Act" in December 31, 2001.

As the improvement projects were introduced to surpass the limit of support projects, they have four differences from support projects. First, support projects were conducted annually from after completion of construction until the end of the dam's function, on the contrary, improvement project is conducted even during the construction period. Second, support project is invested annually by small amount of funds while improvement project is invested all together by large amount of money reaching KRW 20~50 billion (USD 20~50 million) in short period. Third, support projects are planned by the dam management organization or dam manager, and carried out by both dam manager and the basic regional government while improvement project is planned by the multi regional government, and mostly managed by the chief of the basic regional government. Lastly, in support project, the amount of earnings earned by sales of electricity and sales of dam water from each individual dam affects the scale of support project of individual dam. It means that financial resource of support depends on the capacity of an individual dam. Compared with it, improvement project emphasizes the scale of dam or damages of incurred from the construction of dam by calculating the scale of the site of the construction or purpose of dam. It is prescribed in the "Dam Act" that the regional government who has jurisdiction over dams which are corresponding to over certain standard should conduct improvement project to promote a regional economy and improve living environment in consideration of the various changes after dam construction. Dams over certain standard mean that its reservoir area is more than 2million m^2 or total reservoir capacity is more than 20 million m^3.

Improvement projects are promoted according to dam area improvement plans that the head of the multi regional government establishes. The boundary of project is within 5km from dam's planned flood level and within 2km from dam's power plant and other areas where the multi regional government recognizes it is necessary for effective operation or balanced development of the region. Support project plans should include a goal and outline of the project, necessary working capital, plans for financial resources, an annual investment plan and the period, operator, contents of project, etc.

Alike as a support project, improvement project is divided into several types, the details of those are as follows Table 4.

The proceeding project's budget as of today is that total working capital of planning and constructing dam are KRW 231 billon (USD 231 million) in 8 dams, total working capital of operating dams are KRW 374.4 billon (USD 374.4 million) in 14 dams.

Figure 2 Promoting living standard

Figure 3 Enhancing residents' living quality

Table 4 Types of improvement project for dam area

Classification	The details of works
Promoting product standard	Improvement of farmland, construction of forest roads and farm roads, collective storage for agricultural, forestry and marine products, facilities of collective cultivation, culture fonds except reservoir
Improving welfare and cultural facility	Medical facilities, cultural assets facilities, schools, libraries, village halls, physical facilities, culture facilities, welfare facilities for elderly people, facilities of communication by wire and radio, resting places, parks, camping grounds, social welfare facilities and etc
Public facility	River maintenance, rental house construction, development of building, roads, bridges, squares, parking lots, water supply, sewerage system, treatment facilities of waste matter, waste disposal plants, perry and port facilities

Figure 4 Facility construction (sewage treatment facility)

Figure 5 Installation of sewage pump

5. Dam area's comprehensive environmentally-friendly restructuring plans

As the purpose of dam construction has mainly focused on solving national water problems, such as water security and floods control until recent years, we have neglected the dam's potential value and functions that could contribute to social and cultural development of a dam region. As a result, people respond to dam construction only in a negative manner and this is regarded as the chief factor causing difficulties in constructing new dams. Thus, KOWACO has endeavored its best effort to minimize the negative image of dam as unwilling on facility from the residents and to make residents consider dam a necessary facility and to help the local development.

Through the revision of the "Dam Act", KOWACO has paved ground that either government or dam manager could preferentially conduct environmentally-friendly project directly as well as in directly such as support or improvement projects and public facilities like road, bridge, water supply, etc. and other necessary project.

As well, the dam manager can raise dam's value as resources much higher by eliminating or reducing difficulties like traffic inconvenience and flooding damage of residents caused by dam construction. Based on the "Dam Act" revision, environmentally-friendly restructuring plans (ex. ecology park, water sports facilities and etc) are under consideration thoroughly within the total contamination control plan to lead local economy active and offer a rest place since 2003. Because dam areas usually locate in several territories, environment-friendly restructuring plan is conducted by each regional government. So overlapping investment or chaotic restructuring measures have been practiced. Therefore

Figure 6 Sample drawing of an environmentally-friendly restructuring plan

Figure 7 Water cultural facility

Figure 8 Restoring place at dam crest

Table 5 Annual environmentally-friendly restructuring plan

Classification	Sum	'02~'03	'03~'04	'04~'05	'05~	Note
Dam	14	Soyanggang dam Andong dam Seomjingang dam	Daecheong dam Chungju dam Juam dam	Imha dam Hapcheon dam Buan dam Boryeong dam	Yeongcheon dam Suo dam Sayeon dam Unmun dam	10 multi purpose dams & 4 water supply dams

comprehensive and systematic development and conservation program are needed.

For this reason, the dam manager makes environmentally-friendly restructuring plans to fit the local characteristics for protection against such a problem and balances between development and conservation and also maximizes the value of dam by offering a leisure place to residents and people.

It has an intention to enhance practical use of dam and to increase effective value of dam. Also, It aims to ease negative recognition through increasing public interest like elimination of damages by dam and providing recreational opportunity.

Environmentally-friendly restructuring plans for existing dams are primarily conducted where development plan is activated and the area is under developed. The annual plan is described in Table 5.

6. The Other Support Projects For Environmental Improvement

6.1 Production of an environmentally-friendly model farm for organic food

Most dam areas are restricted development to maintain water quality by designating the water supply source protection area.

Accordingly residents of dam area face many hardships, like restrictions to property rights and in methods of farming, the major income method. These may affect on the reservoirs as they are contamination sources.

To solve these problems, an environmentally-friendly farm that produces safe organic products is planned in dam areas and hereby conducted to increase the income level of residents and boosting local economy.

3 multi-purpose dam areas, including the Soyanggang Multi-purpose Dam, are operating the farm plan and is going to be expanded to other multi-purpose dam areas in nationwide by the existing model farm's results. Through it, KOWACO tries to harmonize the conservation of water quality with the development of a stable income resource for the residents.

6.2 Conducting welfare improvement projects of the elderly people retiring from public life

In dam areas, the ratio of the elderly people is over 20% and their standard of living is getting worse and worse as the unity of family is getting weak by growth of a nuclear family. So the countermeasure to support them is urgent. As operation of a project about the elderly people among support projects for dam areas construction of welfare centers and its operation program are planned and fulfilled to lead positive recognition of dam.

In the first place, the project is scheduled for the area adjacent to the Hapcheon Multi-purpose Dam and 1,475 the poor and elderly people over 65 years old will benefited. Major programs are consisting of home-visiting ser-

Figure 9 Sign of environmentally-friendly farm

Figure 10 View of environmentally-friendly farm

Figure 11 A Sample plan view of welfare center for the elderly people

vice, day-care service and short term care service.

Home visiting service offers household affairs service, including assistance for meals, bathing, laundry, hair-cut, cooking, cleaning, transportation aid, education and self-reliance support service.

Day-care service offers meals, bathing and group activities at the welfare center to the weakened and disabled elderly people whose family can't afford to send them to welfare center. Its purpose is to reduce the physical and mental burden of a dependent family.

Short term care service offers needed services to weaken and disable elderly people at the welfare center in case that a patron who cares for them can't care in a short term due to a vacation, sickness, overseas business trip.

At the Hapcheon Multi-purpose Dam, the welfare center is planned at an estimated cost of KRW 1 billion (USD 1 million). The welfare improvement project for the elderly people is conducted as a first attempt by a public institution, KOWACO. It is expected that the negative image of dam construction will be improved through the project.

7. Conclusion

In spite of various kinds of systems for residents, support projects at dam areas still do not satisfy the entire residents. An increasing dissatisfaction towards dam at present stems from discontent about support programs together with advancement of residents' awareness.

However, because water resources are scarce in Korea, large scale water resources shortages will occur in the near future. If there were not a thorough analysis of present situation and a long-term countermeasure, we might get into difficulty. The fact is that Korea needs sustainable development by constructing large or mid-sized dams, underground water and exiting dam renewal to secure practical water resources, support and development of dam area must be propelled in accordance with it.

The minority can't be forced to bear the damage any longer by development of water resources for a benefit of the majority and thus there must be a compensation according to it. Proper coordination and consultation are required essentially between a sufferer and a beneficiary caused by dam. More concerns are needed for the development of dam area residents' living environment and support measure more than before. Through the analysis of the social and economic effect to dam area or a self-governing body, we must continuously prepare more advanced solution to solve the problems for upcoming future. It is necessary to expand support methods to develop a local economy and give benefits to dam area's residents preferentially.

Especially in case of exiting dam area, dam's effect on ecosystem and regional development is considered from all aspects. And through leading positive images on existing dam area by the enlargement of support and master plan concerned with symbiosis with existing areas, additional water resource securing plan must be promoted smoothly.

Rehabilitation of Soyanggang Dam in Korea with Construction of Supplementary Tunnel Spillway

Jesung Jeon
Principal researcher, Dam Safety Research Center, KOWACO, Korea

Hangyu Park
Director, Dam Safety Research Center, KOWACO, Korea

Kiyoung Kim
Senior researcher, Dam Safety Research Center, KOWACO, Korea

Youngkwon You
Supervisor, Soyanggang Dam Office, KOWACO, Korea

Abstract

The Soyanggang Multipurpose Dam, completed in 1973, is the biggest central core type rockfill dam in Korea with the gross storage capacity of $2,900\times10^6$ m^3, flood control capacity of 500×10^6 m^3, and flood water level of EL. 198.0 m. However, recent abnormal precipitation, such as the typhoon Rusa, etc., resulted in two great floods close to the design flood water level, which put residents in the vicinity of the dam downstream under extreme uneasiness.

For this reason, a review on the flood control capacity of the dam was performed. Since the design standard modified in 2001 has considered the maximum credible flood instead of floods with certain return periods due to the abnormal precipitation, it is concluded that the current spillway does not have a secure disposal capacity against the maximum credible flood. This conclusion led to this supplementary tunnel spillway plan to improve the long-term flood control capacity. Construction of a supplementary tunnel spillway will be completed in 2007 and it includes environmental improvement near dam site with preparation of various facilities. Rehabilitation project of Soyanggang dam with construction of supplementary tunnel spillway has been under construction since 2004. Due to its inherent characteristic of various field conditions, this tunnel project calls great amount of attention and technical support from experts for the successful completion.

Keywords: *Soyanggang dam, rehabilitation, supplementary tunnel spillway, hydrological stability*

1. Introduction

Soyanggang multi-purpose dam, completed in 1973, is a center core rockfill dam of 123m in height and 530m in length, with the total storage capacity of $2,900\times10^6 m^3$, the flood control capacity of $500\times10^6 m^3$, the water supply capacity of $1,213\times10^6 m^3$/year, the power generation capacity of 353GWh/year, and the design flood level of 198m.

In Korea, an increase of heavy rainfalls in recent years has led to building various plans to investigate the hydrological stability and ensure the safety of existing dams. To reflect the influence of the increased heavy rainfalls, a flood frequency concept has been changed to a probable maximum flood discharge concept in dam designs. Recent records on the damages due to floods in the area near Soyanggang dam indicated that the damage occurred during a year 2002 was about 75% of the total damages occurred for the past 10 years (from 1993 to 2002). The floods occurred in 2002 were "the Lamasoon", a typhoon occurred between July, 5th and July, 6th, heavy rainfalls between August, 4th and August, 11th, and "the Rusa", a typhoon occurred between August, 30th and September, 1th. When the Rusa passed the Soyanggang dam area, precipitation recorded 897mm/day, exceeding PMP (810mm/48hr) of the area. The Rusa brought about the loss of 152 lives and the damage amount of 2,519.3 billions. While the Soyanggang multi-purpose dam was originally designed to be stable for a 1000-year flood, it was revealed to be unsafe hydrologically from the experiences of the recent large floods. Accordingly, for the purpose of increasing hydrological stability of the dam, a plan to build a supplementary spillway of tunnel type was made, and it is currently in construction.

Fig. 1 Soyanggang multi-purpose dam

2. Scope of Work

Besides increasing hydrological stability, the supplementary tunnel spillway is constructed for increasing water supply and power generation amount, building nature-familiarized dam environment, and activating the local economy through an increase in sightseeing demands.

The supplementary spillway was designed as an inclined tunnel type consisting of two parallel tunnels (L1=1,276.4m, L2=1,206.4m). The elevation (height) and width of the overflow weir are EL.185.5m and 58.5m, respectively. The types of gate and hydraulic energy dissipater are a hydraulic radial gate type (B14.7m×H14.0m×4 gates) and a flip bucket type, respectively.

In order to make nature-familiarized dam environment and increase sightseeing demands through it, a public square, a big fountain, a water cultural center, an observation tower, and an ecological park are placed around the dam and the spillway. Near the dam right abutment, the dock and other facilities were renovated by widening the road of 429.7m in length entering the dock and paving it with clay blocks.

The water cultural center in the right abutment is 1,718.63m^3 in area. It consists of various sub-centers, such as the environment conservation institute, the experience science center, the theme science center, and the local cultural center.

For citizens to use the area around the dam as a rest space, the dam right abutment park and the crest are open to public, and in the dam left abutment area, the observation tower of traditional type is placed.

The spoil bank is placed in open cut tunnel area in downstream of the dam left abutment, where the ecological park with the area of 94,910m^3 is built for citizens to have opportunities to experience ecology.

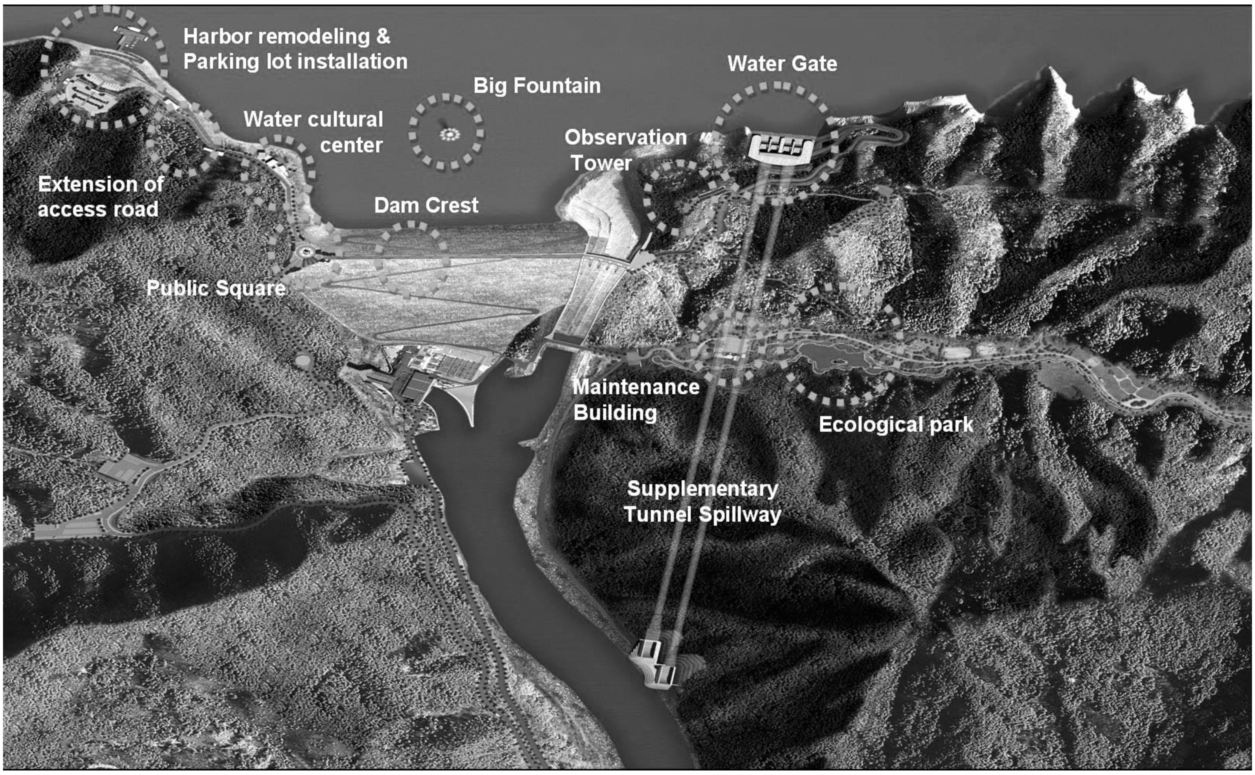


Fig. 2 Planned supplementary tunnel spillway and other facilities

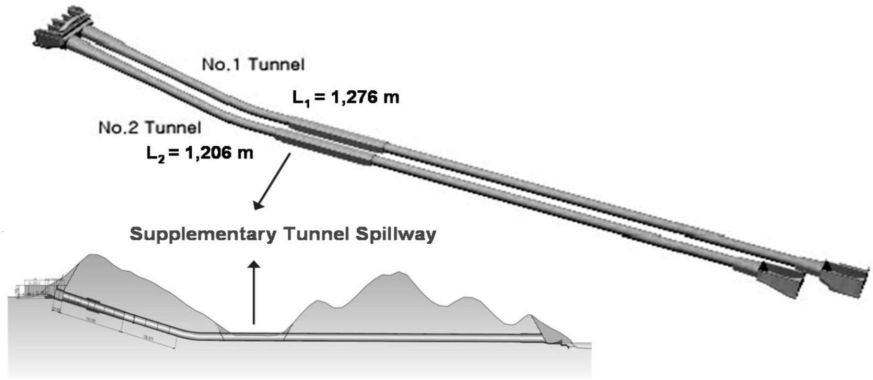
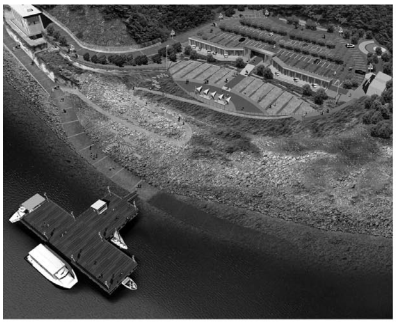
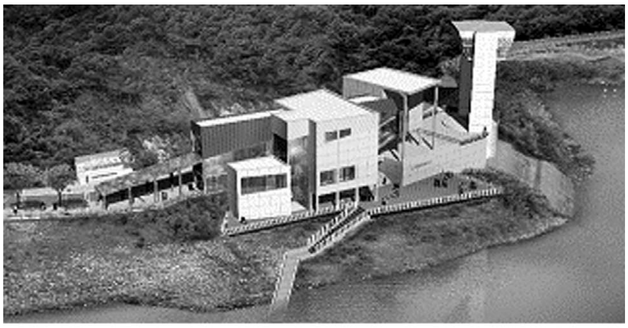


Fig. 3 Schematic diagram of supplementary tunnel spillway



(a) Harbor remodeling



(b) Water cultural center

(c) Public square

(d) Dam crest

(e) Observation tower

(f) Ecological park

Fig. 4 Nature-familiarized dam environment

3. Site Investigation

Geological features

The project is located in the northeast of Kangwon province, whose geological features consist of mainly schist, gneiss and partially silicate and granite. It is examined that a large scale of fold structure found at Gammagol valley (open tunnel section) and several fault zones govern the geological features of the site. Mica schist dominates the upward tunnel and quartz schist does the downward tunnel. A small scale of Jurassic granite and Pre-Cambrian granite gneiss is also observed.

Geotechnical investigation

To obtain detailed design information on planned tunnel lines, field borings, geophysical exploration, and various field tests, including the Grouting injection test, Hydraulic pressure test, Flow velocity test, Borehole image processing test, Pressure meter test, Borehole shear test, and Trial blasting were performed. Also, large-scaled triaxial tests and laboratory soils and rock tests were conducted to determine geotechnical parameters for tunnel design.

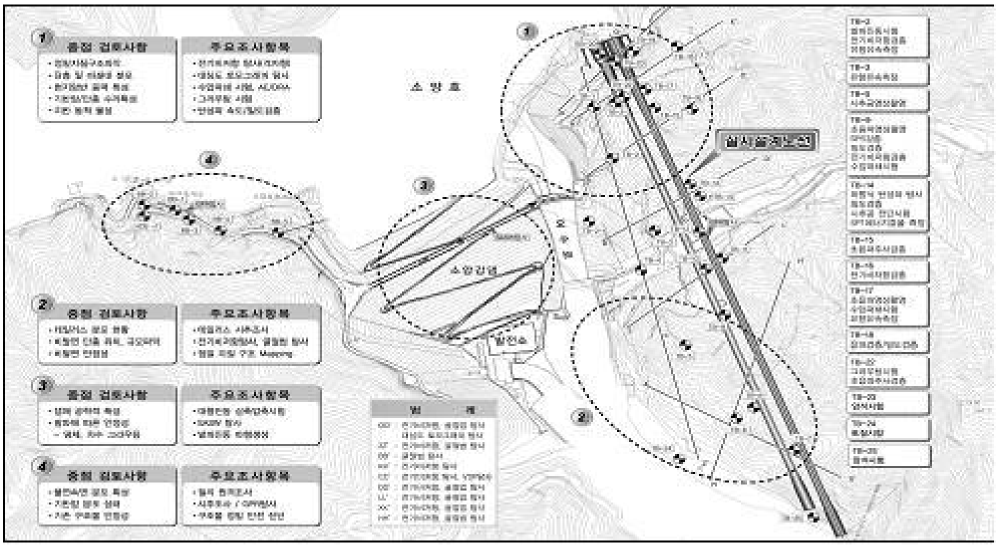


Fig. 5 Outline of geotechnical investigations

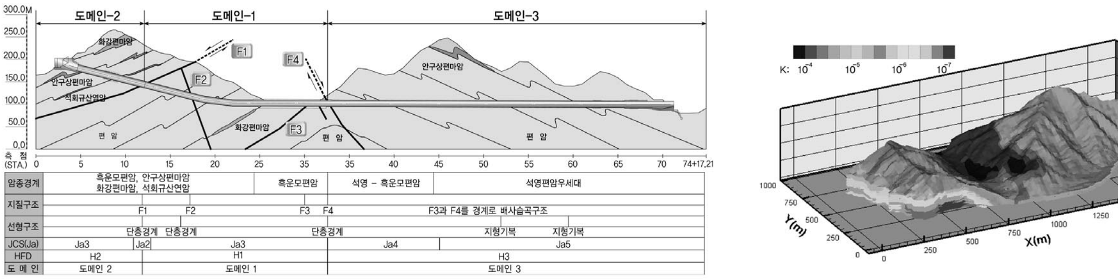


Fig. 6 Geological and hydraulic conductivity characteristics of the ground along the tunnel lines

The ground of the entire tunnel lines was divided into three groups by the characteristics of its geological structures (e.g., discontinuity distribution). For each group, the sections for detailed stability analysis were determined and the design parameters for discontinuities, reflecting the characteristics of rock joint or fault distributions were estimated at each section.

For determining geotechnical design parameters on the tunnel line ground, continuum design parameters were estimated in accordance with the rock mass classifications made for the ground. The rock mass classification for the ground was determined by applying RMR and Q-classification methods. For the ground without boring records, three dimensional rock classification methods and geophysical explorations were used to determine the rock mass classification.

Except the center of inclined upward tunnel section and the entrance of downward tunnel, where faults and fracture zones intersect, overall the tunnel ground corresponds to grade I or II in rock mass classification. Especially, downward tunnel ground was that of the first rock mass rating, being in an extremely favorable condition. This classification based on the rock mass rating was used later in designing tunnel supports considering geological conditions.

The estimation of continuum design parameters was first done by analyzing field and laboratory test results. To improve reliability in the parameters, statistics and evidence theory were applied. Also, Hoek and Brown (2000) method considering rock type, rock condition, and ground disturbance due to excavation was used. Furthermore, MCS (Monte Carlo Simulation) technique was used to consider uncertainties inherent in estimating the parameters.

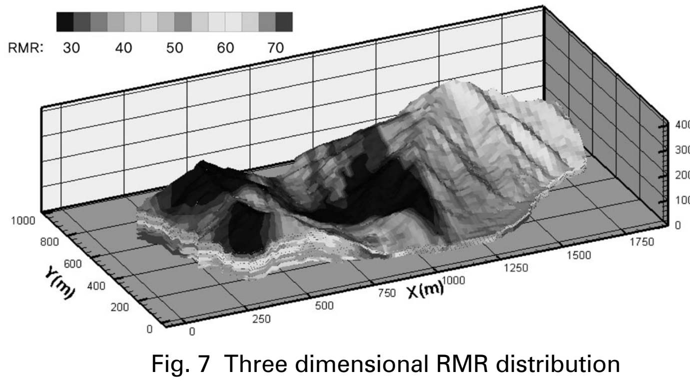


Fig. 7 Three dimensional RMR distribution

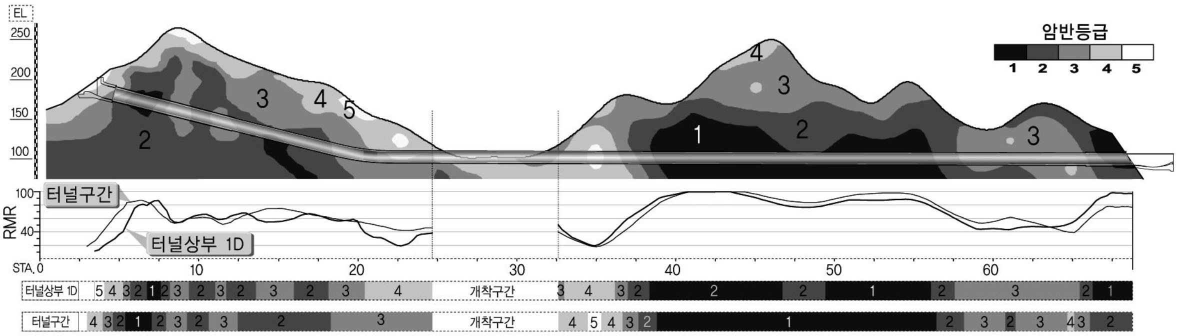


Fig. 8 Rock mass classification of tunnel section ground

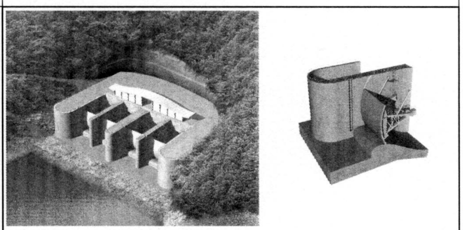
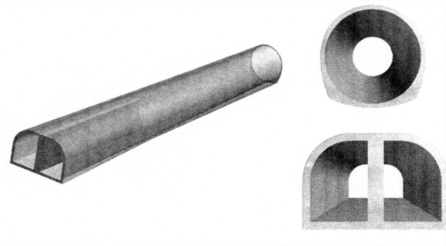
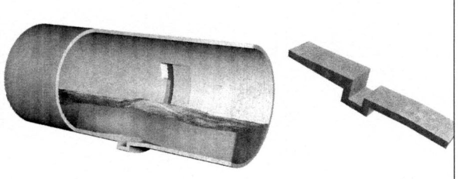
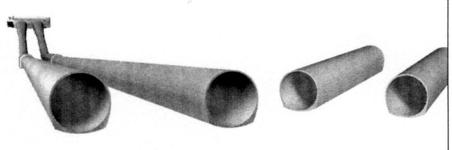
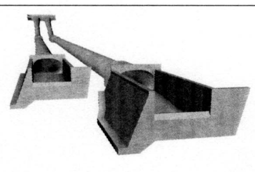
4. Tunnel Design

Main features

Total tunnel sections consist of upward tunnel, downward tunnel, and open-cut tunnel of center area. In order to have hydraulically favorable flow and better constructability, the upward tunnel was designed to be inclined as much as 14 degrees. The upward tunnel is composed of transition zone(circle→rectangular shape), inclined zone, curved zone, and horizontal zone, whereas the downward tunnel horizontal zone and transition zone.

Conveyance area was designed to be smaller than 75% of tunnel area, and a design flow rate was 6,700m³/s. The hydraulic energy dissipater was flip

Table 1 Design outlines

Section	Design outline	Schematic diagram
Approach channel and Overflow zone	• Approach flow velocity is 2.5m/s when 4.0m/s is for 200-year flow velocity • Design head is 15m • Overflow zone width is 58.8m • Overflow zone height corresponds to EL. 185.5m	
Transition zone (Entrance zone)	• Entrance zone is located in inside tunnel to minimize damage to natural environment and to have favorable flow • Transition zone of 155m length is required to have conveyance area smaller than 75% of that of tunnel	
Aerator	• Groove type • Hydraulic analysis using WS77 and TRAJ program (USBR) • Maximum air content : 213m3/s • Air inflow velocity : 36.7m/s • Size : 1.0m×1.0m	
Tunnel zone	• Conveyance area was designed to be smaller than 75% of tunnel area • Upward tunnel was designed to be inclined as much as 14 degrees to have hydraulically favorable flow and better constructability • D=14.0m×2 • Maximum velocity : 38.8m/s (in PMF)	
Hydraulic energy dissipater	• Flip bucket type • Width : 19.4m • Ejection orbit : 68m (in PMF)	

bucket type, and the width of tunnel was planned to be 19.4m by considering tunnel transition and area angle. The aerator was placed in 30m upstream from the curved zone. The outlines of the design of each section of the tunnel spillway are shown in Table 1.

Excavation

In excavating a tunnel, there are many methods depending on the size and shape of tunnel section, tunnel length, and slope. An appropriate method is selected usually by considering economy, stability, constructability, and construction period.

In general, full face excavation or half face excavation method are preferred in terms of constructability and economy. However, the methods have a shortcoming that tunnel face may collapse when the tunnel excavation width is very large or ground condition is poor. In the present project, the tunnel was excavated by dividing the tunnel face into several sections, lowering the face height, and stabilizing the face with split excavations, in order to avoid the collapse of tunnel face during excavation.

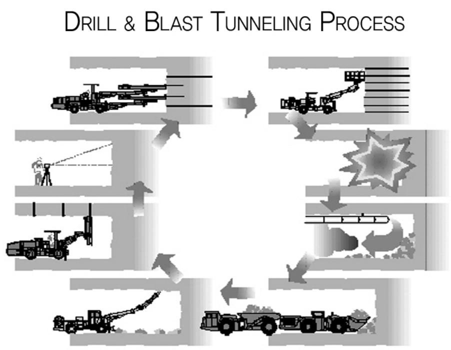


Fig. 9 NATM(D&B) tunnel excavation

Horizontal split excavation section in the tunnel was planned to be divided into two sections, an entrance transition section and a general non-transition section. The excavation in the entrance transition section is done by excavating upper half part of the tunnel face, following pilot tunnel excavations. The lower half part of the tunnel face is then excavated in six steps.

For the general non-transition section, a three-split excavation (or by temporary invert installation) or a ring-cut method were planned to be used depending on ground conditions.

Support systems

Six patterns of tunnel support were used in tunneling based on the rock mass classification results.

Main supports used were shotcrete, rock bolt, and steel supports. The shotcrete is basically wet and wire mesh type. For the entrance tunnel section, however, steel fiber-reinforced shotcrete was used to increase the shear strength of shotcrete and the effect of arching. For the types of rock bolt and steel supports, fully grouted rock bolts using deformed steel bars and lattice girders with high waterproofing capacity and better constructability were used. Besides the main supports, supplementary supports, such as pipe roof, forepoling, steel pipe reinforced multi-step grouting, face shotcrete and rock bolt, and chemical grouting methods, were used to excavate the tunnel in safer and more efficient ways in

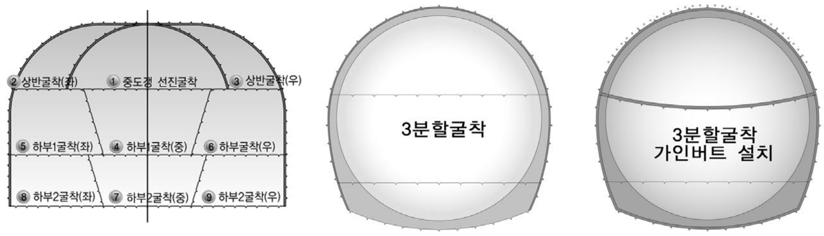


Fig. 10 Tunnel excavation methods with sections

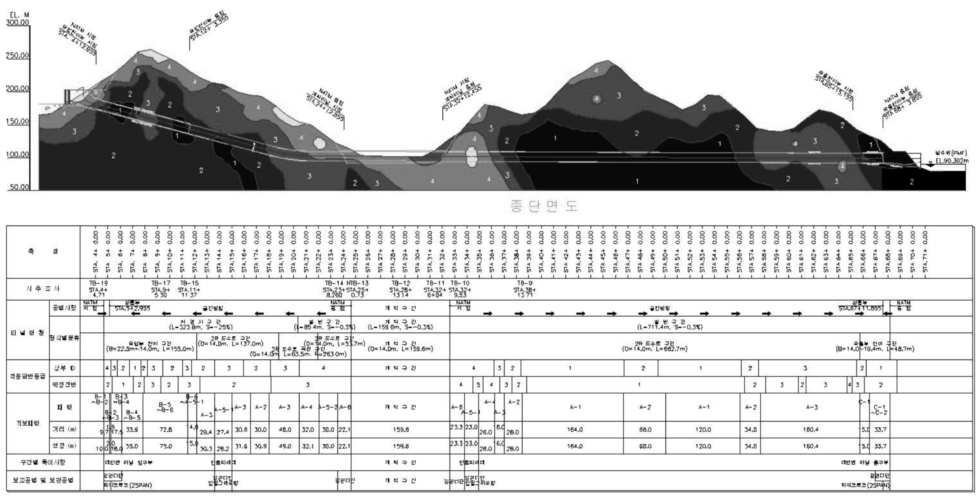


Fig. 11 Tunnel support pattern with sections

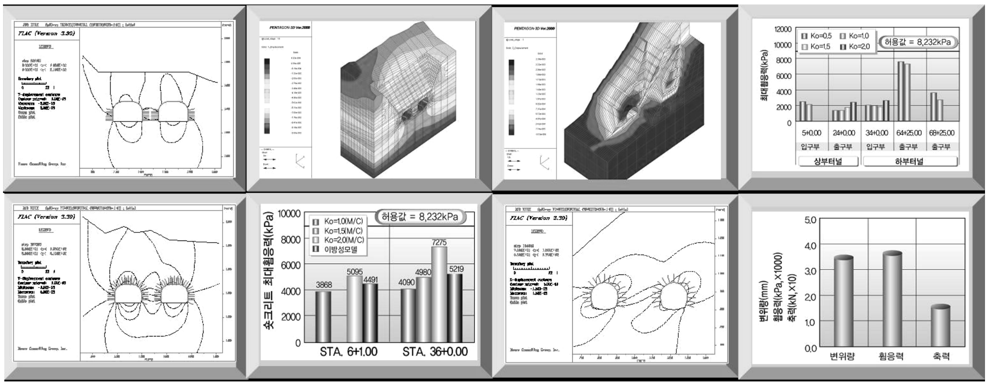


Fig. 12 Tunnel stability analysis

poor ground conditions. Specifically, tunnel face was reinforced by using both shotcrete (t=50mm) and rock bolts to ensure the stability of tunnel face, when the tunnel passes fault or weak zones.

Stability evaluation

The evaluation of tunnel stability was carried out on the general non-transition, entrance, and exit sections of the tunnel by performing the stability analysis using numerical calculations. Furthermore, the effects of anisotropic characteristics of rocks and eccentric earth pressure were investigated in the analysis.

5. Tunnel Construction

In excavating a tunnel, it is the most important to examine the ground response in a current excavation step and reflect it in a next excavation step.

In the present project, the tunnel excavation is basically progressed by following orders: face observation, face mapping, drilling for blasting, blasting and ventilation, mucking, steel support installation, rock bolting (for previous excavation step), construction of 1st shotcrete (for current excavation step) and 2nd shotcreting (for previous excavation step), and a next excavation step.

Considering varying ground conditions and uncertainties in preliminary investigations, geological examinations at every excavation step is the most important step in tunnel excavation.

For every excavation step, the rock condition, rock strength, RQD, groundwater condition, and joint characteristics (i.e., distribution, interval, length, and gouge etc.) are examined, and all the results are recorded in one data sheet. Also, the comprehensive information on tunneling progress, resulted from combining the ground investigation from the design stage and the current investigations for every tunnel face, is provided so that they can be used effectively in determining the excavation method and support pattern for next step.

For the inclined upward tunnel, excavations are progressed by installing the fugitive dust exclusion walls

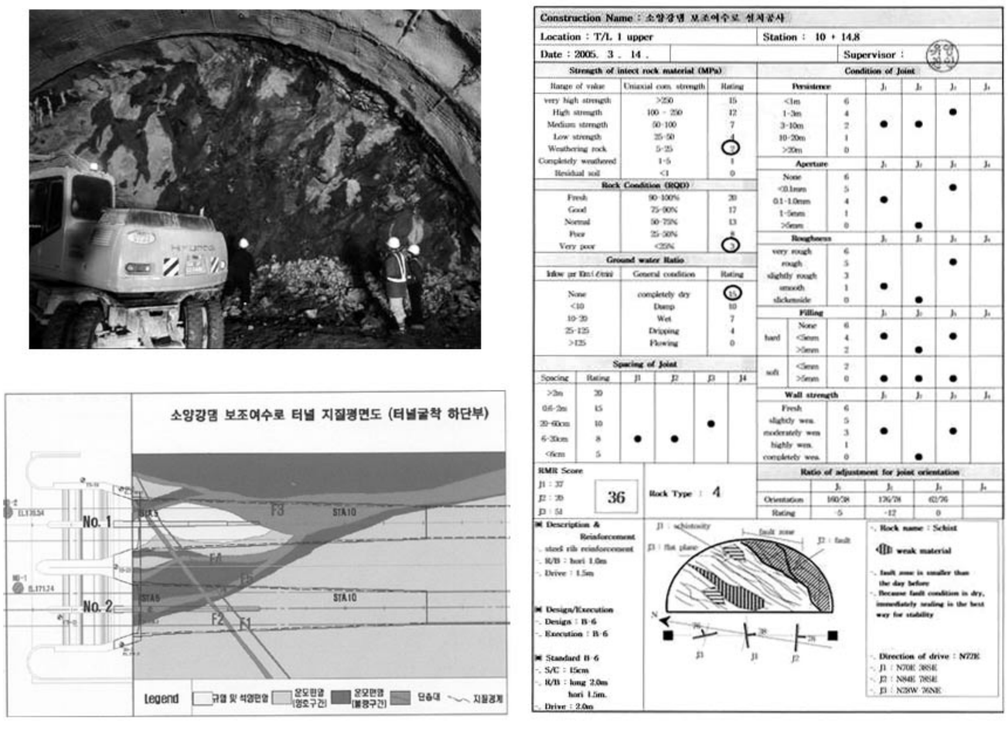


Fig. 13 Tunnel face mapping during excavation and a comprehensive information sheet

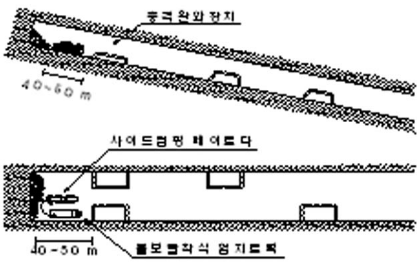


Fig. 14 Installation of fugitive dust exclusion wall

crossing from left and right at 40m behind from blasting locations, in order to minimize the influence of the fugitive dusts during blasting and to save mucking time.

There are two types of monitoring during construction: A-monitoring and B-monitoring. The A-monitoring is performed everyday and includes tunnel investigations, tunnel convergence measurement, crest settlement measurement, and rock bolt pull-out force measurement. On the other hand, the B-monitoring is conducted for more detail monitoring and applied typically in fault zone and tunnel portal. The measurements of shotcrete stress, rock bolt axial force, ground displacement, vibration due to blasting are the examples of the B-monitoring.

The monitoring of tunnel displacement was carried out by applying Austria's Geodetic method. For every excavation step, ground movements were monitored and appropriate support patterns were determined on a basis of the monitoring results.

6. Conclusion

The supplementary tunnel spillway of the Soyanggang dam, currently in construction, is for increasing water supply and power generation amount, building a nature-familiarized dam environment, and activating the local economy through an increase in sightseeing demands. The construction was initiated on August 2004, and now about 32% of the total construction is completed.

The tunnel spillway includes a variety of interesting factors in the construction: large section excavations, presence of transition zone in a tunnel section, presence of an inclined tunnel section, and specific geological conditions etc. The tunnel spillway project, a rehabilitation project of the Soyanggang dam, therefore, is currently spotlighted by both domestic and foreign engineers and scientists.

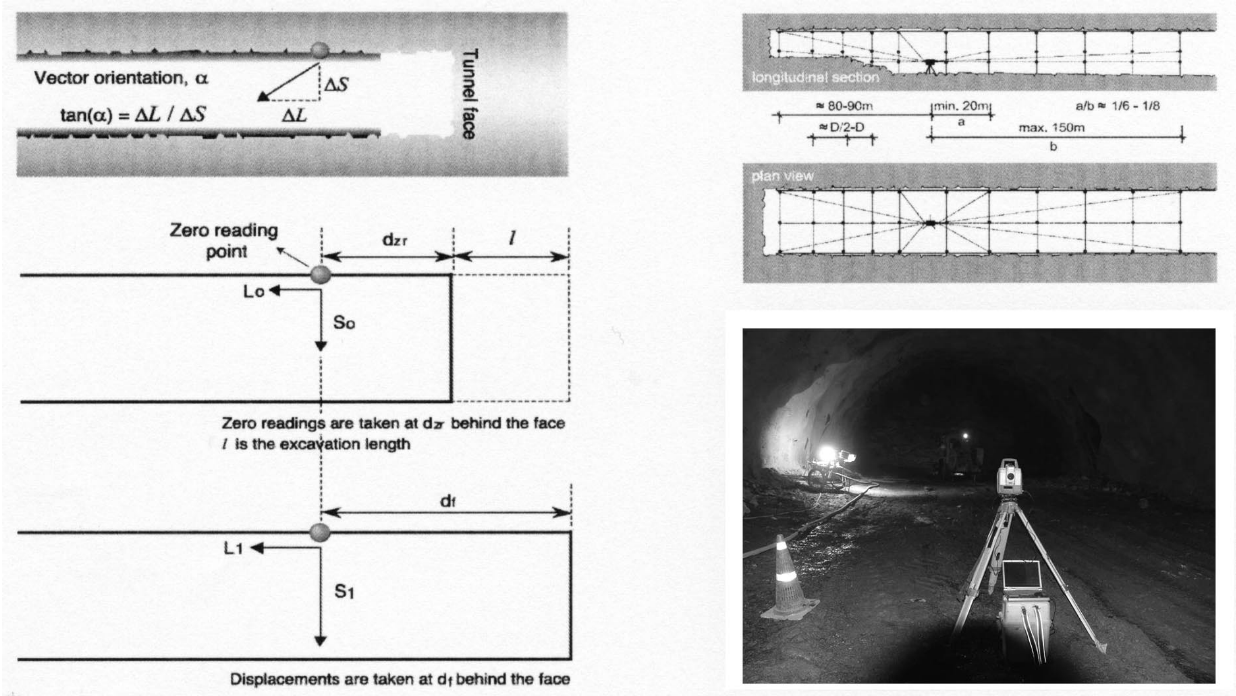


Fig. 15 3-dimensional monitoring and analysis

Fig. 16 Split excavation, downward tunnel portal, exit section

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Study on the Rupture Mechanism along Layers of RCC Arch Dams

Zhang Zhong-qing
College of Civil and Architectural as Engineering, Guangxi University
Nanning, Guangxi, China

Abstract

This paper was written in connection with the non-linear finite-element calculation of Shapai RCC Arch Dam in China, simulated structure model test, and field investigation during demolition of Yantan RCC arch coffer dam in China, for researching the inner structural relationship of RCC and the cause of weak link formation along layers. The weak links are calculated numerically and simulated by test. Cracks take place along layers during overload. and increase with loading. The loading capacity of arch dam decreases until total failure along layers. This is quite different from homogeneous gypsum model test in quality, because the latter fails in 45° direction which can not reflect the special feature of layers. The demolition of Yantan coffer dam verified the rupture mechanism along layers. It is concluded that "heat joint" placement and putting cushions between layers is important in increasing shearing and tensile strength of layers.

Keywords: *RCC arch dams, rupture mechanism, layers*

For conventional concrete, gypsum model is generally used in simulation model test. Gypsum is a brittle material and can be treated as homogeneous. It is similar to concrete. Most cracks are perpendicular to dam foundation or inclined to horizontal surface. The writer in undertaking the project "Study on loading capacity of high RCC arch dams" used prototype material to simulate the arch dam, thus avoided the complicated transformation of material simulation and achieved direct perceptivity through senses. It is firstly used in China. The construction technique was simulated and broke the hypothesis on homogeneity of dam body. RCC layers were considered and the rupture was along layers. Calculation on non-linear finite-element by ANSYS achieved similar result. It is quite different from conventional concrete. Prototype investigation on the demolition of Yantan arch coffer dam proved this kind of rupture.

Fig. 1. Shapai RCC Arch Dam in normal operation

1. The inner-structural relationship of RCC arch dam

The RCC arch dam is compacted by layers of 30cm thick. The treatment of layer joints affects its bonding condition. The so called "hot joint" is to place successive layers before initial setting of the former concrete, there fore time interval of stop is short. Aggregates of concrete between layers are mutually inlaid. This can be proved from drilled cores. No "hot joint" can be seen. This may be approximately regarded as homogeneous. Intervals of stop are longer than final setting. Even though the layer surface is well treated and coated with cement-flyash mortar, the aggregates are not mutually inlaid. and a weak link is then formed. For treatment after initial setting by placing layer cushion, the bonding requirement can be met, but relatively speaking, the non-homogenuity is a bit bigger than "hot joint".

As to hot joint and those joint with layer cushion placed within allowable interval, concealed layer structure is easily formed and affects the bonding strength. This is due to such causes: segregation of aggregate after mixing during transport and spreading; pores formed in the process of RCC compaction obstruct dewatering and weaken the bonding, moisture condition of underlying concrete surface does not meet the construction requirement, too large or too small consistency Vc, crowding unloading, unsuitable thickness of spreading, insufficient power of vibrating compaction, as well as density less than required. All above factors are causes of weakness. For suitable Vc, covering layers before initial setting, and formation of bitting of aggregate of successive layer into the underlying, the layer surface is invisible.

32 test pieces (150×150×150mm) are prepared in two groups for cleavage test. The first group is cloven along 1/2 horizontal surface of test pieces, and the second group perpendicular. Tensile strength of the first group is 10-12% less than that of the second one with few only 5-6%. The second group represents the real performance of RCC which is heterogeneous.

2. Simulation of layer sucface

With no inlaied aggregate on the layer surface of cold joints and those joints coated with layer cushion spread within allowable time, the tensile and shearing strength are lower than the concrete itself. The tensile strength is 20%. Simulation calculation for Shapai arch dam is made by 10 m layers according to construction condition. The ANSYS program is tremendous. Number of joints should be over 20 thousands to simulate the layer condition. For overload calculation only self weight and water pressure are considered. Temperature load is not considered in calculation.

Due to unsymmetry of both banks, radial displacements (δr) under design water pressure for both banks are unsymmetrical. Displacement of right half arch is bigger than the left max radial displacement 32.08mm takes place on the upstream side of top crown a little to the right bank. Distance from the crown is 10-15m, approximately. Fig. 2 shows the upstream radial displacements (brackets show figures of No.1 model test value.)

While water pressure increases to 2.82 times of the design value with constant self-weight, the arch dam loses its loading capacity. The peripheral cracks penetrate the dam and main cracks take place on the horizontal surface of the middle dam body. Fig. 3 shows the main cracks on upstream and downstream sides of the arch dam (it is impossible to figure out cracks after rupture, because the program can not give convergent solution when the arch is unable to sustain load). It is visible from the figure, that layer surfaces on downstream side are wholly opened before failure, and layer cracks at bottom elevation of arch dam, 1760.0m~1800.0m nearly pierce through the dam, above elevation 1770.0m near the springing cracks penetrate nearly to the crown, the right springing is badly cracked. At 1.5 times of overload, the surface cracks at 1760.0m~1800.0m are mainly observed at the springing. Above elev. 1800m beside the springing the downstream crown is also partially cracked. When water pressure increases to 2.8 times, the surface cracks develop from downstream to upstream and from springing to crown until total fail are.

3. Overload test and rupture characteristics

The Shapai arch dam in China completed in 2002 is 132m. high, of which the arch dam is 117.5m high, arc-height ratio 2.13, thickness-height ratio 0.238, max radius R=224.35m. It is 3-center circular arch. 2-grade C20, 3-grade C20 are used. Model ratio 1:80, crown thickness 0.12m, bottom thickness 0.35m, arch length 3.13m. Simulated layer thickness 0.10m, time interval 4 hr. The first model was made in sep.4 1998 and the second one in June 3, 2000. The load was applied by eccentric jack with water pressure load. The simplified self weight load was applied at dam crest.

3.1 The second model

Stress distribution and radial displacement under 2.0 times of design water pressure.

Arch stress σ_y: upstream max. σ_y=–10.87MPa, it decreases towards both banks until totally compressive. On the downstream side, it is nearly compressive, max. occurs at both springing, σ_y=–12.66MPa, stresses at the middle are affected by transverse joints shaped like a saddle.

Cantilever stress σ_z: At dam toe on the upstream side is mainly tensile, max tensil σ_z= 3.56MPa (actual ε = 137 $\mu\varepsilon$), max compressive σ_z=3.17 MPa; Stresses on the downstream side are mainly tensile, max tensile σ_z =1.45 MPa.

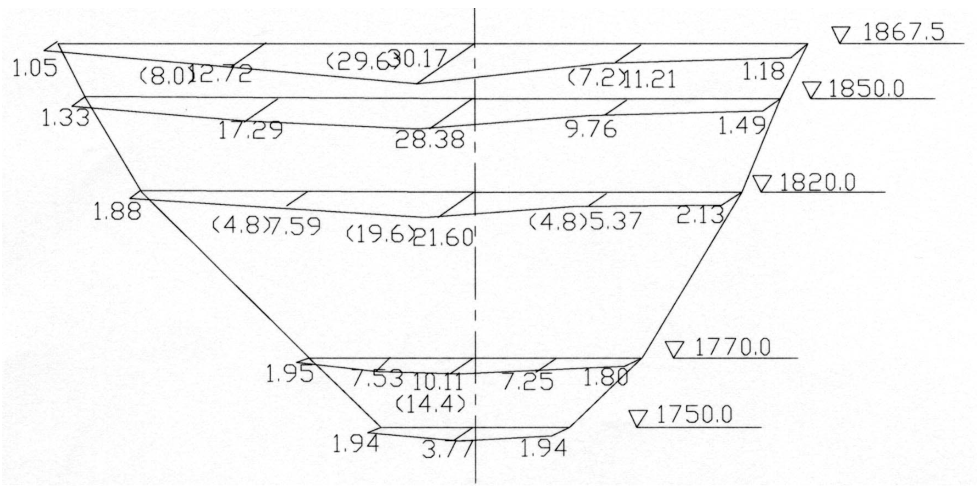

Fig. 2. Diagram of radial displacement under designed water pressure (in mm)

Max. radial displacement occurs at arch crown, diminishes gradually towards both banks, and also towards dam bottom. Stagger was observed at two transverse joints. Max. ridial stagger 6.0mm. Gap of right transverse joint along the arch 3.2mm.

Stress distribution and radial displacement under 2.7 times of design water pressure.

Arch stress σ_y: max. arch stress σ_y =–12.74MPa takes place at 1830.0m on the upstream side and diminishes towards both banks. All are compressive. Compression on the right is bigger than the left. All downstream arch stresses are compression, and max. value occurs at both ends, σ_y =–17.78MPa, Stresses at the middle shape like a saddle.

Cantilever stress σ_z: max. tensile on the upstream side σ_z =3.69MPa (actual $\varepsilon = 142\mu\varepsilon$), most downstream stresses are tensile, its max. σ_y = 7.64 MPa (actual $\varepsilon = 293\mu\varepsilon$).

Max. radial displacement occurs at crown ε_r = 224.8 mm and diminishes towards both banks and decreases from crest to bottom. Max radial stagger at the transvere joint is 13.6mm. Gap of the transverse joint along the arch on right bank is 8.0mm.

3.2 Rupture characteristics

For the first model, while water pressure increases to 2.5 times of the design value, at elev. 1786.70m and 1798.70m, cracks along horizontal compacted layers are observed and extend quickly towards the left up to the left bank. At elev. 1827.50 cracks start at the right banks, extend close to the crown, afterwards turn vertically to the dam crest. Stagger between layers is 3mm for model, 240mm for prototype. While the radial displacement increases from 159.2mm to 207.2mm, loading cannot be continued and the structure loses its loading capacity. Main cracks of the arch dam after rupture shows in Fig. 4.

For the second model, while water pressure increases to 2.7 times of the design value, the structure loses loading capacity. Rupture condition is similar to the first model.

In gypsum or other homogeneous body model, rupture takes place mainly along 45°, i.e. perpendicular to the bank slope, because max. principal tensile stress takes place here. Several intersected destructive cracks are observed. RCC is layer structure and its layer surface is weak. Special feature of simulated model is the simulation of weak links for layer surfaces. Cracks start on the weak layer surfaces. Rupture shown in Fig. 3 reflects the case. In the acceptance session for state nineth five-year main scientifico-technical key projects, specialists confirmed the reasonability of this rupture type, which reflects the structural relationship of concrete layers.

It should be noted that the tensile stress bigger than 2.2 MPa is unreasonable, because tensile stress bigger than the tensile strength will induce softening of σ-ε curve, resulting in decrease of σ and increase of ε. For convenience, $\sigma = E\varepsilon$ is still used and E is kept constant. Then the above result appears.

The writer now undertaking the similated structure model test of Wan Jia Kouz RCC arch dam in China under constrution comes to the same conclusion of rupture along layers.

4. Demolition of Yantan coffer-dam.

The downstream coffer dam of Yantan has radius R=124.84m, central angle 60˚. It was completed in May 1988 and demolished in 1991. During construction there was a stop for every 1.8-2.4m for moulding. The interval was one or several days. Cold joints took place in the dam body due to some causes. Observation for such cold joints after demolition shows big area of "smooth plate" on the joints without any unevenness. No inlaid aggregate was found on the surface. Area of such surface was as big as several tens of square meters. Tensile and shearing strength on the surface was remarkably reduced and would easily lead to crack.

Different rupture types are shown in Fig. 4. A shows stepped cracks, B shows stepped "smooth plate" which has rather bad cohesion. C shows sheer

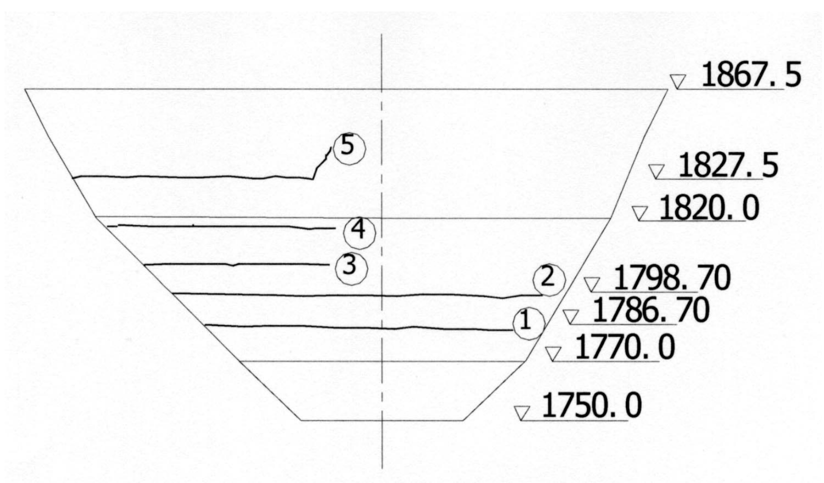

Fig. 3. Rupture characteristics

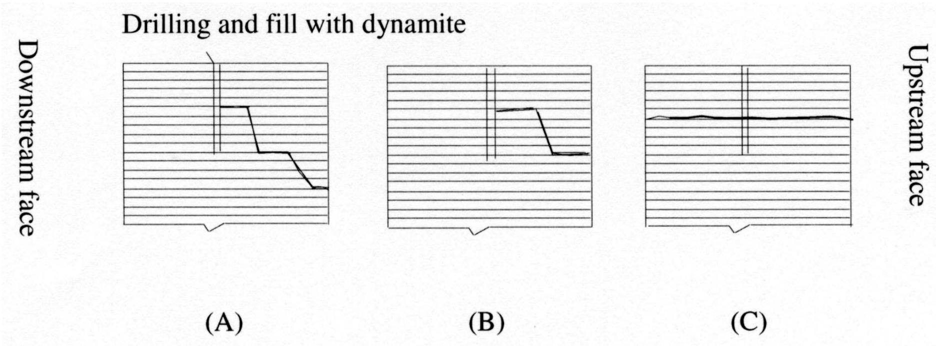

Fig. 4. Sketch showing cracks due to blasting

smooth plate. In case the boring hole was located in 1/2 of the dam body, crack protrude to outer surface, i.e. to upstream side in radial direction. When quality of the upstream side is better than the downstream side, individual crack might lead to downstream side. Fig. 4 shows the inhomogeneity of the same surface. Viewing on the plane, cracks was funnel-shaped instead of straight lines. When boring hole was closely located on a straight line, crack could be shaped close to a straight line. It should be mentioned that cement consumption for Yantan coffer dam, a short-life structure, was low and quality control was worse the, above result was consequently inevitable.

5. Conclusion

By means of non-linear calculation for RCC arch dam, simulated structure model test and in situ investigation on demolition of Yantan coffer dam, it is noticed that during RCC arch dam construction many weak links exist due to different causes. Therefore overload rupture start on the layer surface, which results in failure along such surface. It is different from the rupture governed by factor of safety, It is beyond the specification's requirement, and has substantial meaning for design and construction technique.

Measure for the Hydrologic Dam Safety Considering Revised Probable Maximum Precipitation

Ick Hwan Ko, Ph.D
Director, Hydrosystems Engineering Center,
Korea Institute of Water and Environment,
KOWACO, Daejeon, Korea

Boosik Kang, Ph.D
Principal Researcher, Hydrosystems Engineering Center,
Korea Institute of Water and Environment,
KOWACO, Daejeon, Korea

Abstract

The Soyanggang Dam is a multipurpose dam for flood control, water supply and hydropower and is located 80km east of Seoul. The dam is located in a narrow valley of the Soyang-gang, 12km upstream of the confluence with the North Han River. The dam consists of a rockfill embankment 123.0m high and the crest length is 203.0m. Construction began in 1967, reservoir filling began in 1972 and the project was placed into operation in 1973. A gated open chute spillway with 5 radial gates (13m wide and 13m high) is located in the left abutment. The catchment area is 2,703km^2. The reservoir has a gross storage capacity of 2,900×10^{6}m^3 The initial spillway was designed for a 200-year event corresponding to inflow of 10,500m^3/sec. With a spillway discharge of 5,500m^3/sec (no downstream flood damage), the maximum water surface would reach EL. 198.0m. The initial extreme event had a return period of 1,000 years with an inflow of 12,392m^3/sec. With a discharge of 7,500m^3/sec, the maximum water surface would be EL. 200.4m. The dam has experienced extraordinarily big storm events in 1984 and in 1990, which exceeded the original spillway design inflow of 10,500m^3/sec. In response to these events, KOWACO (Korea Water Resources Corporation) has reviewed and updated PMF several times in 1995, 1997 and 2001. The final PMP and PMF were selected as 810mm and 20,712m^3/sec, respectively. As the permanent measure, the methods of increasing the discharge consist of modifications to the existing discharge facilities and spillway, providing an auxiliary spillway in the left abutment located in the existing spillway. As a temporary measure, parapet wall and lowering restricted water level were proposed.

1. Introduction

The Soyanggang Dam was designed for 1000 years return period according to the design criteria at the time ('67～'73) of construction. In the mean time, the design criteria standard has been raised to PMF (Probable Maximum Flood) owing to the recent extraordinary storm events which happened twice in 1984 and 1990 when the reservoir water level was close to design flood level. Therefore, the reasonable measures for flood were necessary to be prepared. As the measures for enhancing flood control capacity, the parapet wall and emergency spillway were proposed for the temporary measure and permanent measure, respectively.

Recently the Soyanggang Dam watershed experienced another record-high intensive storm from the typhoon "RUSA" in August 30, 2002. Based on the recent large-scale storms, the PMP and PMF were computed again based on the new procedure and basic plan for the emergency spillway design for optimal size, location, type, etc, has been revised.

2. General Description of Existing Reservoir and Spillway

The Soyanggang Dam, which was constructed about 30 years ago, have 12,392m^3/sec as the maximum inflow discharge of design flood which is equivalent to 1000 years of return period flood. For the design of spillway, the inflow discharge of 10,500m^3/sec was used which is equivalent to 200 years of return period flood. The design release discharge through the spillway is 5,500m^3/sec at flood water level (EL. 198.0m). The fundamental configuration of the Soyanggang Dam is following;

- Catchment Area: 2,703km^2
- Dam Type: Rock Fill Dam
- Dam Crest: EL. 203.0m
- Height: 123.0m
- Flood Water Level: EL. 198.0m
- Normal Water Level: EL. 193.5m
- Restricted Water Level during Flood Season: EL. 190.3m
- Reservoir Capacity: 2,900×10^{6}m^3

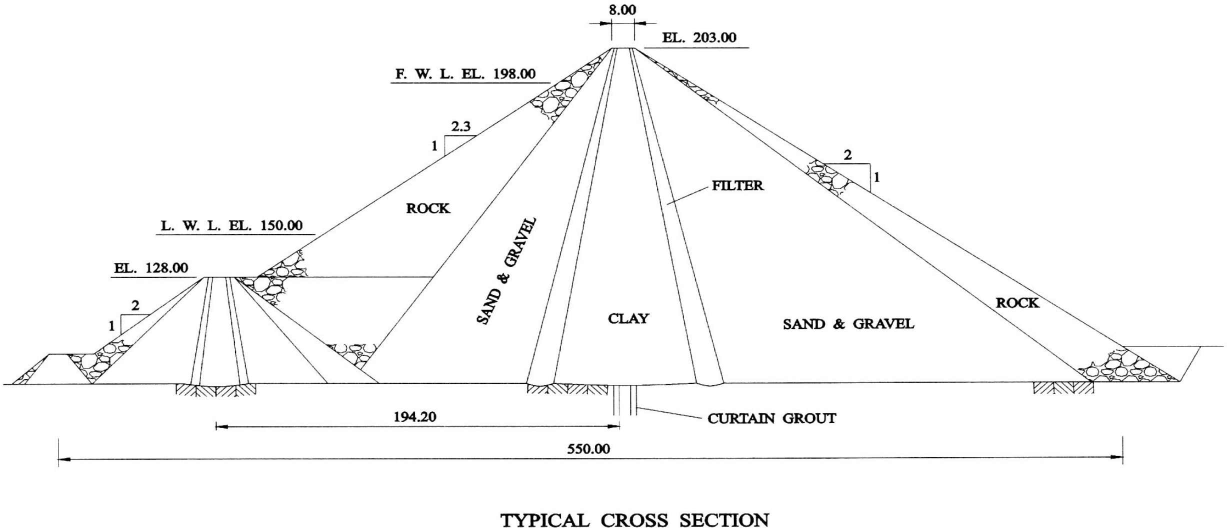

Figure 1 Typical Cross Section of Soyang-gang dam

3. Review of Hydrological Safety

3.1 Hydrological Study

30 years of operational data of the Soyanggang Dam shows that the design inflow was underestimated. Because dam failure may cause serious damage downstream, it is very important to secure dam safety from an extreme flood such as PMF. Since the Soyanggang Dam, which is located upstream of the Han River, has experienced several significant floods recently (Table 1), it is required to increase the flood control capacity of the Dam in order to secure dam safety from an increased design flood, which was recently estimated again according to the new procedure (Table 2).

3.2 Hydrological Safety Analysis

The flood control scale of the Soyanggang dam was designed with a peak inflow return period of 1,000 years. The selection of 690mm for a 48 hour duration as the PMP for the Soyang-gang Dam in was based on of Hydrological Safety Analysis (1992) and Hydrological re-design (1995). These analyses used the three flood events of 1984, 1987, 1990. The same data was used in the Feasibility Study (1996), Basic Design (1997) and Detailed Design (2001) plans. The PMP and PMF in those studies are shown in Table 2. Table 3 summarizes the precipitation of various return periods.

Table 1 Significant flood events that occurred in 1984 and 1990

Design factor	Design Flood for Spillway (200yrs)	Observed Inflow Discharges during the Significant Floods	
		Sep, 1984	Sep, 1990
Inflow Discharge	10,500㎥/sec	11,995㎥/sec	10,653㎥/sec
Highest Water Level(EL.m)	EL.198m	EL.197.79m	EL.197.99m

Table 2 PMPs and PMFs of Soyanggang Dam in Previous Studies

Category		Design	Study 1*	Study 2**	Study 3***
Precipitation (mm)	200yrs	499.9	530.5	530.5	509.7
	PMP	631.9(1000yrs)	690.0	690.0	690.0
Inflow (㎥/sec)	200yrs	10,500	13,373	15,166	12,489
	PMF	12,392(1000yrs)	17,468	20,906	17,258
Highest Water Level (EL.m)	Restricted W.L	190.30	190.30	187.00	**187.00**
	PMF	200.40(1000yrs)	202.66	201.34	**201.11**

*Study 1 : Feasibility Study on enhancing Flood Control Capacity(1996)
**Study 2 : Nippon Koei(Japan) Analysis(1997)
***Study 3 : Analysis(2001)

Table 3 The results of previous studies (2 days peak probability rainfall, mm)

return period (year)		Design	Study 1*	Study 2**	Study 3***	Study 4****	
100		446.1		473.7	473.7	456.5	
200		499.9		530.5	530.5	509.7	
1000		**631.9**		662.1	662.1	632.7	
PMP	**2day**		690.0	**690.0**	**690.0**	**696.5**	**810.0**
	3day		**760.0**	760.0			910.0

* Study 1 : Hydrological Stability Analysis (1992)
** Study 2 : Feasibility Study on enhancing Flood Control Capacity (1995)
*** Study 3 : Nippon Koei (Japan) Analysis (1997)
**** Study 4 : Analysis (2001)

When using 1999's heavy rainfall from Cheolwon province, the PMP of the Soyangang Dam was estimated at 810mm/2 days. The center of heavy rainfall was located along the border of North and South Korea. It is the result of DAD analysis that only uses rainfall data for the South Korea side. PMF is estimated at 20,712m³/s according to PMP of 810mm/2 days. It should be noted that the PMF in the study of 2001 greatly exceeds the value of the previous studies Table 4.

Probability analysis using the data from before (1958~1968, 11 years) and after (1974~1998, 25 years) dam construction is shown in Table 5.

If the maximum outflow is restricted to 5,500m³/sec, the reservoir water level reaches EL.198.98m at peak discharge (Qp=12,489CMS) of the 200 year probable flood. (0.98m higher than flood water level EL.198.0m). In such

Table 4 Flood probability comparison

return period (year)	2 day peak							
	Design		Study 1*		Study 2**		Study 3***	
	rainfall (㎜)	peak outflow (㎥/sec)	rainfall (㎜)	peak outflow (㎥/sec)	rainfall (㎜)	peak outflow (㎥/sec)	rainfall (㎜)	peak outflow (㎥/sec)
50	394.0	8,200	416.7	10,437	-	-	403.2	9,716
100	446.1	9,400	473.7	11,909	-	-	456.5	11,100
200	**499.9**	**10,500**	**530.5**	**13,373**	**530.5**	**15,166**	**509.7**	**12,489**
500	573.7	12,100	605.4	15,298	-	-	579.8	14,325
1000	631.9	12,392	662.1	16,753	-	-	632.7	15,726
PMP	**631.9**	**12,392**	**690.0**	**17,468**	**690.0**	**20,906**	**690.0**	**17,258**
							810.0	**20,712**

* Study 1 : Feasibility Study on enhancing Flood Control Capacity (1995)
** Study 2 : Nippon Koei (Japan) Analysis (1997)
*** Study 3 : Analysis (2001)

Table 5 Comparison of flood probability to actual flood events using verified measurement

(Unit : ㎥/sec)

Return period (year)	Design	Result of the analysis using observed data	Study 1*		Study 2**	
			Inflow	Compare with observed data (%)	Inflow	Compare with observed data (%)
50	8,240	10,010	10,437	1.2	9,716	-2.93
100	9,360	11,356	11,909	1.7	11,100	-2.25
200	10,500	12,698	13,373	2.8	12,489	-1.64
500	12,090	14,467	15,298	2.4	14,325	-0.98
1000	12,392	15,805	16,753	2.6	15,726	-0.50
PMP	-	-	17,468	-	17,258	-
					20,712	

* Study 1 : Feasibility Study on enhancing Flood Control Capacity (1995)
** Study 2 : Analysis (2001)

Table 6 Result of the Reservoir Routing for each study

Classifications		Design	Study 1*	Study 2**	Study 3***	
Probability Rainfall (㎜)	200year	499.9	530.5	530.5	509.7	
	PMP	631.9	690.0	690.0	690.0	810.0
Inflow discharge (㎥/s)	200year	10,500	13,373	15,166	12,489	
	PMF	12,392	17,468	20,906	17,258	20,712
Outflow discharge (㎥/s)	200year	5,500	6,953 (5,500)	6,269 (5,500)	6,132 (5,500)	
	PMF	7,500	9,301	8,986	8,384	11,219
Highest W.L	200year	EL.198.0m	EL.199.80m (200.42)	EL.198.91m (199.13)	EL.198.71m (198.89)	
	PMF	EL.200.4m	EL.202.66m	EL.202.29m	EL.201.91m	EL.203.94m

* Study 1 : Feasibility Study on enhancing Flood Control Capacity (1995)
** Study 2 : Nippon Koei (Japan) Analysis (1997)
*** Study 3 : Analysis (2001)

a case, we are worry about damage upstream of the Soyanggang dam due to inundation. When the flood regulations do not limit discharge downstream of the Dam, the highest water level at the PMF (690mm) is EL.201.91m (1.51m higher than the extreme water level of EL.200.4m). If any case, the highest water level must be lower than the EL.200.40m. When we apply 810mm for PMP, the highest water level is estimated as EL.203.94m at restricted water level EL.190.3m. It is 3.54m greater than compared with peak flood water level EL.200.4m taken from the design specs. When reservoir operations lowered the restricted water level from EL.190.3m to EL.185.5m, the highest water level is estimated at EL.202.93m (Table 6).

4. Measures for Enhancing Dam Ssafety and Flood Control Capacity

We have tried to develop solutions for enhancing dam safety and the flood control capacity of the Dam through consultation with numerous experts. The proposed solutions include as following;

- Lowering the restricted water level during flood season
- Installation of an auxiliary spillway (e.g. a tunnel)
- Increasing the existing dam's height
- Construction of a new dam upstream of the Dam
- Installation of Parapet wall on the Dam Crest

Table 7 Summary of results in evaluations for each plan

Plan	Evaluation	Summary
Adjustment of restricted water level during flood season	Extra effluent facilities are needed, upper stream alternative dam is required as substitutes for water management and power generation losses	N/A (Not Accepted)
Extra emergency spillway construction	Actually we need extra solutions for the downstream damage, but PMF inflow flood damage = natural disaster ≠ man's disaster, so extra solutions are to be predicted	**Accepted**
Heightening of existing dam	This is a good solution as it is profitable and a decrease of water use benefit will not occur, but local appeals deriving from additional compensation and difficulties in securing an absolute construction period are the key problems to construction	**Accepted**
Construction of a new dam at the upstream of the Dam	It is not economic and has been finally cancelled due to resistance from NGOs and local residents	N/A
Installation of parapet wall on the Dam crest	If we construct this along with modifying restricted water levels, then there is a possibility for implementation. It is not suitable for a permanent solution, but can be a practical solution	**Accepted**

After consulting with national and international experts, we came up with the conclusion that installation of a parapet wall on the Dam crest should be implemented, along with the adjustment of restricted water levels during flood season. This could be the best solution both economically and technologically. However, it's urgent that we think about other alternatives and reexamine the existing research because installation of a parapet wall on the Dam crest and the adjustment of restricted water levels is not a permanent solution.

Although the construction of a new flood control dam was selected as the best solution, the plan was rejected by local residents. So, we considered installing an auxiliary spillway (such as tunnel) or enlarging existing spillway, as the method for enhancing dam safety and the flood control capacity.

To enhance the flood control capacity of the Soyanggang Dam, we examined the alternatives given by previous studies and evaluation results for each plan as following Table 7.

5. Conclusion and Development Plan

The methods to increasing the storage in order of cost are changing the project operation plan to lower the reservoir prior and during the season of highest risk, raising the dam and constructing an upstream reservoir. The methods of increasing the discharge consist of modifications to the existing discharge facilities and spillway, providing an auxiliary spillway in the left abutment located in the existing spillway. Finally, auxiliary spillway is selected as best solution for increasing the discharge capacity. According to this result, we will finish the detailed design until April, 2004 and undertake the construction of auxiliary spillway in June, 2004 and finish this works until December, 2006. The summary of construction works are as follows;

- Outline of the Project
  - —Total Required Cost : 133 million $
    - – Construction Cost : 118 million $
    - – Compensation Cost : 1.5 million $
    - – Management Cost and etc. : 13.5 million $
  - —Construction Period: April, 2003 – December, 2006

- Outline of Auxiliary Spillway
  - —Type: Tunnel type spillway with a gentle slope
  - —Maximum discharge: 6,700m³/s (existing spillway 7,500m³/s)
  - —Gate Type: Radial Gate (B14.7m x H14.013m x 4 each)
  - —Size of Tunnel: D14m x 2 lines (L1=1,280.2m, L2=1,200.4m)
  - —Flip Bucket Type

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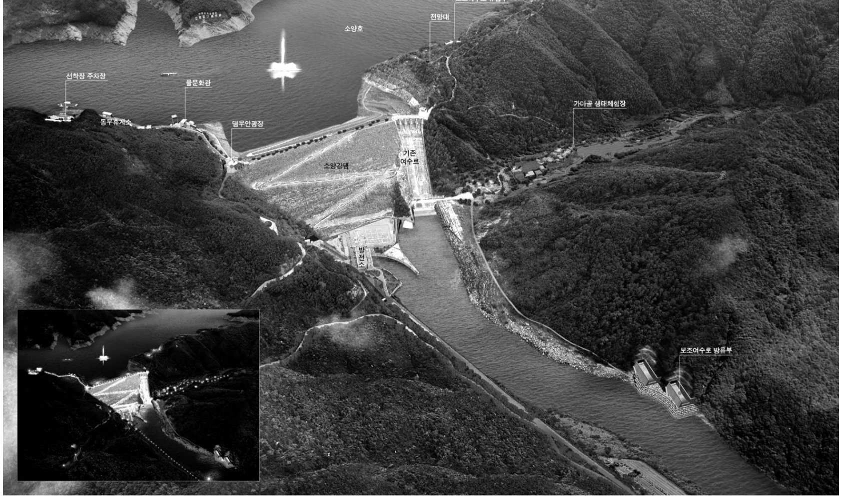

Figure 2 Bird's-eye view of Soyanggang Dam after construction of auxiliary spillway

The Utilization Efficiency on Fish Conservation Facilities in Lake Tamjin

Dong-Sup Kim
Head Researcher, Environmental Research Center, KRRI, KOWACO, Daejeon, Korea

Jinwon Seo
Senior Researcher, Environmental Research Center, KRRI, KOWACO, Daejeon, Korea

Chong Seo Hwang
Stream Corridor Restoration Laboratory, Suwon, Korea

Gilson Hwang
Water Environment Research Center, Kunpo, Korea

Abstract

Tamjin Dam is built in the upper reaches of the Tamjin River which flows through the Janghunh-gun and Gangjin-gun of the South Jeolla Province. In order to map out a preservation strategy of the fish ecology in consideration of the impact the dam construction will have on the fish habitat environment. Fish conservation Facilities are (1)3 fishways constructed in Tamjin river, (2)stream-type spawning ground, (3)fishway on dam, (4)floating spawning ground within reservoir, (5)habitat for *Anguilla japonika*. There was a total 36 fish species inhabition the upper and lower reaches of the Tamjin river. 3 Fishways constructed in the Tamjin river are pool type ice-harbor fishway and the fishway slope is 1:20. Total 28 fish species were utilized fishways, and utilization periods were from early march to early December on the condition of water temperature over 10°C. Fish migration time in a day was largest around the sunset and almost all fishes migrated through the both edge part in fishway. Crab, snail and larva of dragonfly was also utilized the fishway. The results of this study suggested that fishways constructed in Tamjin river is a good role for fish migration. And more study was needed on the subject of the relationship between fish size and swimming ability, and migration mechanism in the fishway.

Keywords: *Lake Tamjin, Fishway, Fish Conservation Facilities*

1. Introduction

With the development of civilization, demand for water grew rapidly, this making the stable supply of water a critical challenge for mankind. The supply of water relies primarily on precipitation. Korea's precipitation pattern does not follow that of a typical temperate climate but that of a tropical monsoon climate, which means that precipitation is concentrated in the summer rainy season. During the other seasons, precipitation is very unstable and scarce.

In such a region, the stable supply of water has become an important challenge. In order to meet this challenge, reservoirs have been blocked in rivers. However, blocking reservoirs for the supply of water resulted in the interruption of the migration path of migratory fish. According to the 1976 Fisheries Resources Protection Act, fish paths were established in reservoirs, which sparked the study of fishway (Agricultural and Fisheries Promotion Corporation, 2000 ; Park, 1998 ; Hwang and Kim, 1991 ; Hwang, 1996). With the execution of the economic development plan, increase in population, industrialization, urbanization led to the great jump in the demand for water. As a result, large dams were constructed in rivers to facilitate the supply of water. Because most of the dams in Korea are built in mountainous regions, both banks are steep in slope. So if there is a change in water level, they are immediately exposed to air. This implies that the surrounding water edges which can be used as spawning ground are lost. A number of dams were constructed in the Youngsan River and Sumjin River in the South Jeolla Province to supply water, but the problem water shortage in the southwestern part of the South Jeolla Province remained unsolved. As part of an effort to provide stable supply of residential and industrial water in this region, a large-scale dam towering 54m tall was built in the Tamjin River which flows through Janghunh-gun and Gangjin-gun.

Under Korea's precipitation pattern, blocking a flowing river with a dam to supply water would lead to an excess of inflow over outflow of water during the spring when the spawning period, drought and supply of agricultural water overlaps. Because the water level fluctuates severely in this period and the water edges that can function as a spawning ground for fish are exposed to air, they cannot provide ground for fish to lay their eggs. Therefore, there arises a need to build an artificial spawning ground for the preservation of fish in the reservoir. There is also a need to secure a habitant and spawning ground for the fish that live in the upper and lower reaches from the dam and ensure they do not lose their mobility.

The Tamjin Dam, being constructed on the upper reaches of the Tamjin River, is located about 28 km north of the river estuary. If the downstream environment has a natural river environment, migratory fish coming upstream from the ocean can migrate to the dam. This thesis will assess the impact of the Tamjin Dam construction on the region's fish ecology and its living environment. It also aims to map out measures to preserve fish as the environment changes from a flowing water environment to a static one.

In this study, We conducted utilization efficiency on 4 fishways constructed in lower dams of Tamjin river.

2. Description of Study Site

The Tamjin River is a mid-sized river that originates from South Jeolla Province Youngam-gun Gumjung-myon Seryu-ri Gungsung-san and flows into the Doam Bay of Gangjin-gun. The surrounding environment of the river is shown on Table 1. The 54m-high Tamjin Dam is being constructed on the border of the Janghung-gun Yuchi-myon and Janghung-gun Busan-myon. The specifics of the dam are shown on Table 2.

3. Plan of Fish Conservation Facilities

Changes in Fish Habitat Environment after Construction of Tamjin Dam: If a flowing river is blocked with a dam, the depth of the filled water increases greatly, making extinct habitat for fish that lived in the shallow and fast current environment. After desalination, due to large fluctuations in water level the reservoir will lose its function as a spawning ground and will not be able to grow aquatic plant. The water edges will be exposed as naked ground. In the upper stream of the reservoir, the habitat of nonmigratory fish will be reduced. The construction of the dam in the lower stream may contribute to the habitat of fish since it will secure adequate water level. However, the Tamjin Dam, located near the estuary, may block the movement of migratory fish which move to and fro between the river and ocean.

Table 1 Tamjin River Drainage Condition

Origin	O Jeollamado Yeongam-gun Geumjung-myun Seryu-ri
Estuary	O Jeollanamdo Gangjin-gun Gangjin-eub mog-ri
Afflux Sea	O Doam bay, South Sea
Inflow Stream	O Yuchi river : flow from eastern slope of mt. whalseong, Yeongam-geun, Geumjung-myun and join at Songjeong-ri, Yuchi-myun, Jangheung-gun O Omcheon river : flow from southern slope of mt. whalseong, Yeongam-gun, Geumjung-myun and join at Dai-ri, Yuchi-myun Jangheung-gun) O Geumgang river : flow from southern slope of mt. Wolchul Yeongam-gun and join at Songam-ri, Jangheung-eub, Jangheung-gun)
Drainage-basin	O Yeongam-gun : Geumjung-myun, yeongam-eub, O Jangheung-gun : Yuchi-myun, Busan-myun, Jangheung-eub, Jangdong-myun O Gangjin-gun : Omchun-myun, Jagchun-myun, Seongjun-myun, Gundong-myun.
The end of Tamjin River	O Boundary line of Mog-ri, Gangjin-eub and Gundong-myun, Gangjin-gun.

Table 2 Data of Tamjin Dam

Locality : Jeonnam Jangheung-gun, Busan-myun, Jichun-ri.

Name of River	: Tamjin River
Draiage area	: 193.0 ㎢
Highest High Water Level	: EL. 84.0 m (in PMF)
High water Level	: EL. 82.8 m (200years frequency)
Normal High water Level	: EL. 82.0 m
Low Water Level	: EL. 55.0 m
Elevation of the Peak of dam	: EL. 85.0 m
Length of dam	: 403 m
Dam height	: 54m

Selection of Species to be Protected in Tamjin River: Migratory species to be protected are *Plecoglossus altivelis* and *Anguilla japonica*, and the *Coreoperca kawamebari* among protected species designated by the Environment Ministry. There is a need to secure habitat for fish that live in still water such as minnow, *Oryzias latipes*, *Acheilognathus yamatsutae*, *Sarcocheilichthys variegatus*, *Microphysogobio yaluensis*, *Iksookimia hugowolfeldi*, *Odontobutis platycephala*. Also, once the dam is built and the water is filled, the *Crucian carp* and *Cyprinus carpio*, which are expected to constitute the main fish ecology, may lose their breeding ground because aquatic plant would not grow due to fluctuations in water level. This also requires solutions.

Protection Measures of Main Fish Species: *Plecoglossus altivelis*, a migratory fish, can hardly migrate under natural conditions because of the 16 beams between the estuary and the Tamjin Dam. (It is possible to migrate when the water level is high enough to exceed the height of beams.) Therefore, as an immediate measure, the number of fish can be increased by landlocking the area and releasing fry into the reservoir and river after the building of the dam. However, it would be better to create a long-term natural migratory environment by building a fishway that would allow *Plecoglossus altivelis* to migrate since the beams in the lower stream will be repaired and improved. In addition, since almost all elver is caught in the estuary and it is realistically difficult to expect *Anguilla japonica* to migrate naturally due to the 16 beams, it would be wise to create a fish resource environment by releasing fry into the reservoir and river and fostering a habitat. *Coreoperca kawamebari*, protected species by the Environment Ministry, inhabits a wide area of the Tamjin River and is not migratory. So it is judged that the already secured habitat will be enough for this species, but a creation of a habitat in the lower stream would attract more fish and then they can be induced into the reservoir through the fishway. In the upper stream, the reduced

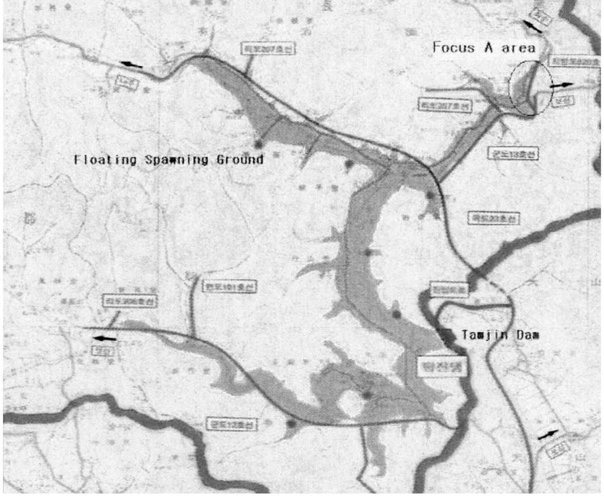

Fig. 1 Plan chart for building fish preservation facility in Tamjin Dam

habitat can be made up in the inflowing mouth of the river. Mountain torrent fish such as *Zacco platypus* and *Oryzias latipes*, cannot move because of the beams established in the upstream where the dam is to be constructed. Therefore, the habitat and spawning ground of this species should be fostered in the upper stream of the dam reservoir and a fishway should be built so that partial migration can take place. The *Crucian carp* and *Cyprinus carpio*, freshwater fish, are expected to increase in the reservoir, but it would be difficult to naturally create a spawning ground due to fluctuations in water level. Therefore an artificial fixture needs to be built in the mouth of the reservoir and near the habitat within the reservoir as a floating natural spawning facility.

Creating a Habitat and Spawning Ground in Mouth of River (Figure 1; Focus A Area): The mouth of the river always showed large fluctuations in water level. The area which was used as agricultural land before the dam construction was exposed as dry land in seasons of less precipitation, and water flowed only in the river channel. Therefore, there is a need to build a facility that can be used as spawning ground in the river channel. The river channel is divided into a riffle zone and a pool zone based on the water flow. Because spawning usually takes place in the pool zone where the water flow is not very fierce, the existing reservoir can be used to maintain the pool zone. A fish path shall be built in the reservoir that is high enough to block the migration of fish. The area selected for this research is the upper reaches of the Tamjin River's mainstream, located in Yongmun-ri. Currently, the movement of fish is blocked by a concrete beams but since the water level is low the water edges are well-developed. A habitat and spawning ground for *Coreoperca kawamebari*, *Zacco platypus*, *Oryzias latipes*, *Plecoglossus altivelis*, *Crucian carp*, *Cyprinus carpio* should be created in this region. The existing concrete beam shall have a multi-tiered fishway, enabling fish to move freely. The aquatic plant zone around the water should be expanded, providing habitat and spawning ground. A floating spawning ground should be built in the deep water, thereby inducing *Coreoperca kawamebari* to breed there.

Floating Natural Spawning Ground within Reservoir (Figure 2): Except for a limited region where the inflowing water mixes with the reservoir, a strip of dry land surround the reservoir. Therefore, fish that spawn in deep water have no where to breed. Only when the water level rises to the aquatic plant zone on the water edges after the summer rain can they lay eggs. Even then, if the eggs do not have enough time to hatch before the water level falls, the eggs are wasted. Therefore, a floating natural spawning ground should be created for the maintenance of this fish ecology. Although this facility would be built mainly for the *crucian carp*, *Cyprinus carpio*, and *Coreoperca kawamebari*, it aims to accomodate more diverse fish. The

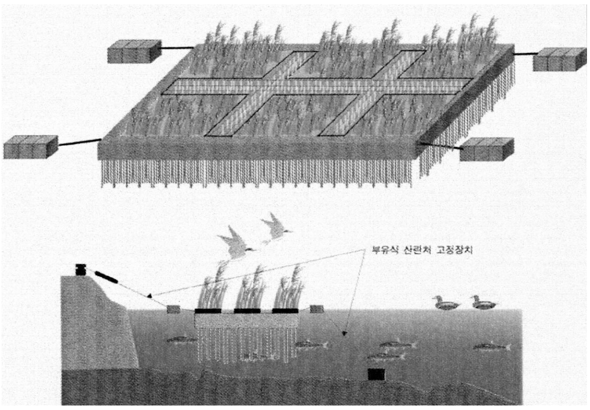

Fig. 2 Concept chart for floating natural spawning ground facility within reservoir

appropriate location for the facility is on the slope within the reservoir where the slant is steep. It should be built deep enough so that it will not be exposed to the air even when the water level is at its lowest. Also in consideration of changes in water level, a pulley should be built into the facility and it should maintain its balance in the wind. For maintenance, the buoyancy should be so that 1~2 people can operate it. A summary concept of the spawning ground facility is demonstrated in Figure 2. A device on which eggs can stick should be attached to the facility's lower part (in the water). On the top, aquatic plant should be let grow so that fish habitat and a shelter for fry can be created. Also, this would have an additional effect of improving water quality by having the plant absorb nutrients in the water.

Habitat for *Anguilla japonica* (Figure 3): *Anguilla japonica* usually hide out under rocks, in caves or in mud during the day and move around at nights and eat almost all aquatic animals such as shrimp, crabs, aquatic insect, worms and small fish.

After the construction of the dam, an organic matter that is even softer than mud will accumulate under the water, making it a very inappropriate place for *Anguilla japonica* to hide out. That is why there is a need to make an appropriate place for *Anguilla japonica* to hide out. This thesis recommends that a pile of rocks be placed in the river channel as a habitat for *Anguilla japonica*. The rocks should be about 30~40cm big, the rock pile being 2 m wide and 1 m high. The water depth of the location should be about 10 m.

Artificial Waterway-style Habitat & Spawning Ground in Lower Stream of the Dam (Figure 4): In order to supplement the habitat and spawning ground of fish that disappears with the building of the dam, a waterway-style habitat and spawning ground should be built using part of the water discharged from power plants into the lower streams. The waterway-style habitat and spawning ground should create a diverse environment so that diverse fish could live and lay fish there. *Crucian carp*, *Acheilognathus yamatsutae*, *Pungtungia Herzi*, *Squalidus gracilis*, *Hemibarbus longirostris*, *Pseudogobio esocinus*, *Zacco temmincki*, *Zacco platypus*, *Misgurns anguilicaudatus (loach)*, *Plecoglossus altivelis* should be allowed to inhabit the artificial waterway. The *crucian carp* lives in area where the water flow is slow or where there is a lot of aquatic plant. *Acheilognathus yamatsutae* inhabit rivers where there

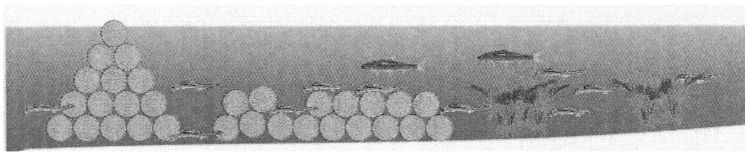

Fig. 3 Concept chart of habitat for *Anguilla japonica*

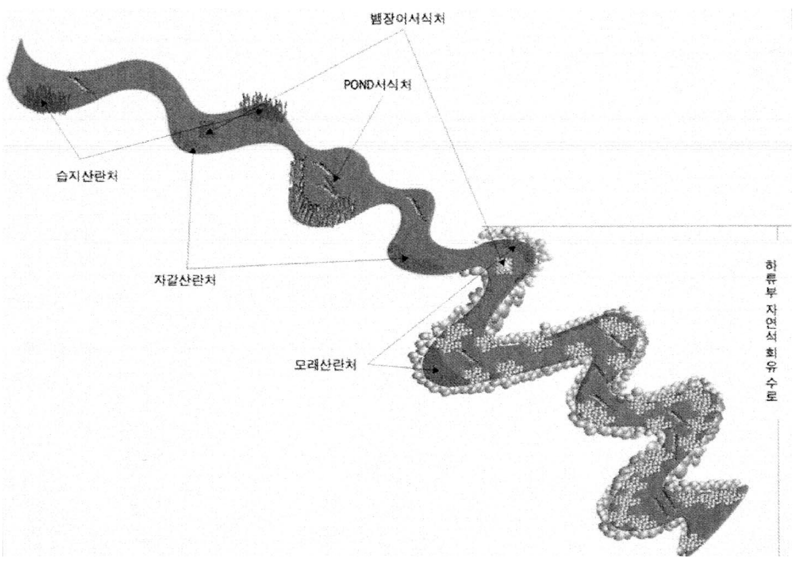

Fig. 4 Concept chart for waterway-style habitat & spawning ground facility in lower stream of dam

is a lot of aquatic plant and shellfish must be placed in the spawning ground for they are a prerequisite for laying eggs. *Pungtungia herzi* live in areas where the water flow is light and where there is gravel on the bottom. *Squalidus gracilis* inhabit the surface or the middle level of the water where the flow is slow. *Hermibarbus longirostris* live in waters with gravel on the bottom and *Pseudogobis esocinus* live on sand. *Microphysogobio yaluensis* usually live stuck to the gravel where the water flow is slow. The *Zacco temmincki* also inhabit waters where the flow is not fierce. Minnows inhabit swift waters, loach stagnant waters, *Plecoglossus altivelis* water where there is gravel or stone on the riverbed.

The artificial waterway must alternate between swift currents and still water. Around the marshes where the water is still, the area of silt/clay and of small rocks should be reconciled so that water plants can grow. The depth of the water should be about 1m, and the water flow should be so that the bottom gravel will not wash away. The bottom gravel field should have the right mix of big and small pebbles.

Fishway in Dam and stream (Picture 5, 6): Building a fishway into a large dam requires an enormous investment and advanced technology (Terakin *et.al.*, 1996). Therefore, it is impossible to build a general fishways vertical slot and denil fishway) into a dam such as the Tamjin Dam, which is 54 meters high and has a 27 meter fluctuation in water levels. In addition, the Borland fishway cannot be built into the Tamjin Dam since it is a rockfill dam. This thesis proposes an operational fishway which is low-cost and highly efficient for the migration of fish. Part of the discharged water from power plants can be used for attracting fish, and a fish-luring facility can be installed so that fish caught for a certain period of time can be transported to the upper streams of the reservoir by a catch box, which will be operated like a cable car. This method is effective in that the fish catching time can be adjusted according to the amount of fish in the river, and can catch diverse types of fish regardless of their swimming or jumping ability. The fact that it can be used only when the migratory fish is migrating is also a benefit of this method. And 3 Fishways are constructed in upper and down stream of the Tamjin Dam. Stream fishways will be attributed to move the stream fish between downstream and reservoir, and between reservoir and upstream.

Fig. 5 One of the Pool type Ice-harbor fishway constructed in downstream of the Tamjin Dam

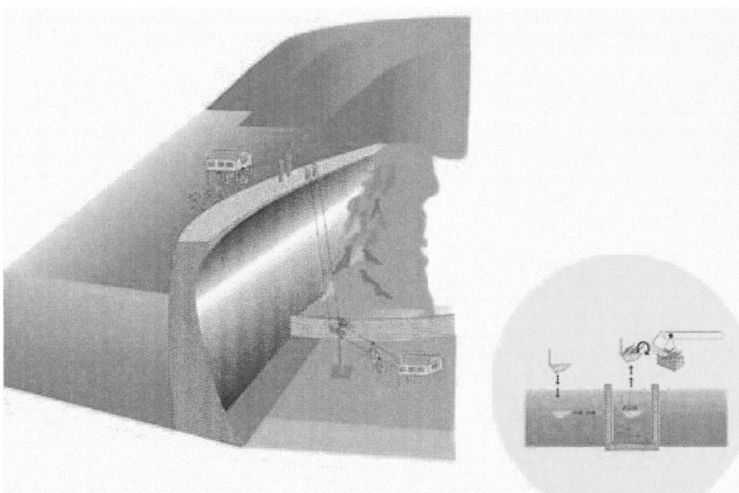

Fig. 6 Concept chart of operational fishway built into dam

4. Materials and Methods

4.1 Investigated fishways

We investigated the usage of fishways at Tamjin River to predict the number and population of fish species that will use the Tamjin Dam fishway after installation.

- Fishways at the downstream: Knock-down Ice Harbor Fishway at Shimcheon-bo and Ice Harbor Fishway at Eoin-bo
- Fishways at the upstream: Knock-down Ice Harbor Fishway of Hyunam-bo and Saddle-bo

4.2 Fish Collection method

As indicated in the Figure 8, we made traps (width 1.0m × length 1.0m × height 0.5, and mesh size 5mm) and installed them at the exit of the fishways (the upstream). We collected all fishes and analyzed the results.

We classified the collected fish on the site. We measured the length of fish by a digital micrometer and took pictures of the fish. After that, and we released the fish to the upstream of the fishways.

We investigated fish migration at Shimcheon-bo by the following time schedule. First, we arrived at the site at noon and installed the traps at six exits of the fishways. Then, we took away the traps every three hours (3PM, 6PM, 9PM) and collected fishes. In the next day, we also took away the traps every three hours (6AM, 9AM, 12PM) and collected fishes. It took about 30 minutes to take away all the traps and install the traps again.

We classified and identified the fish followed by the system of 'Korean freshwater fish (written by Kim, ik soo)'.

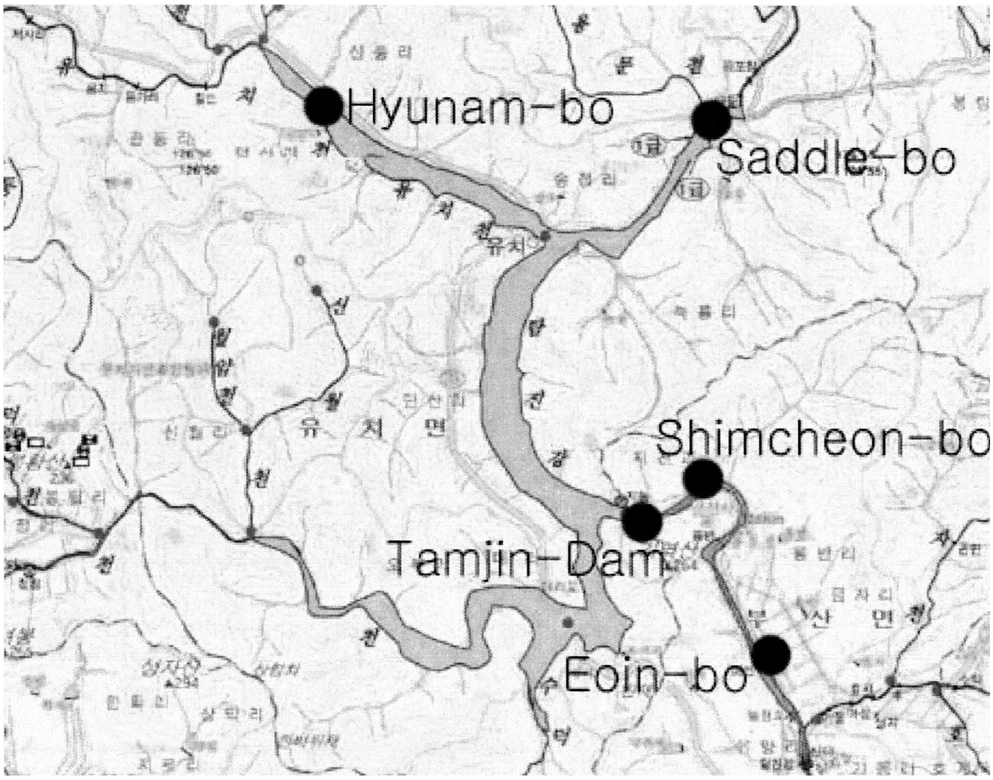

Fig. 7 Location of the fishways

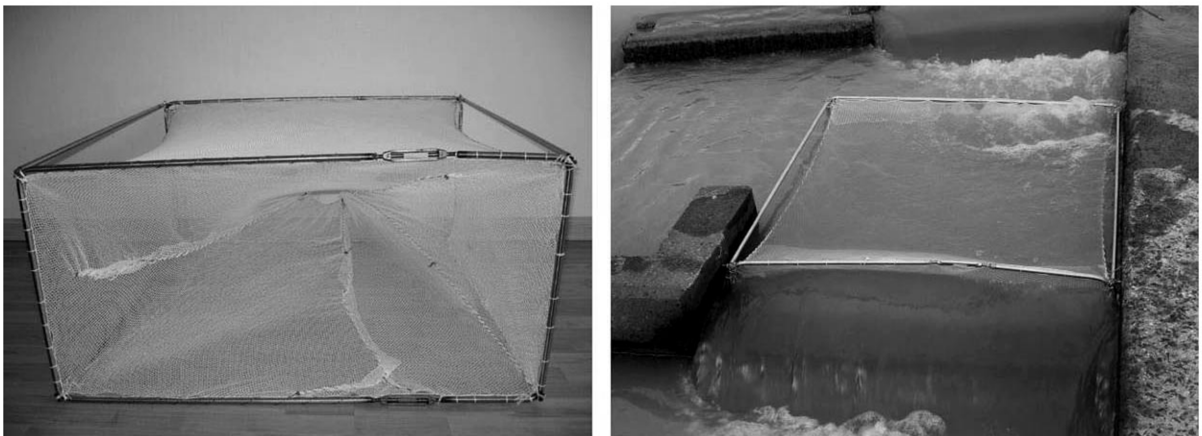

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Fig. 8 The trap which used to investigate the fishways

4.3 Experiment period

Shimcheon-bo, Tamjin River

In 2003 (Refered from Eco 21 research project of 'Ministry of Environment')

1st experiment: May 16, 2003
2nd experiment: June 2, 2003
3rd experiment: June 26, 2003
4th experiment: October 2, 2003
5th experiment: October 10, 2003
6th experiment: October 16, 2003
7th experiment: November 18, 2003

In 2004 (This study)

1st experiment: January 8, 2004
2nd experiment: March 11, 2004
3rd experiment: March 23, 2004
4th experiment: April 16, 2004
5th experiment: May 6, 2004
6th experiment: June 9, 2004
(collecting data at every three hours)
7th experiment: June 24, 2004
8th experiment: July 24, 2004
(collecting data at every three hours)
9th experiment: August 9, 2004
(collecting data at every three hours)
10th experiment: September 2, 2004
(collecting data at every three hours)
11th experiment: September 24, 2004
12th experiment: October 14, 2004
(collecting data at every three hours)
13th experiment: October 21, 2004
(collecting data at every three hours)
14th experiment: November 2, 2004
15th experiment: November 16, 2004
(collecting data at every three hours)

Hyunam-bo, Tamjin River

1st experiment: June 10, 2004
2nd experiment: June 24, 2004
3rd experiment: July 23, 2004
4th experiment: August 10, 2004

Saddle-bo, Tamjin River

1st experiment: July 23, 2004

Eoin-bo, Tamjin River

1st experiment: October 21, 2004
2nd experiment: November 16, 2004
3rd experiment: December 4, 2004

Fig. 9 Four investigated fishways at Tamjin River

5. Results and Discussions

5.1 Fish species which use the fishways

From May 16, 2003 to December 4, 2004, a total of 5,016 fish from 28 fish species were captured in four fishways at Tamjin River in Table 3. However, no fish was collected at January 8, 2004 because the water temperature dropped to 3.6˚C.

Figure 10 shows that pale chub (*Zacco platypus*) used fishways more than other fish species and it represented 51% of the total. Dolmaja (*Microphysogobio yaluensis*) made up 9.8% of the total and it ranked second. Dark chub (*Zacco temmincki*) made up 7.0%, oily bitterling (*Acheilognathus koreensis*) made up 6.9%, long nose barbel (*Hemibarbus longirostris*) made up 5.8%, and striped shinner (*Pungtungia Herzi*) represented 4.9% of the total fish.

Pale chub (*Zacco platypus*), dolmaja (*Microphysogobio yaluensis*) and dark chub (*Zacco temmincki*) were collected mainly, which are also three major fishes living at Tamjin River. Therefore, these results do not indicate that pale chub (*Zacco platypus*), dolmaja (*Microphysogobioyaluensis*) and dark chub (*Zacco temmincki*) used the fishways more frequently than the other species. Rather it implies that most of the fishes at Tamjin River used the fishways.

Also, when we performed the 'fish-fauna investigation of Tamjin River', we experimented during the daytime. On the other hands, we investigated fish migration at the fishways for 24 hours. So in this fish passage investigation, we collected some fish species which were not collected at the 'fish-fauna investigation of Tamjin River'.

In the past, it was believed that only some fish species like pale chub (*Zacco platypus*) and ayu (*Plecoglossus altivelis*) which have strong swimming ability, could use fishways. But in this research, we showed that most of fishes at Tamjin River can use fishways.

The carnivore fish species such as Japanese aucha perch (*Coreoperca kawamebari*), black bullhead (*Pseudobagrus koreanus*), Korean dark sleeper (*Odontobutis platycephala*) and far eastern catfish (Silurus asotus) do not have high mobility, but we found that these kinds of fishes also used the fishways. A territorial fish both the young and adult Japanese aucha perch (*Coreoperca kawamebari*) used the fishways at the same time. On June 24, 2004, we captured 61 individuals of black bullhead (*Pseudobagrus koreanus*) in Hyunam-bo (Figure 12) and all of them had their eggs. So, we showed that black bullhead (*Pseudobagrus koreanus*) were moving for spawning in groups. Also, in the past, Crusian carp (*Carassius auratus*) and Acheilognathinae were known for not moving because these fish species like the still water. However, oily bitterling (*Acheilognathus koreensis*) made up 7% of the total captured fishes and it was one of the most largest fish species which used the fishways. In the past, small fish species such as Korean slender gudgeon (*Squalidus gracilis majimae*), small gudgeon (*Abbottina springeri*) and dolmaja (*Microphysogobio yaluensis*) were known to have difficulties to use a fish ladder, because they have weak swimming ability. But we found that these kinds of fish species could use fishways even when the velocity of current exceeds 1.0m/s. Especially in autumn, some fries with a length of 20-30mm used the fishways.

Among the collected fish species, ayu (*Plecoglossus altivelis*) is the unique migratory fish from the ocean to a river. From this, we supposed that the fishways at 18 weir from the mouth of river to the Shimcheon-bo were poorly maintained or designed. We also observed 11 endemic fish species such as Korean rose bitterling (*Rhodeus uyekii*), oily bitterling (*Acheilognathus koreensis*), *Acheilognathus koreensis*, Korean spined bitterling (*Acanthorhodeus gracilis*), Oily shiner (*Sarcocheilichthys variegatus* wakiyae), gudgeon (*Squalidus gracilis majimae*), small gudgeon (*Abbottina springeri*), dolmaja (*Microphysogobio yaluensis*), South-

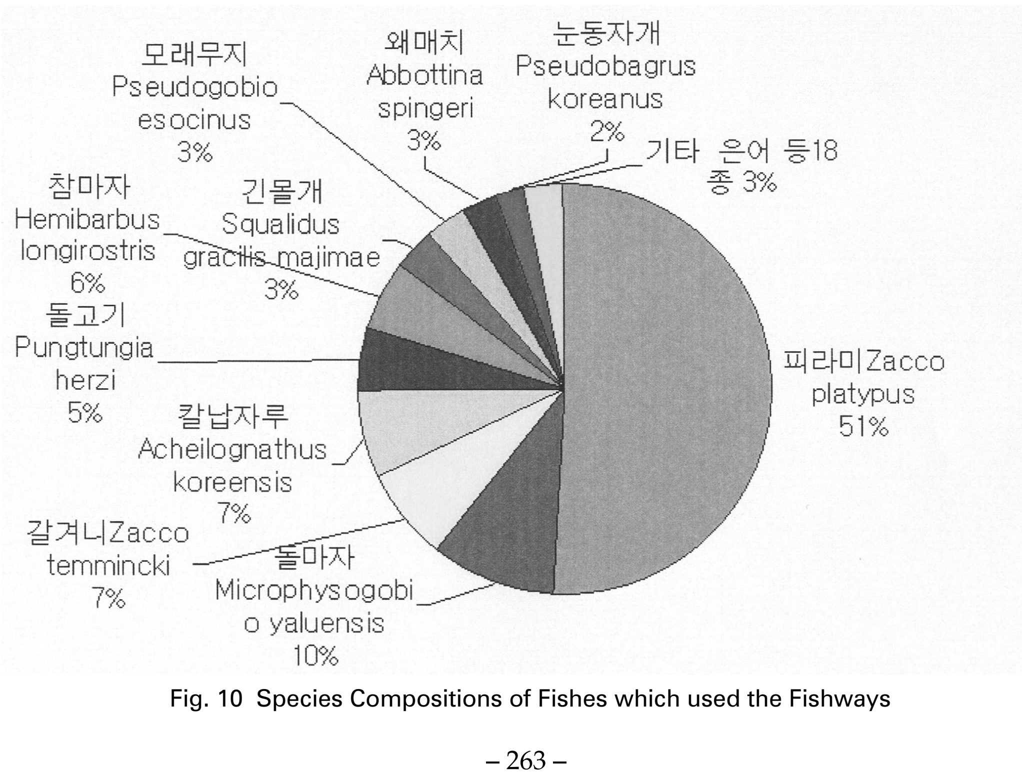

Fig. 10 Species Compositions of Fishes which used the Fishways

Table 3 Fish species were captured in four fishways at Tamjin River (Date: 030516-041204)

Scientific name Korean name	Left	Center	Right	Total	%	Remark
Cyprinidae 잉어과						
Carassius auratus 붕어	1	2	4	7	0.1	F
Acheilognathinae 납자루아과						
Rhodeus ocellatus 흰줄납줄개	1	1	-	2	0.0	F
Rhodeus uyekii 각시붕어	2	-	1	3	0.1	F*
Rhodeus notatus 떡납줄갱이	-	1	-	1	0.0	F
Acheilognathus lanceolatus 납자루	-	1	-	1	0.0	F
Acheilognathus koreensis 칼납자루	163	71	112	346	6.9	F*
Acheilognathus yamatsutae 줄납자루	3	1	-	4	0.1	F*
Acheilognathus rhombeus 납지리	3	2	-	5	0.1	F
Acanthorhodeus gracilis 가시납지리	2	1	-	3	0.1	F*
Gobioninae 모래무지아과						
Pungtungia herzi 돌고기	42	29	175	246	4.9	F
Sarcocheilichthys variegatus wakiyae 참중고기	16	15	13	44	0.9	F*
Squalidus gracilis majimae 긴몰개	49	51	71	171	3.4	F*
Hemibarbus longirostris 참마자	134	62	94	290	5.8	F
Pseudogobio esocinus 모래무지	54	48	58	160	3.2	F
Abbottina spingeri 왜매치	43	24	70	137	2.7	F*
Microphysogobio yaluensis 돌마자	131	154	207	492	9.8	F*
Danioninae 피라미아과						
Zacco temmincki 갈겨니	131	78	140	349	7.0	F
Zacco platypus 피라미	970	571	1015	2256	51.0	F
Cultrinae 강준치아과						
Hemiculter eigenmanni 치리	1	2	-	3	0.1	F
Cobitidae 미꾸리과						
Iksookimia hugowolfeldi 남방종개	5	-	3	8	0.2	F*
Cobitis lutheri 점줄종개	9	7	15	31	0.6	F
Bagridae 동자개과						
Pseudobagrus koreanus 눈동자개	23	10	88	121	2.4	F*
Siluridae 메기과						
Silurus asotus 메기	-	1	2	3	0.1	F
Osmeridae 바다빙어과						
Plecoglossus altivelis 은어	2	1	1	4	0.1	P
Centropomidae 꺽지과						
Coreoperca kawamebari 꺽저기	3	3	3	9	0.2	P
Odontobutidae 동사리과						
Odontobutis platycephala 동사리	1	1	1	3	0.1	P*
Gobiidae 망둑어과						
Rhinogobius giurinus 갈문망둑	1	-	-	1	0.0	P
Rhinogobius brunneus 밀어	6	1	9	16	0.3	P
Total	1796	1138	2082	5016	100.0	

Habitat analysis	Left	Center	Right	Total	
F:Freshwater fish(1st):	20	21	16	23	Species
P:Peripheral fish:	5	4	4	5	Species
Total species number:	25	25	20	28	Species

* Endemic species

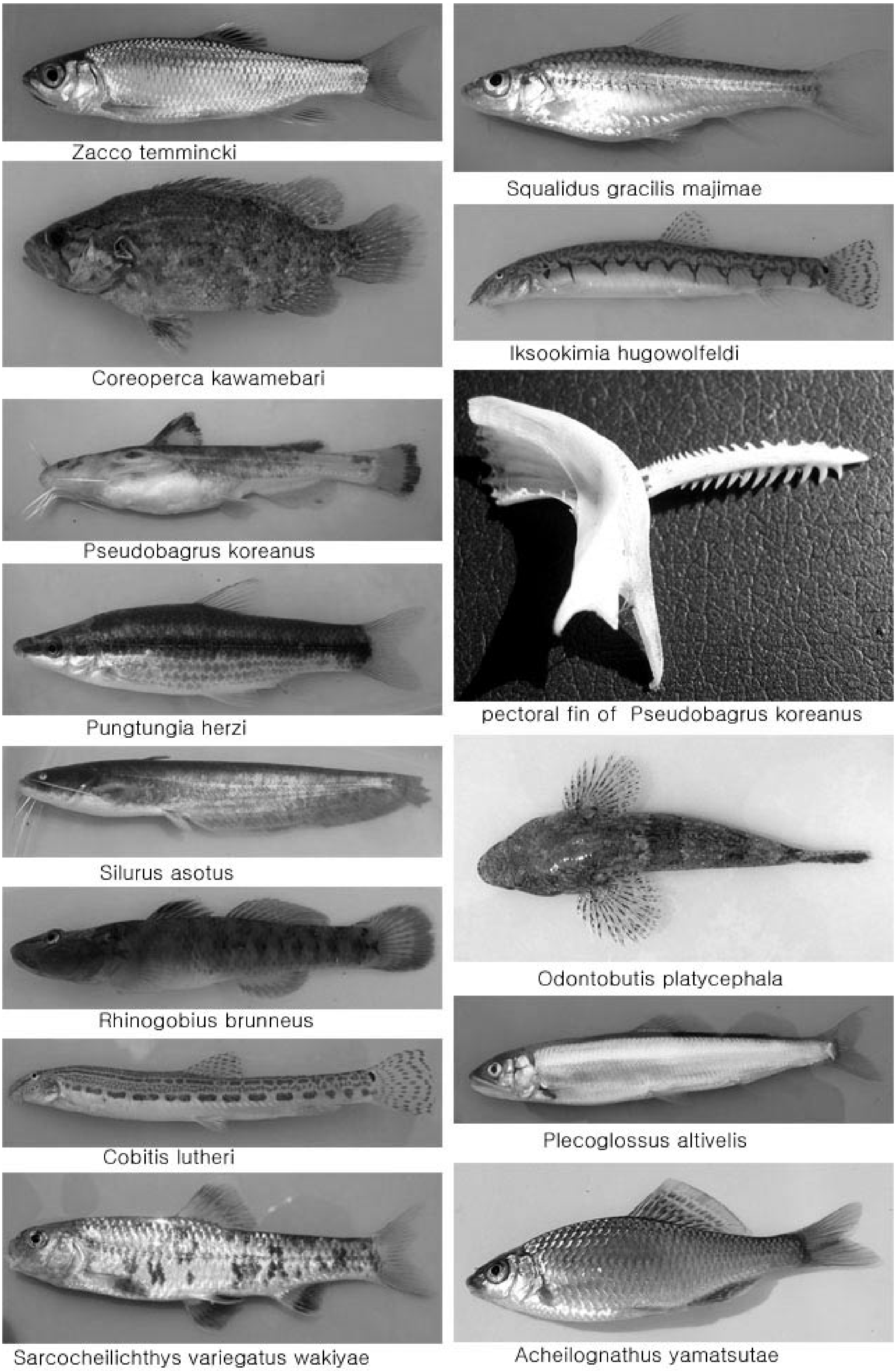

Fig. 11 Collected fishes in the fishways at Tamjin River

Fig. 12 collected black-bullhead and other fishes (July 24, 2004, in Hyunam-bo)

ern king spine loach (*Iksookimia hugowolfeldi*), Black bullhead (*Pseudobagrus koreanus*) and Korean dark sleeper (*Odontobutis platycephala*). From this, we confirmed that the fishways are effective at conserving endemic fish species.

In the past, it was widely believed that only large sized fish species used the fishway for spawning. But in this research at Tamjin River we observed various kinds of fish species. So we confirm that all kinds of fish species in the river can use the fishways.

In the fishways, we also collected Korean freshwater snail (*Semisulcospira libertina*), larvae of dragonflies, hair crab (*Eriocheir sinensis*) and Chinese brown frogs (*Rana chensisensis*) . So we presume that they can also use the fishways besides the fishes.

From the result of this investigation, we conclude that most creatures including fishes can migrate from down stream to upper stream through the Ice Harbor fishways in the river.

5.2 Seasonal fish migration

Table 4 and Figure 13 show that the numbers of collected fishes and fish species by the experiment date. In general, the number of captured fishes in 2004 were nearly three times larger than that in 2003. Also, the number of collected fish species in 2004 increased almost twofold compared with that in 2003.

In the past, it was believed that the most of the fishes used the fishways in the spring season such as April, May and June. However, in Figure 13, fishes used the fishway on July, August and September more frequently than other months.

As people presumed that the fishes use fishways for spawning in spring, we confirmed the fact through the experiment. We also observed that in summer, after some young fishes drift down the river during a flood, they went up again to the upper stream through fishways. In addition, we observed that Japanese aucha perch (*Coreoperca kawamebari*) used the fishway when they leave their mother and become independent.

We examined the data from November to January to show the fish migration in winter. On January 8, 2004, the water temperature was the lowest (3.6°C) and we didn't collect any fish because it was too cold for fishes to migrate. On November 16, 2004, the water temperature in Shimcheon-bo was 9.8°C and we collected 20 fish from 5 fish species. On March 10, 2004, 2 pale chub (*Zacco platypus*) were captured and the water temperature was 10.2°C. Also we collected 2 pale chub (*Zacco platypus*) on November 16, 2003, and the water temperature was 13.2°C.

Finally, we collected 58 fish from 5 fish species on November 2, 2004 and the water temperature was 16.5°C.

In summary, we found that the fishes didn't move below 3.6°C and it began to move above 9.8°C. But, we have to investigate more to determine when the fishes stop their movement between 3.6°C and 9.8°C.

In the past, it was widely believed that fishes used the fishway for spawning.

So on May 16 and June 3, we found that adult fishes with nuptial color used the fishways for spawning. But on March 10 and 23, the investigation showed that adult fishes without having nuptial color used the fishway regardless of spawning. Also, after June 26, fries used the fishways, too.

From these results, we confirmed that all fishes and all fish species use the fishways because of not

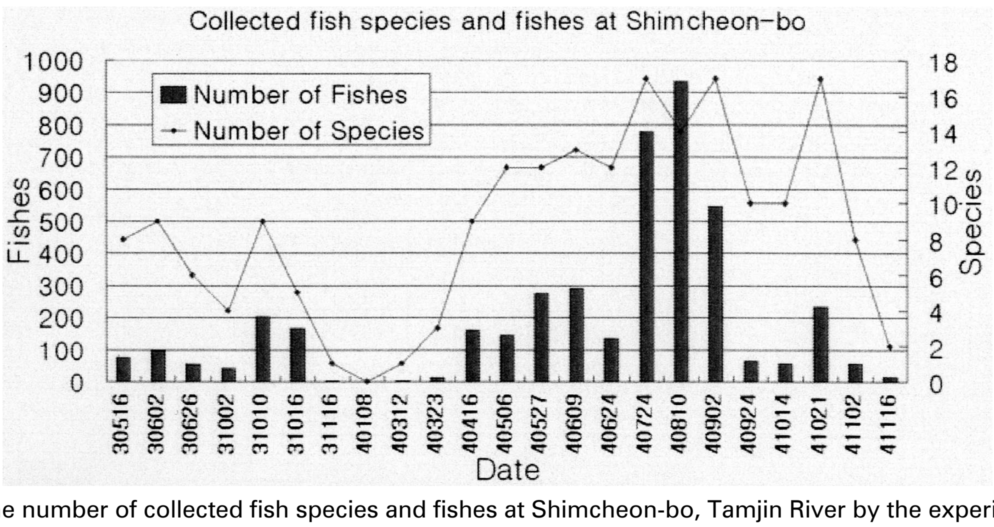

Fig. 13 The number of collected fish species and fishes at Shimcheon-bo, Tamjin River by the experiment date

Table 4 The investigation results (Shimcheon-bo, Tamjin River)

Sampling Date	2003							2004																		
Scientific name Korean name	05/16	06/02	06/26	10/02	10/10	10/16	11/16	01/08	03/10	03/23	04/16	05/05	05/27	06/09	06/24	07/24	08/09	09/02	09/24	10/14	10/21	11/02	11/16	Total	%	Rem.
Carassius auratus 붕어	-	-	-	-	-	-	-	-	-	-	-	-	1	-	-	-	2	-	-	1	1	-	-	5	0.1	F
Rhodeus ocellatus 흰줄납줄개	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	1	1	-	-	-	-	-	2	0.0	F
Rhodeus uyekii 각시붕어	-	-	-	-	3	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	3	0.1	F*
Rhodeus notatus 떡납줄갱이	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	1	-	-	-	-	-	-	1	0.0	F
Acheilognathus lanceolatus 납자루	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	1	-	-	-	-	-	-	-	1	0.0	F
Acheilognathus koreensis 칼납자루	4	-	-	1	46	-	-	-	-	-	7	2	1	1	11	36	6	120	2	-	80	4	-	321	7.4	F*
Acheilognathus yamatsutae 줄납자루	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	1	-	1	-	-	-	3	0.1	F*
Acheilognathus rhombeus 납지리	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	1	4	-	-	-	-	-	5	0.1	F
Acanthorhodeus gracilis 가시납지리	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	1	-	-	-	-	1	1	-	3	0.1	F*
Pungtungia herzi 돌고기	1	5	5	-	3	-	-	-	-	-	24	1	24	8	7	10	-	3	-	1	3	-	-	95	2.2	F
Sarcocheilichthys variegatus wakiyae 참중고기	-	2	-	-	-	-	-	-	-	-	-	2	5	3	4	9	2	7	-	4	5	1	-	44	1.0	F*
Squalidus gracilis majimae 긴몰개	13	4	-	-	-	-	-	-	-	1	8	47	16	12	-	16	-	36	-	-	1	-	-	154	3.5	F*
Hemibarbus longirostris 참마자	24	3	-	16	24	18	-	-	4	1	32	12	9	4	12	14	16	16	17	4	8	16	-	250	5.7	F
Pseudogobio esocinus 모래무지	1	2	3	5	29	43	-	-	-	-	4	2	3	5	15	8	10	5	12	3	5	-	-	155	3.6	F
Abbottina spingeri 왜매치	-	-	-	-	8	1	-	-	-	-	-	19	14	6	11	42	4	18	5	3	1	1	-	133	3.1	F*
Microphysogobio yaluensis 돌마자	12	23	15	-	22	-	-	-	-	-	8	17	20	70	15	73	87	100	6	5	14	1	-	488	11.2	F*
Zacco temmincki 갈겨니	4	2	-	-	8	2	-	-	-	-	1	4	10	7	10	59	85	49	5	1	2	8	-	257	5.9	F
Zacco platypus 피라미	15	55	27	21	60	103	2	-	-	10	77	40	170	160	44	494	716	143	15	34	100	26	14	2327	53.5	F
Hemiculter eigenmanni 치리	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	2	-	1	-	-	-	-	-	3	0.1	F
Cobitis lutheri 점줄종개	-	-	-	-	-	-	-	-	-	-	1	-	-	6	2	4	-	1	-	-	3	-	-	17	0.4	F
Pseudobagrus koreanus 눈동자개	-	1	-	-	-	-	-	-	-	-	-	1	-	1	2	1	-	41	1	-	6	-	-	54	1.2	F*
Silurus asotus 메기	-	-	-	-	-	-	-	-	-	-	-	-	-	-	1	-	-	-	1	-	-	-	-	2	0.0	F
Plecoglossus altivelis 은어	-	-	1	-	-	-	-	-	-	-	-	-	-	-	-	2	-	1	-	-	-	-	-	4	0.1	P
Coreoperca kawamebari 꺽저기	-	-	3	-	-	-	-	-	-	-	-	-	-	-	-	5	-	-	-	-	1	-	-	9	0.2	P
Odontobutis platycephala 동사리	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	1	-	-	1	0.0	P*
Rhinogobius giurinus 갈문망둑	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	1	-	-	1	0.0	P
Rhinogobius brunneus 밀어	-	-	-	-	-	-	-	-	-	-	-	1	2	5	-	-	1	2	-	1	-	-	-	12	0.3	P
Total fishes	74	97	54	43	203	167	2	0	4	12	162	148	275	289	134	777	933	548	65	57	233	58	15	4350	100.0	
Total species number:	8	9	6	4	9	5	1	0	1	3	9	12	12	13	12	17	14	17	10	10	17	8	2	27		

* Endemic Species

only spawning but also their instinct.

In the past, it was believed that fishes used the fishway from April to June because of spawning. But this investigation showed that the fishes used the fisways in the summer months more than other months. Also, in the fall, the fishes sill used the fishways. Therefore, the results indicate that we have to manage the fishways form March to early December.

5.3 Fish migration by time

From June 10, 2004 to November 16, 2004, we experimented seven times by the following time schedule, to find when the fishes used the fishway most. (Table 5)

In the past, it was believed that fishes migrate through the fishways at the sunset and sunrise time. But according to Figure 14, most fishes used the fishway from 3 PM to 9 PM whereas fishes rarely used the fishway from 9 PM to 3 PM (next day), which is significatnly different from the previous belief. Also the Figure shows that the number of fish species didn't depend on time much.

Table 5 The fish migration by time at Shimcheon-bo, Tamjin River (Date: 0406-0411)

Scientific name Common name	Time 21-06	06-09	09-12	12-15	15-18	18-21	Total	%	Rem.
Cyprinidae 잉어과									
Carassius auratus 붕어	3	-	-	-	-	1	4	0.1	F
Acheilognathinae 납자루아과									
Rhodeus ocellatus 흰줄납줄개	-	-	2	-	-	-	2	0.1	F
Rhodeus notatus 떡납줄갱이	-	-	1	-	-	-	1	0.0	F
Acheilognathus lanceolatus 납자루	-	1	-	-	-	-	1	0.0	F
Acheilognathus koreensis 칼납자루	2	6	11	12	134	78	243	8.5	F
Acheilognathus yamatsutae 줄납자루	-	1	-	-	1	-	2	0.1	F
Acheilognathus rhombeus 납지리	-	-	1	1	-	3	5	0.2	F
Acanthorhodeus gracilis 가시납지리	-	-	-	-	1	1	2	0.1	F
Gobioninae 모래무지아과									
Pungtungia herzi 돌고기	-	1	1	4	5	14	25	0.9	F
Sarcocheilichthys variegatus wakiyae 참중고기	1	1	2	1	8	17	30	1.1	F
Squalidus gracilis majimae 긴몰개	2	9	11	5	19	19	65	2.3	F
Hemibarbus longirostris 참마자	28	3	7	8	7	9	62	2.2	F
Pseudogobio esocinus 모래무지	14	4	1	2	7	8	36	1.3	F
Abbottina spingeri 왜매치	4	9	6	12	18	25	74	2.6	F
Microphysogobio yaluensis 돌마자	30	66	48	34	83	88	349	12.2	F
Danioninae 피라미아과									
Zacco temmincki 갈겨니	14	12	12	25	96	44	203	7.1	F
Zacco platypus 피라미	130	129	92	102	344	865	1662	58.3	F
Cultrinae 강준치아과									
Hemiculter eigenmanni 치리	-	2	-	-	1	-	3	0.1	F
Cobitidae 미꾸리과									
Cobitis lutheri 점줄종개	-	-	-	5	8	1	14	0.5	F
Bagridae 동자개과									
Pseudobagrus koreanus 눈동자개	31	5	5	1	1	6	49	1.7	F
Osmeridae 바다빙어과									
Plecoglossus altivelis 은어	1	-	-	1	1	-	3	0.1	P
Centropomidae 꺽지과									
Coreoperca kawamebari 꺽저기	-	1	1	-	2	2	6	0.2	P
Odontobutidae 동사리과									
Odontobutis platycephala 동사리	-	-	1	-	-	-	1	0.0	P
Gobiidae 망둑어과									
Rhinogobius giurinus 갈문망둑	-	-	-	-	1	-	1	0.0	P
Rhinogobius brunneus 밀어	3	2	2	1	-	1	9	0.3	P
Total	263	252	204	214	737	1182	2852	100.0	

Habitat analysis	21-06	06-09	09-12	12-15	15-18	18-21	Total
F:Freshwater fish(1st):	11	14	14	13	15	15	20 Species
P:Peripheral fish:	2	2	3	2	3	2	5 Species
Total species number:	13	16	17	15	18	17	25 Species

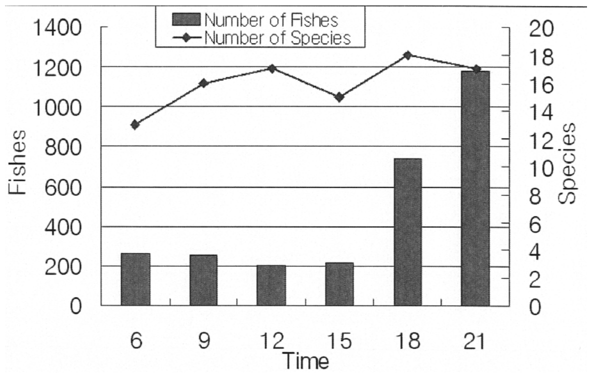

Fig. 14 The fish migration by time (Shimcheon-bo)

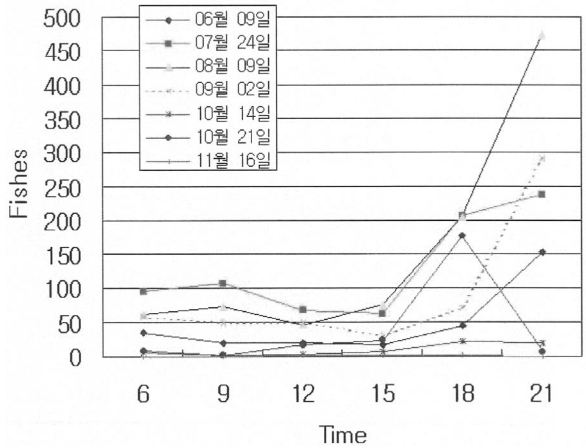

Fig. 15 The number of collected fishes by time and date

From these results, we came to a conclusion that the sunrise doesn't affect the fish migration. After that, we analyzed data to make sure the relationship between fish migration and the water temperature.

Figure 15 shows that the number of collected fishes by time and date. According to this graph, on June 10, 2004, 53% of the total collected fishes used fishway between 6 PM and 9 PM. And on July 24, 57% of the total used fishway between 3 PM and 9 PM. On August 9, 73% of the total used fishway between 3 PM and 9 PM. Also, on September 2, 53% of the total used fishway between 6 PM and 9 PM. On October 14, 77% of the total used fishway between 3 PM and 9 PM. And on October 20, 76% of the total used fishway between 3 PM and 6 PM. Finally on November 16, 80% of the total collected fishes used fishway between 3 PM and 6 PM.

Especially, on August 9, 2004, more fishes used fishway than other days (Figure 16). And the water temperature at Shimcheon-bo was 29.0°C at 6 AM, 29.7°C at 9 AM, 31.5°C at noon, 33.2°C at 3 PM, 33.6°C at 6 PM and 31.0°C at 9 PM. Between 9 PM and 3 PM, less than 70 fishes used fishway. But between 3 PM and 6 PM, the water temperature was highest and we collected 204 fishes. And, around the sunset times (between 6 PM and 9 PM) 474 fishes were captured. But we can't confirm that this trend occurred because of the high water temperature or the sunset. Also, the investigation shows that after the sunset, not many fishes migrate. So, we have to investigate more to prove the relationship between fish migration and the water temperature or the sunset.

5.4 Fish migration in fishways by location

Table 6 shows the preference of fishes when they use the fishway at Shimcheon-bo. Also, according to Figure 17, many fish species had a preference of the leftmost part of the fishway, and 22 fish species used that part of fishways. And about 16 fish species used the rest part of the fishway.

Also, Figure 11 shows that we collected 1,189 fishes on the leftmost part of the fishway and 1,084 fishes on the rightmost part of the fishway. The results indicated that fishes used the leftmost and rightmost part of the fisway twice more than the rest of part. This results are well in line with the previous theory that most of the fishes used the leftmost and rightmost part of fishways.

We installed a small fish trap in a orifice region to collect fishes. But we didn't collect fish at all and orifice migration of the fishes was not inspected. But we found that the speed of current on the weir was about 1.0m/s whereas the speed of current on the orifice region was about 0.6m/s. So we suppose that some

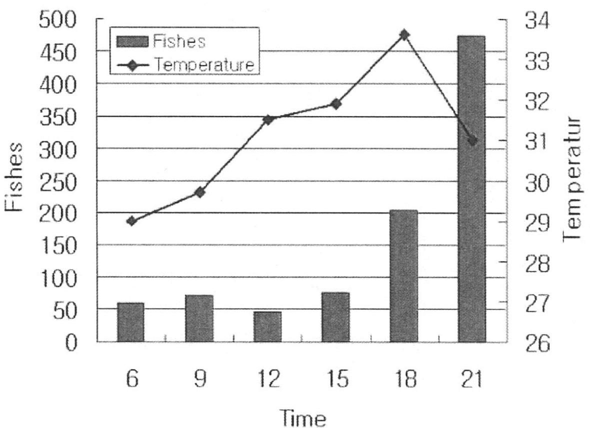

Fig. 16 The number of collected fishes and water temperature at August 9

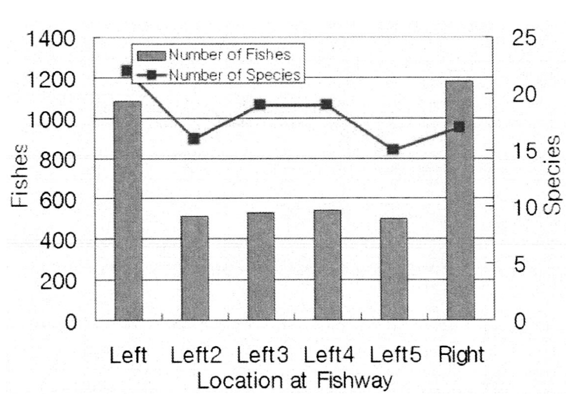

Fig. 17 The number of collected fishes and fish species by location

Table 6 The fish migration by location at Shimcheon-bo, Tamjin River (Date: 03-041204)

Scientific name Korean name	Left	Left2	Left3	Left4	Left5	Right	Total	%	Rem.
	Location at fishway								
Cyprinidae 잉어과									
Carassius auratus 붕어	1	-	-	1	2	1	5	0.1	F
Acheilognathinae 납자루아과									
Rhodeus ocellatus 흰줄납줄개	-	1	-	1	-	-	2	0.0	F
Rhodeus uyekii 각시붕어	2	-	-	-	-	1	3	0.1	F
Rhodeus notatus 떡납줄갱이	-	-	1	-	-	-	1	0.0	F
Acheilognathus lanceolatus 납자루	-	-	-	1	-	-	1	0.0	F
Acheilognathus koreensis 칼납자루	135	21	33	35	21	76	321	7.4	F
Acheilognathus yamatsutae 줄납자루	2	-	1	-	-	-	3	0.1	F
Acheilognathus rhombeus 납지리	3	-	1	1	-	-	5	0.1	F
Acanthorhodeus gracilis 가시납지리	1	1	-	1	-	-	3	0.1	F
Gobioninae 모래무지아과									
Pungtungia herzi 돌고기	23	9	19	10	8	26	95	2.2	F
Sarcocheilichthys variegatus wakiyae 참중고기	12	4	9	6	1	12	44	1.0	F
Squalidus gracilis majimae 긴몰개	29	17	24	25	20	39	154	3.5	F
Hemibarbus longirostris 참마자	85	38	30	27	21	49	250	5.7	F
Pseudogobio esocinus 모래무지	37	14	14	33	16	41	155	3.6	F
Abbottina spingeri 왜매치	31	10	10	14	18	50	133	3.1	F
Microphysogobio yaluensis 돌마자	81	48	40	114	44	161	488	11.2	F
Danioninae 피라미아과									
Zacco temmincki 갈겨니	59	26	36	27	37	72	257	5.9	F
Zacco platypus 피라미	561	316	302	235	290	623	2327	53.5	F
Cultrinae 강준치아과									
Hemiculter eigenmanni 치리	1	-	1	1	-	-	3	0.1	F
Cobitidae 미꾸리과									
Cobitis lutheri 점줄종개	3	-	1	1	4	8	17	0.4	F
Bagridae 동자개과									
Pseudobagrus koreanus 눈동자개	13	6	2	8	14	11	54	1.2	F
Siluridae 메기과									
Silurus asotus 메기	-	-	-	1	-	1	2	0.0	F
Osmeridae 바다빙어과									
Plecoglossus altivelis 은어	1	1	1	-	1	-	4	0.1	P
Centropomidae 꺽지과									
Coreoperca kawamebari 꺽저기	2	1	3	-	-	3	9	0.2	P
Odontobutidae 동사리과									
Odontobutis platycephala 동사리	-	-	1	-	-	-	1	0.0	P
Gobiidae 망둑어과									
Rhinogobius giurinus 갈문망둑	1	-	-	-	-	-	1	0.0	P
Rhinogobius brunneus 밀어	1	2	-	-	3	6	12	0.3	P
Total	1084	515	529	542	500	1180	4350	100.0	

Habitat analysis	Left	Left2	Left3	Left4	Left5	Right	Total
F:Freshwater fish(1st):	18	13	16	19	13	15	22 Species
P:Peripheral fish:	4	3	3	-	2	2	5 Species
Total species number:	22	16	19	19	15	17	27 Species

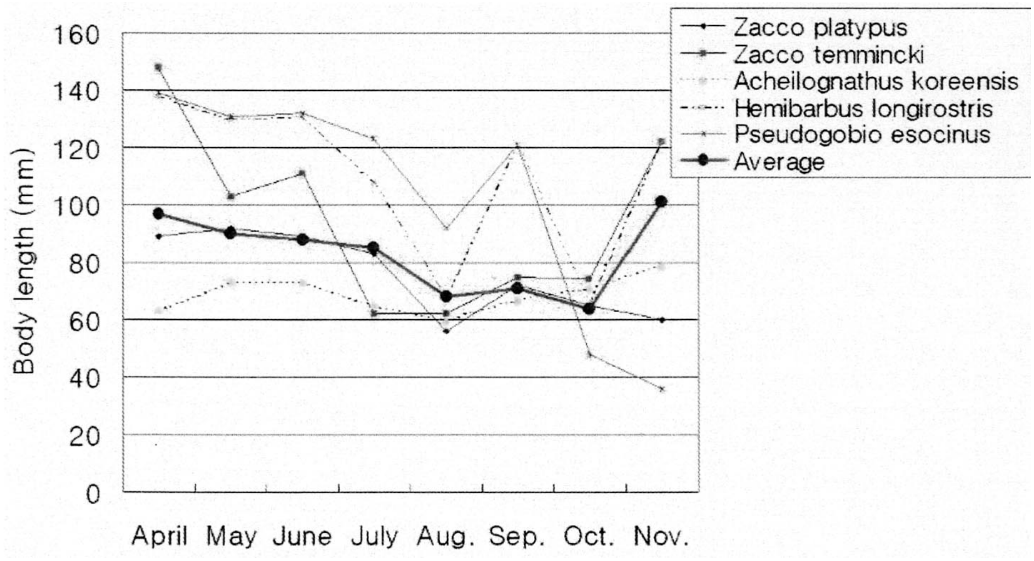

Fig. 18 The body length of collected fishes by mont

fishes with weak swimming capabilities or benthic fishes may use the orifice.

5.5 Fish migration by body length

We examined the body length to investigate a relationship between swimming capabilities and fish passage. We examined a total of 4,350 fishes from 27 fish species and we calculated that the average body length of the fishes was 74mm (Table 7). Among fish species which we examined more than 10 individuals, long nose barbel (*Hemibarbus longirostris*) had the largest length and its average body length was 107mm. The average body length of sand spine loach (*Cobitis lutheri*) was 100mm, striped shinner (*Pungtungia Herzi*) was 89mm and goby minnow (*Pseudogobio esocinus*) had 82mm of the average body length.

One of the smallest fishes was pale chub (*Zacco platypus*) (17mm) and Japanese aucha perch (*Coreoperca kawamebari*) (20mm). And the largest fish was far eastern catfish (Silurus asotus) and it was 360mm. Especially, pale chub (*Zacco platypus*) used the fishways more than other fish species and their body length was between 17mm and 230mm. But we found that smaller pale chub (*Zacco platypus*) used the fishways mainly.

Nakamura said "Fishes have 4BL of cruise speed and 10BL darting speed in their swimming capabilities." (BL means the length.) And from our investigation, we found that the average body length of the fishes was 74mm, the smallest one was 17mm and the largest one was 360mm. So we can estimate that the average cruise speed of the fishes is 0.3m/s and the range of cruise speed are between 0.07m/s and 1.4m/s. Also, the average darting speed of the fishes is 0.7m/s and the range are between 0.17m/s and 3.62m/s.

The depths of water were various depending on the date. But on October 16, 2003, young fishes migrated more than other days. And that day, the depth of water was more than 14cm and the velocity of current were between 1.2m/s and 1.5m/s. So according to Nakamura, young fishes with a body length of less than 20mm couldn't used the fishway even though they migrated at their darting speed. But we found that the young fishes could use the fishway even when the velocity of current exceeds 1.2m/s.

It means that young fishes migrate through the edges of fishway instead of the middle of fishway with high velocity of current. Also, we suppose that young fishes passed at the weir by their darting speed after resting at the pool. Actually, we collected most of fishes near the edges. Also, the depths of water were usually between 15cm and 17cm and the head of weirs was 10cm. So there would be submerged flow around 5cm to 7cm. So we confirmed that the fishes used a whirlpool in the submeged flow region and used the fishway when the velocity of current was low.

From these results, we show that we have to design a fishway which has low head between weirs. So some fishes which have weak swimming capabilities or tend not to jump will use the fishway.

Figure 18 shows the body length of collected fishes by month. According to Figure 11, we can know that larger fishes used the fishway in the spring and winter months while smaller fishes used the fishway in summer and fall. So we can suppose that in the spring adult fishes mainly used the fishway for spawning. And after July, fries began to migrate through the fishways, so in the fall many fries with 20-30mm used the fishway.

On June 26, 2003, we collected small fries of Japanese aucha perch (*Coreoperca kawamebari*) (Figure 19). Japanese aucha perch (*Coreoperca kawamebari*) is one of the sedentariness fishes and usually builds their territory. So when their fries become independent, they have to quarrel their habitats with their mother and should leave their mother. So we conclude that some sedentariness fishes use fishways because of these reasons.

Table 7 Fish migration by body length at Shimcheon-bo (Date: 03-041204)

Scientific name Korean name	Fishes	Body length(mm) Average	Minimum	Maximum
Cyprinidae 잉어과				
Carassius auratus 붕어	5	168	155	202
Acheilognathinae 납자루아과				
Rhodeus ocellatus 흰줄납줄개	2	72	67	76
Rhodeus uyekii 각시붕어	3	36	23	58
Rhodeus notatus 떡납줄갱이	1	47	47	47
Acheilognathus lanceolatus 납자루	1	96	96	96
Acheilognathus koreensis 칼납자루	321	69	32	94
Acheilognathus yamatsutae 줄납자루	3	69	53	92
Acheilognathus rhombeus 납지리	5	61	53	75
Acanthorhodeus gracilis 가시납지리	3	84	72	92
Gobioninae 모래무지아과				
Pungtungia herzi 돌고기	95	89	58	132
Sarcocheilichthys variegatus wakiyae 참중고기	44	76	42	105
Squalidus gracilis majimae 긴몰개	154	69	47	92
Hemibarbus longirostris 참마자	250	107	23	210
Pseudogobio esocinus 모래무지	155	82	19	191
Abbottina spingeri 왜매치	133	75	52	95
Microphysogobio yaluensis 돌마자	488	58	25	96
Danioninae 피라미아과				
Zacco temmincki 갈겨니	257	71	32	198
Zacco platypus 피라미	2327	73	17	230
Cultrinae 강준치아과				
Hemiculter eigenmanni 치리	3	92	87	97
Cobitidae 미꾸리과				
Cobitis lutheri 점줄종개	17	100	92	107
Siluriformes 메기목				
Bagridae 동자개과				
Pseudobagrus koreanus 눈동자개	54	58	21	250
Siluridae 메기과				
Silurus asotus 메기	2	345	330	360
smeriformes 바다빙어목				
Osmeridae 바다빙어과				
Plecoglossus altivelis 은어	4	162	109	232
Perciformes 농어목				
Centropomidae 꺽지과				
Coreoperca kawamebari 꺽저기	9	67	20	102
Odontobutidae 동사리과				
Odontobutis platycephala 동사리	1	100	100	100
Gobiidae 망둑어과				
Rhinogobius giurinus 갈문망둑	1	51	51	51
Rhinogobius brunneus 밀어	12	49	31	62
Total	4350	74	17	360

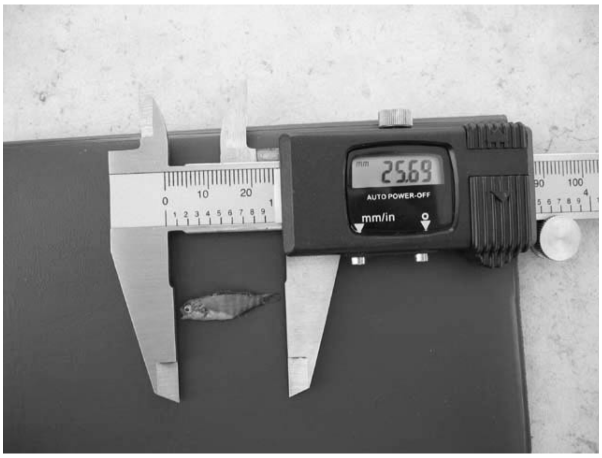

Fig. 19 The measurement of the body length of Japanese aucha perch (*Coreoperca kawamebari*)

5.6 Other creatures which use the fishways

Figure 20 shows the list of aquatic animals that were collected at the fishways. Besides fishes, we observed that hair crab (*Eriocheir sinensis*), larvae of dragonflies and Korean freshwater snail (*Semisulcospira libertina*) use the fishways at Tamjin rivers. Specifically, we found a lot of Korean freshwater snail (*Semisulcospira libertina*) at the bottom of the Weir and observe that many Korean freshwater snail (*Semisulcospira libertina*) falls after trying to migrate upward, but we observe large population of Korean freshwater snail (*Semisulcospira libertina*) at the exit of fishways. Therefore, we conclude that Korean freshwater snail (*Semisulcospira libertina*) uses the fishways. Also, we collected Chinese brown frogs (*Rana chensisensis*) at traps. Finally, we observed the excreta of Eurasian otter (*Lutra lutra*) in the fishways because Eurasian otter (*Lutra lutra*) lives at fishways to prey on the fishes in the fishway.

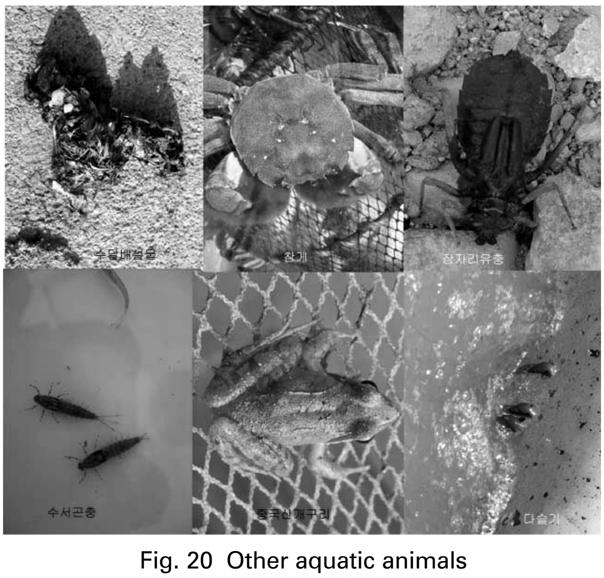

Fig. 20 Other aquatic animals which used the fishways

6. Conclusion

1. The experiment at the Knock-down Ice Harbor type fishway at Tamjin River shows that 28 species of fishes including ayu (*Plecoglossus altivelis*) and Japanese aucha perch (*Coreoperca kawamebari*), migrate through the fishway. Therefore, we conclude that most of the fishes living at Tamjin river uses the fishway regardless of their jumping and swimming capabilities
2. Most of the fishes are collected from early March to the early December, which shows that fishes usually migrates when the water temperature exceeds 10°C.
3. At Tamjin river, fishes used the fishway most frequently just before the sunset, and few fishes used the fishway at the sunrise when the water temperature is the coldest.
4. Most of the fishes used the leftmost and rightmost part of the fishway.
5. Contrary to the common belief that the most of the fishes use fishways when they migrate to spawn, we showed that young fishes use fishways to branch family and fishes that washed away during a flood also use fishways to return to the upstream.
6. The average body length of the fishes that use the fishways at Tamjin river was 74mm. Based upon the average body length, we estimated that the cruising speed and darting speed of the fishes are 0.3m/s and 0.7m/s, respectively. However, the average current velocity of the water at the fishway exceeds 1.0m/s, which exceeds the darting speed of the average fishes. Therefore, more research on how the fishes migrates through the fishways is required.
7. Besides fishes, aquatic animal such as hair crab (*Eriocheir sinensis*), Korean freshwater snail (*Semisulcospira libertina*) and larvae of dragonflies also uses fishways.

Numerical Simulation of Braided Rivers

Chang-Lae Jang
Senior Researcher, Hydrosystems Engineering Center,
Korea Institute of Water and Environment, KOWACO,
Daejeon, Korea

Yasuyuki Shimizu
Associate Professor, Division of Environment and Resource Engineering,
Hokkaido Univeristy, Japan

Ick Hwan Ko
Director, Hydrosystems Engineering Center,
Korea Institute of Water and Environment,
KOWACO, Daejeon, Korea

Woogu Kim
President, Korea Institute of Water and Environment,
KOWACO, Daejeon, Korea

Abstract

Understanding the process and mechanism of morphological behavior in braided river is very important for river engineering purposes to manage hydraulic structures and prevent disaster from flood, and environmental purposes to maintain river ecosystem and landscape. A two-dimensional numerical model was developed to simulate braided river with erodible bed and banks composed of well sorted-sandy materials. A generalized coordinate system was employed to calculate water flow, bed change, and bank erosion. CIP (Cubic Interpolated Pseudo-particle) method was used to calculate flow, which introduced little numerical diffusion. Sediment transport equation in the streamline and transverse wise, considering the secondary flow, was used to estimate bed and bank evolution in time. Bank erosion was simulated by following the procedure proposed by Shimizu (2002).

Braided river in laboratory was reproduced for verifying the numerical model in the channel filled with nearly uniform sandy materials. Comparison of numerical results and experimental data has shown relatively good agreements.

Keywords: *Braided river; Bank erosion; Numerical model; CIP method; Moving boundary-fitted coordinate system.*

1. Introduction

The morphological changes of rivers are deeply interrelated to the bed deformation and bank erosion because of the mutual relationship between water flow and sediment transport, and better understanding of these process and mechanism is very important for river engineering purposes to manage hydraulic structures and prevent disaster from flood, and environmental engineering purposes to maintain river ecosystem and landscape.

In the process of channel development, bars emerge under certain hydraulic conditions as the channel widens from an initially straight channel, with erodible bed and banks. Previous investigations examined the mechanical processes of channels with erodible banks theoretically (Ikeda et al., 1981; Parker et al., 1982), and have provided a method to reproduce lateral changes in the channel.

Recently several numerical models have been developed to reproduce braided rivers with fixed banks (Murray and Paola 1994). However, the model has some limitations to calculate braided rivers with unconstrained banks. Shimizu (2002) proposed a numerical model to simulate braided rivers with erodible banks, and showed the possibility of simulating braided rivers considering bank erosion.

The numerical model we developed is capable of simulating two-dimensional bed changes and width variations in braided river with erodible bed and banks composed of non-cohesive materials. A generalized coordinate system was employed to describe a natural shaped boundary assuming that erosion and sediment deposition taken place in the cross sectional direction transformed the plane shape of the water channel into an arbitrary shape. Flow field was calculated by using a high-order Godunov scheme, that is, CIP method (Yabe et al. 1990). It was assumed that the erosion of banks occurred when the gradient in the cross-sectional direction of the banks was steeper than the submerged angle of repose because of the bed erosion in the vicinity of banks. And the amount of sediment beyond the submerged critical angle of bank slope was included in the computation of the bed evolution as a supply of sediment from the bank. The model results were compared with laboratory experiments, which demonstrated its applicability.

2. Governing Equations

2.1 Water Flow

The governing equations, i.e. continuity and momentum equations, for water flow are transformed from the Cartesian coordinate system to a moving boundary-fitted coordinate system due to the deformation of side banks. The equations in the moving boundary-fitted coordinate system are as follows.

Continuity equation:

$$\frac{\partial}{\partial \tau}\left(\frac{h}{J}\right)+\frac{\partial}{\partial \xi}\left[\left(\xi_t+u^{\xi}\right)\frac{h}{J}\right]+\frac{\partial}{\partial \eta}\left[\left(\eta_t+u^{\eta}\right)\frac{h}{J}\right]=0 \tag{1}$$

Momentum equations in ξ and η directions:

$$\begin{aligned}&\frac{\partial u^{\xi}}{\partial \tau}+(\xi_t+u^{\xi})\frac{\partial u^{\xi}}{\partial \xi}+(\eta_t+u^{\eta})\frac{\partial u^{\xi}}{\partial \eta}+\alpha_1 u^{\xi}u^{\xi}+\alpha_2 u^{\xi}u^{\eta}+\alpha_3 u^{\eta}u^{\eta}-D_{\xi}\\&=-g\left[\left(\xi_x^2+\xi_y^2\right)\frac{\partial H}{\partial \xi}\left(\xi_x\eta_x+\xi_y\eta_y\right)\frac{\partial H}{\partial \eta}\right]-\frac{C_d u^{\xi}}{hJ}\sqrt{\left(\eta_y u^{\xi}-\xi_y u^{\eta}\right)^2+\left(-\eta_x u^{\xi}-\xi_x u^{\eta}\right)^2}\end{aligned} \tag{2}$$

$$\begin{aligned}&\frac{\partial u^{\eta}}{\partial \tau}+(\xi_t+u^{\xi})\frac{\partial u^{\eta}}{\partial \xi}+(\eta_t+u^{\eta})\frac{\partial u^{\eta}}{\partial \eta}+\alpha_4 u^{\xi}u^{\xi}+\alpha_5 u^{\xi}u^{\eta}+\alpha_6 u^{\eta}u^{\eta}-D_{\eta}\\&=-g\left[\left(\eta_x^2+\eta_y^2\right)\frac{\partial H}{\partial \eta}\left(\xi_x\eta_x+\xi_y\eta_y\right)\frac{\partial H}{\partial \xi}\right]-\frac{C_d u^{\eta}}{hJ}\sqrt{\left(\eta_y u^{\xi}-\xi_y u^{\eta}\right)^2+\left(-\eta_x u^{\xi}-\xi_x u^{\eta}\right)^2}\end{aligned} \tag{3}$$

In the above equations, ξ and η = spatial coordinate components in the boundary-fitted coordinate system; τ = time coordinate (= t in the present model) in the coordinate system, x and y = spatial coordinate components in the Cartesian coordinate system; H = water surface elevation; g = gravitational acceleration; C_d= bed friction coefficient; J = Jacobian of the coordinate transformation given as $J = \tau_t\xi_x\eta_y + \xi_t\eta_x\tau_y + \eta_t\tau_x\xi_y - (\eta_t\xi_x\tau_y + \xi_t\tau_x\eta_y + \tau_t\xi_y\eta_x)$; u^{ξ} and u^{η} = contravariant components of flow velocity in the ξ and η directions defined as $u^{\xi} = \xi_x u + \xi_y v$ and $u^{\eta} = \eta_x u + \eta_y v$; u and v = depth-averaged velocity components in x and y directions; D_{ξ} and D_{η} are momentum diffusion terms in the ξ and η directions, which are described in Shimizu and Itakura (1991). Coefficients, a_1∼a_6, read as

$$\alpha_1=\xi_x\frac{\partial^2 x}{\partial \xi^2}+\xi_y\frac{\partial^2 y}{\partial \xi^2},\quad \alpha_2=2\left(\xi_x\frac{\partial^2 x}{\partial \xi\partial \eta}+\xi_y\frac{\partial^2 y}{\partial \xi\partial \eta}\right),\quad \alpha_3=\xi_x\frac{\partial^2 x}{\partial \eta^2}+\xi_y\frac{\partial^2 y}{\partial \eta^2} \tag{4a}$$

$$\alpha_4=\eta_x\frac{\partial^2 x}{\partial \xi^2}+\eta_y\frac{\partial^2 y}{\partial \xi^2},\quad \alpha_5=2\left(\eta_x\frac{\partial^2 x}{\partial \xi\partial \eta}+\eta_y\frac{\partial^2 y}{\partial \xi\partial \eta}\right),\quad \alpha_6=\eta_x\frac{\partial^2 x}{\partial \eta^2}+\eta_y\frac{\partial^2 y}{\partial \eta^2} \tag{4b}$$

2.2 Sediment Transport

Two-dimensional sediment continuity equation in a moving boundary-fitted coordinate system reads as

$$\frac{\partial}{\partial t}\left(\frac{z_b}{J}\right)+\frac{1}{1-\lambda}\left[\frac{\partial}{\partial \xi}\left(\frac{q_b^{\xi}}{J}\right)+\frac{\partial}{\partial \eta}\left(\frac{q_b^{\eta}}{J}\right)\right]=0 \tag{5}$$

in which Z_b = bed elevation; λ = porosity of the bed material; q_b^{ξ} and = q_b^{η} contravariant components of the bed load transport rate per unit width in the ξ and η directions, which are given by defining s in the direction of the stream line and n in the perpendicular to the streamline:

$$q_b^{\xi}=\frac{\partial \xi}{\partial s}q^s+\frac{\partial \xi}{\partial n}q^n=\left(\xi_x\frac{\partial x}{\partial s}+\xi_y\frac{\partial y}{\partial s}\right)q^s+\left(\xi_x\frac{\partial x}{\partial n}+\xi_y\frac{\partial y}{\partial n}\right)q^n \tag{6}$$

$$q_b^{\eta}=\frac{\partial \eta}{\partial s}q^s+\frac{\partial \eta}{\partial n}q^n=\left(\eta_x\frac{\partial x}{\partial s}+\eta_y\frac{\partial y}{\partial s}\right)q^s+\left(\eta_x\frac{\partial x}{\partial n}+\eta_y\frac{\partial y}{\partial n}\right)q^n \tag{7}$$

where q^s and q^n= the bed load transport rate components in the s and n directions, respectively.

The sediment transport rate in the streamline is calculated by using a Ashida and Michiue's formula (1972) modified by Hasegawa (2000) to explain the effect of the bank slope considering the gravitational effect on the side bank proposed by Kovacs and Parker (1994).

$$q^s=\frac{17}{\cos\theta_b}\tau_*^{3/2}\left(1-\frac{\tau_{*c}}{\tau_*}\right)\left[1-\sqrt{\frac{2\tau_{*c}\cos\theta_b}{\tau_*}}+2\left(\tan\theta_b-\frac{\partial z_b}{\partial s}\right)\right]\sqrt{\left(\frac{\rho_s}{\rho-1}\right)dg^3} \tag{8}$$

In this equation, θ_b = channel bed slope in the downstream direction; ρ_s= density of the bed material; τ_*= non-dimensional shear stress on the bed; τ_{*c} = non-dimensional critical shear stress on the bed, obtained by using Iwagaki's formula (1956).

Sediment transport rate, q^n , in the normal to the stream line proposed by Hasegawa (2000) is as follows

$$q^n=q^s\left(\frac{h}{r_s}N_*-\sqrt{\frac{2\tau_{*c}\cos\theta_b}{\tau_*}}\frac{\partial z_b}{\partial n}\right) \tag{9}$$

in which r_s = radius of curvature of a stream line; N_*= coefficient of the strength of secondary flow and assumed to be 7.0, proposed by Engelund (1974); μ_s= static friction coefficient of sand grain (=1.0); μ_k = kinetic friction coefficient of sand grain (=0.45).

The first term of the right hand side in Eq. (8) is the intensity of secondary flow in curved stream line due to the bank erosion and bed deformation, and the curvature of a streamline, $1/r_s$, is determined by the angle (=θ_s) between a streamline in the main stream direction and the x axis in the Cartesian coordinate:

$$\frac{1}{r_s}=\frac{\partial \theta_s}{\partial s}=\frac{\partial}{\partial s}\left[\tan^{-1}\left(\frac{v}{u}\right)\right]=\frac{\partial}{\partial T}\left[\tan^{-1}(T)\right]\frac{\partial T}{\partial s}=\frac{1}{1+T^2}\frac{\partial T}{\partial s} \tag{10}$$

In the above equation,

$$T=\frac{v}{u} \tag{11}$$

$$\frac{1}{1+T^2}=\frac{1}{1+\left(\frac{v}{u}\right)^2}=\frac{u^2}{u^2+v^2}=\frac{u^2}{V^2} \tag{12}$$

$$\frac{\partial T}{\partial s}=\frac{\partial}{\partial s}\left(\frac{v}{u}\right)=\frac{u\frac{\partial v}{\partial s}-v\frac{\partial u}{\partial s}}{u^2} \tag{13}$$

where $V=\sqrt{(u^2+v^2)}$.

The curvature of streamline, r_s, in the moving boundary-fitted coordinate system is given as follows

$$\begin{aligned}\frac{1}{r_s}=\frac{1}{V^3}\Bigg[&u^2\left(\xi_x\frac{\partial v}{\partial \xi}+\eta_x\frac{\partial v}{\partial \eta}\right)+uv\left(\xi_x\frac{\partial v}{\partial \xi}+\eta_y\frac{\partial v}{\partial \eta}\right)\\&-uv\left(\xi_x\frac{\partial u}{\partial \xi}+\eta_x\frac{\partial u}{\partial \eta}\right)-v^2\left(\xi_y\frac{\partial u}{\partial \xi}+\eta_y\frac{\partial u}{\partial \eta}\right)\Bigg]\end{aligned} \tag{14}$$

The channel bed slope in the s and n directions, $\partial z_b/\partial_s$

and $\partial z_b/\partial n$ used in Eqs. (8) and (9), are defined as

$$\frac{\partial z_b}{\partial s}=\frac{\partial z_b}{\partial \xi}\frac{\partial \xi}{\partial s}+\frac{\partial z_b}{\partial \eta}\frac{\partial \eta}{\partial s}=\frac{\partial z_b}{\partial \xi}\left(\xi_x\frac{\partial x}{\partial s}+\xi_y\frac{\partial y}{\partial s}\right)+\frac{\partial z_b}{\partial \eta}\left(\eta_x\frac{\partial x}{\partial s}+\eta_y\frac{\partial y}{\partial s}\right) \quad (15)$$

$$\frac{\partial z_b}{\partial n}=\frac{\partial z_b}{\partial \xi}\frac{\partial \xi}{\partial n}+\frac{\partial z_b}{\partial \eta}\frac{\partial \eta}{\partial n}=\frac{\partial z_b}{\partial \xi}\left(\xi_x\frac{\partial x}{\partial n}+\xi_y\frac{\partial y}{\partial n}\right)+\frac{\partial z_b}{\partial \eta}\left(\eta_x\frac{\partial x}{\partial n}+\eta_y\frac{\partial y}{\partial n}\right) \quad (16)$$

Non-dimensional bed shear stress, τ_*, in Eqs. (13) and (14), is given by

$$\tau_*=\frac{C_d V^2}{\left(\frac{\rho_s}{\rho-1}\right)gd}=\frac{C_d\left(u^2+v^2\right)}{\left(\frac{\rho_s}{\rho-1}\right)gd} \quad (17)$$

in which C_d = bed friction coefficient; d = mean diameter of the bed material.

3. Numerical Simulations

3.1 Numerical Method

The governing equations, i.e. continuity and momentum equations, for water flow are calculated numerically by using the finite difference method with computational grids in (ξ, η) coordinate systems. To solve Eqs. (2) and (3), a high-order Godunov scheme known as the Cubic Interpolated Psuedo-particle (CIP) method, proposed by Yabe et al. (1990), is employed.

3.2 Bank Erosion and Channel Migration

The computations for the flow and the bed evolution are conducted by using the governing equations described in the previous two sections. With special attention on the bed evolution near the banks, the deformation in the plane shape of the channel is calculated according to the following procedure proposed by Shimizu (2002).

3.3 Computational Procedures

The computational model applies the following process to calculate the changes in flow and the shape of a channel with time at infinitesimal intervals up to the designated time under the given initial conditions.

- Computing the depth-averaged flow field in the given plane shape of a water channel
- Computing the secondary flow perpendicular to the streamline of the depth-averaged flow
- Computing the sediment-transport rate and riverbed evolution
- The bank erosion, sediment deposition and shape alteration of a channel
- Setting a coordinate system along a new boundary and updating the computational data
- Updating time

3.4 Boundary Conditions

The flow velocities at the inlet and outlet boundaries are set to the same value for each grid square, and the data for the two grid squares with adjacent inlet and outlet boundaries are shared as a periodic boundary condition. At the sidewalls of emerged bars and banks, the no-slip condition is used in the transverse direction and a slip velocity is adopted in the stream-wise direction.

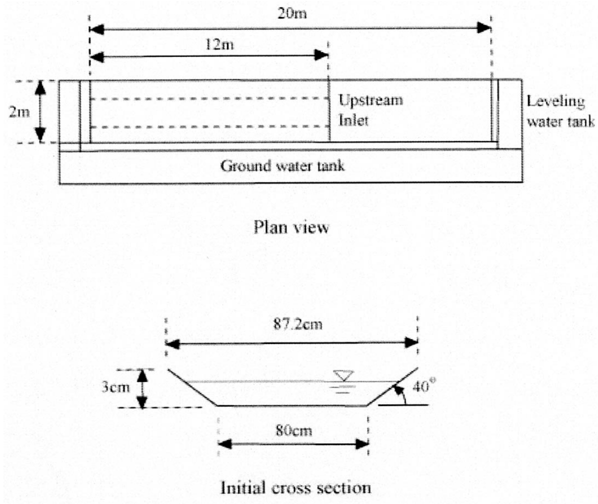

Fig. 1 Sketch of experimental setup

4. Comparisons of Experimental and Numerical Results

4.1 Experimental Procedure

Laboratory experiments were carried out in a flume with 12m in length and 2m in width, and sidewalls of the flume were wooden plates. The flume was filled with well sorted-sand with 1.25mm of mean diameter. An initial channel was set to trapezoidal shape with 80cm at the bottom, 3cm at the height, and 40 degree at the bank slope as shown in Fig.1. Water dis charge was controlled to 4.5 l/s and 3.5 l/s under the initial conditions. The non-dimensional channel (width/depth ratio) was 59.1 and 85.3, respectively.

Sediments were given by hand regularly at the upstream end of the channel to maintain the channel shape without local disturbance during the experiments. The bed was graded to the intended slope by using a wide scraper and the initial channel was cut by a wooden prototype attached to the lower end of the scraper at the same time. Before starting the experiments, a little water flowed over the bed to saturate the bed surface. Water depth was checked at some places using a point gage after 15 minutes of water flow. Plastic screen was set to maintain nearly steady state inflow into the channel at the front of the channel inlet, and water depth was regulated by sluice gate to minimize the outlet effect at the downstream of the channel. Bed configuration was measured using a laser bed profiler without water flow.

Although the channel length was 12m, the upstream 2m of the channel was excluded from the experimental reach since the bed and side banks were not changed due to the effects of inlet of the channel. The longitudinal bed profile was measured from 1m at the upstream to 8m at every 20cm in the experimental reach since the upstream 1m was not considered to avoid the effects of inlet and disturbance of water profile due to the sediment supply. The transverse bed profile was surveyed between 0.05m and 1.95m at every 1cm due to the moving limitation of the profiler.

4.2 Verification

The suggested numerical model is applied to verify the applicability at the actual experimental reach, 10m, with erodible banks. Experiment for Run-1 was carried out under the initial condition that the channel width to depth ratio corresponding to the transition regime (aspect ratio = 59.1) between alternate bars and braided bars, and for Run-2 was done corresponding to the condition for braided bras (aspect ratio = 89.9), according to the Kuroki and Kishi (1985)'s regime criteria on bars and braids in alluvial straight channels in the space given by dimensionless tractive force and the channel width to depth ratio considering the channel slope.

Fig. 2 shows that braided river is developed from meandering channel because aspect ratio is increased more and more while bank erosion is advanced on left and right alternately as time is progressed. The process and mechanism of braided river from an initial straight channel with erodible banks investigated by Ashmore (1982) were shown for Run-1. Alternate bars were grown up in the straight channel, and then a meandering channel was developed due to the side banks erosion by the flow deflection of the bars. Fig. 3 displays that a braided river is shown up from the initial stage and is remarkably developed while the channel width is enlarged since the side banks are eroded. And also the mechanism of braided river under large aspect ratio explained by Ashmore 1991) were illustrated for Run-2. Flow was concentrated into lower channels, and some of them were scoured deeply. Bars appeared to the downstream of the lower channels and complicated braided bars were developed due to flow division around the bars, leading to bank erosion.

The calculation results are in overall agreement with the experimental results, although the longitudinal wavelength and thalweg of the bed in the calculation are a little difference from those of the experiment.

Fig.4 displays the satisfactory agreement between the predicted results and the observed data for the bed change in the cross section for Run-1. However, the predicted results for Run-1 in Fig. 4(a) and (c) show that the deeply eroded bed is taken place in the vicinity of the left bank, while the bed in the laboratory experiment is eroded deeply near the right bank. The channel width for Run-2 is relatively simulated well with the experiment in Fig. 5(a). As time increase, however, the channel width for Run-2 is slightly underestimated, and the transverse bed change is overestimated in Fig. 5(b) and (c).

The difference between the numerical and experiment results is probably due to the fact that the numerical model does not simulate the three-dimensional flow structure at the confluence of the flow, and the numerical model does not exactly reflect unavoidable experimental error, e.g., water pump conditions and flow perturbation at the entrance of flow when the sand was fed, etc. Nevertheless, the numerical results are in relatively good agreement with the experimental data.

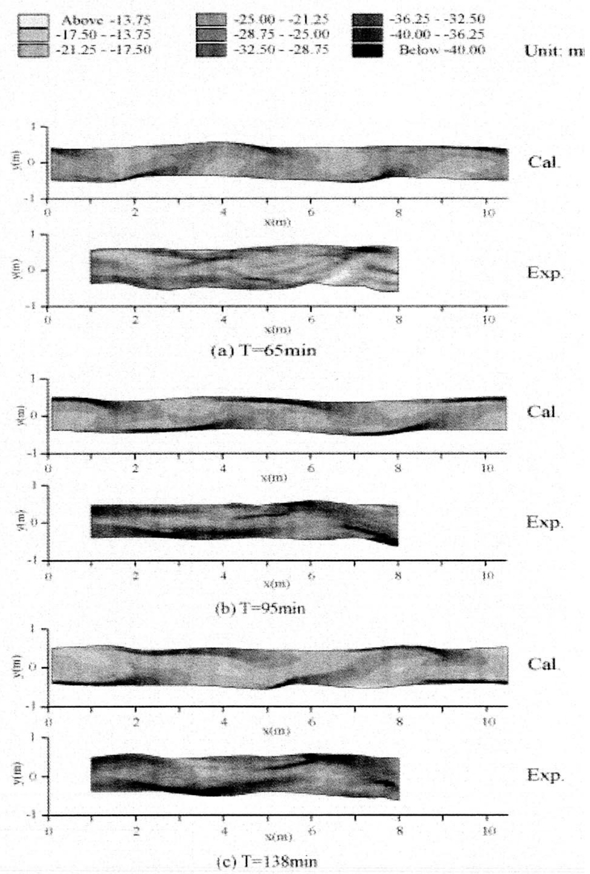

Fig.2 Comparison of the channel deformation between calculation and experiment for Run-1

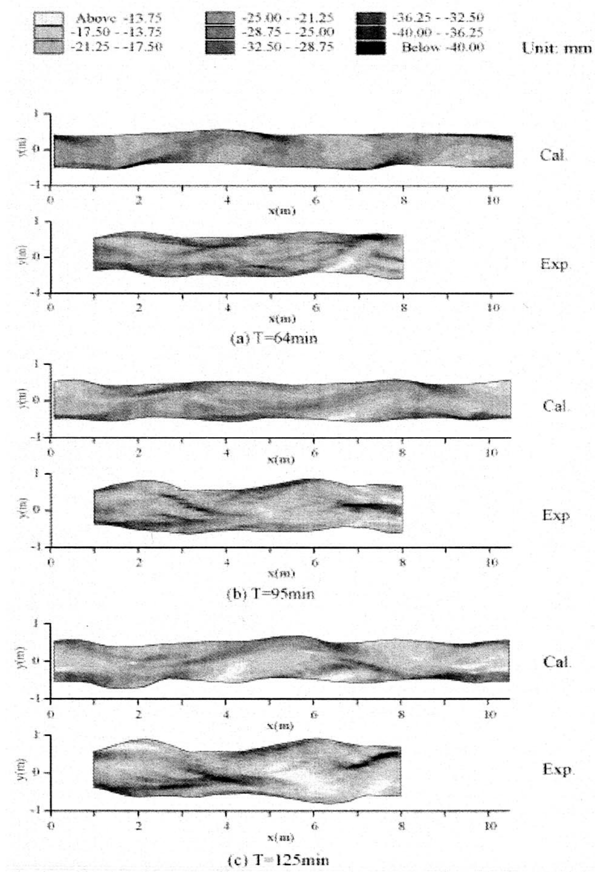

Fig.3 Comparison of channel deformation between calculation and experiment for Run-2

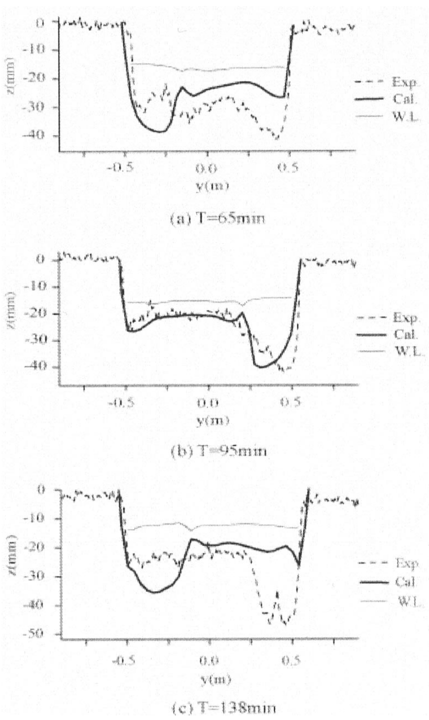

Fig.4 Comparison of cross sectional change between calculation and experiment for Run-1

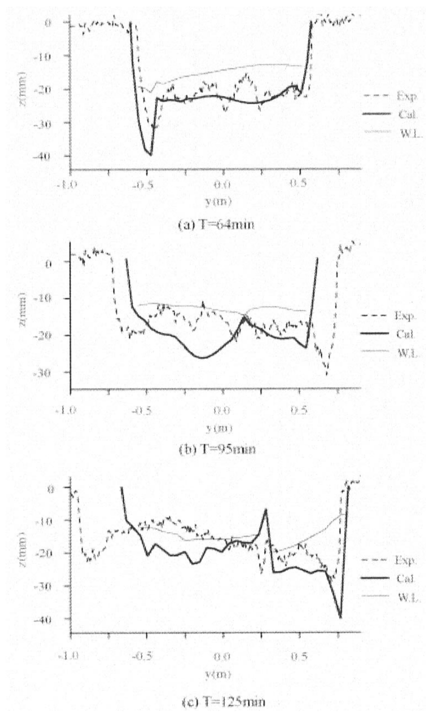

Fig.5 Comparison of cross sectional change between calculation and experiment for Run-2

5. Conclusions

In this paper, a model applicable to braided rivers with erodible banks has been presented to estimate to the channel evolution quantitatively. The model was verified by experiments on braided channel with erodible banks. The calculation results of the longitudinal change in time for Run-1 and Run-2 were relatively satisfied with the experimental results, although the longitudinal wavelength and thalweg of the bed in the calculation are slightly different from those of the experiment. The channel width was in good agreement between calculation and experiment for Run-1, while the width was underestimated for Run-2 as time increased. The comparison of cross sectional changes between calculation and experiment at 6m from upstream for Run-1 and Run-2, respectively, showed a little difference, because the model does not simulate the three-dimensional flow structure at the confluence. It is important that this presented model can simulate braided river with unconstrained banks, where previous numerical models have not been applied, although the model has this limitation.

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Study on Dynamic Control of Wuqiangxi Reservoir Water Level during Flood Season

Peng Cai-de
Mid-South Design & Research Institute for Hydroelectric Projects of CHECC, Changsha, Hunan, China

Abstract

The results of studying the flood features of the Yuanshui River and findings of analyzing the flood control effect by lowering the Wuqiangxi Reservoir flood control level on the power generation benefits have indicated that the reduce of 98m design flood control level of Wuqiangxi Reservoir down to 94-96m would bring about a great influence on the power generation operation and a limited increase of flood control function. The calculations have shown that reducing 3-5m initial water level of Wuqiangxi Reservoir by applying forecasting and pre-discharging in the forecast period could take an active role in flood control. For this reason, this paper outlines a use of dynamic control of water level at different periods upon the meteorological and hydrological conditions as well as flood forecasting and pre-discharging, and a better coordination between flood control and power generation.

Key Words: *flood control level; flood forecasting and pre-discharge; regulation by stage; reservoir operation; Wuqiangxi Reservoir*

The Wuqiangxi Hydropower Station, located on the lower reaches of the Yuanshui River within Yuanling County, has a catchment area of 83,800km^2 above the dam site, accounting for 93% of the total riverbasin area. The mean annual flow at the dam site is 2040m^3/s, and the annual runoff 64.3 billion m^3. The project has been developed mainly for power generation, concurrently with multipurpose benefits of flood control and navigation improvement. The Wuqiangxi reservoir has a normal pool level of 108m, with a corresponding storage capacity of 3.05 billion m^3; a design flood storage capacity of 1.36 billion m^3; and a flood control level of 98m. The design flood control period is from May to July. The reservoir operates jointly with the Fengtan Reservoir which has a flood storage capacity of 0.28 billion m^3. In case the safe discharge in the downstream river course is 20,000m^3/s, the reservoir operation can raise the flood control standard in the downstream river course to a one-in-20 year flood. The dam is designed to resist a one-in-1000 year flood with a flood level of 111.62m, and is checked based on a one-in-10,000-year flood with a flood level of 114.70m. The hydropower station has a total installed capacity of 1200 MW, a firm output of 255MW, and a mean annual power generation of 5,370 GWh. The hydro complex mainly consists of a dam, a powerhouse, a flood-release structure, and a shiplock. The dam is a concrete gravity one with a crest elevation of 117.5m, a max. height of 87.5m, and a total crest length of 724.4m. The spillway section, 249.75m long in total, is arranged with 9 surface bays, 1 high-level outlet, and 5 low-level outlets. At the surface bay, the weir crest is at El. 87.80m, and the radial gate is 19m wide by 23.3m high. The inlet inverts at the high-level outlet and low-level outlet are at El.76m and El.67m respectively. The high-level outlet and low-level outlet are 9m wide by 13m high and 3.5m wide by 7m high respectively. The shiplock is arranged at the left bank, and the powerhouse is arranged at the dam toe on the right bank.

1. Analysis on Flood Characteristics of the Yuanshui River

The Yuanshui River lies in the subtropical monsoon climatic zone, warm, wet and rainy. The rainy season in the Yuanshui riverbasin normally starts from late March to early April, followed by rainstorm. Due to the effect of westerlies such as upper low trough, low vorticity, and shearing line, and ground cold front or stationary front, the large-area, long-lasting, high-intensity rainstorms often occur in June and July, forming big floods in the riverbasin, such as the big floods in the years of 1969, 1970, 1995, 1996, 1998 and 1999. The historic catastrophic floods in 1766 and 1911 also occurred in June or July. The occurrence of yearly max. flood in different month from 1925 to 1999 (absent for 1945 and 1946) at the Wuqiangxi damsite is shown in Table 1. Table 1 indicates that number of big flood occurrences accounts for 80.8% of the total from May to July and 61.6% from June to July, and the peak flood discharge greater than 30,000m^3/s mostly occurred in June and July (see Table 2).

Table 1. Occurrence of Yearly Max. Flood in Different Month at Wuqiangxi Damsite

Description	April	May	June	July	August	September	October	Total
Number of occurrence	4	14	20	25	8	1	1	73
Probability（%）	5.5	19.2	27.4	34.2	10.9	1.4	1.4	100

Note：The years included in the statistics are 1925 through 1944, and 1947 through 1999.

Table 2. Occurrence of Various Discharges in Different Months at Wuqiangxi Damsite

Peak flood discharge（m^3/s）	Number of occurrences in different month					
	May	June	July	August	September	October
⩾30000	0	2	6	0	0	0
⩾25000	0	4	8	1	0	0
⩾20000	0	7	13	4	0	1

Note：The years included in the statistics are 1925 through 1944, and 1947 through 1999.

As the monsoon comes slowly and quits quickly, the flood characteristics before and after August are quite different. The floods in June and July are often characterized by high peak, large volume, long duration and multiple-peak hydrograph. The floods in and after August are often characterized by high peak, low volume, short duration, and single-peak hydrograph.

2. Reservoir Operation Scheme in Flood Season

The Yuanshui River is rich in water resources, with an annual runoff up to 64.3 billion m^3, secondary to the Minjiang River among the tributaries of the Yangtze River. In addition, the floods are of high peak and large volume, and rush down with a terrifying force. Therefore, the design flood control capacity of 1.36 billion m^3 of the Wuqiangxi Reservoir can play a limited role in the flood control. Based on this, the proposed reservoir operation principles for flood control are: when the reservoir water level is below 108m, the reservoir will operate to meet the flood control requirements in the downstream areas, i.e., the dam retains the flood for the downstream area and controls the discharge in the downstream river channel not exceeding 20,000 m^3/s (equivalent to a flood control standard of one-in-20-year flood); and when the reservoir water level reaches 108m, the reservoir will operate to meet the dam safety requirements, i.e., when the flood control standard of one-in-20-year flood is exceeded, the Wuqiangxi Reservoir will not operate to specially retain flood for the downstream areas, and will release flood based on the inflow.

The Youshui River, a tributary of the Yuanshui River, lies in the north of the Yuanshui Riverbasin. The Fengtan Reservoir, located on the lower reaches of the Youshui River, has a catchment area of 17500km^2 above the dam site, accounting for 95.1% of the total of the riverbasin. Its design flood control task is, in collaboration with the Wuqiangxi Reservoir, to provide flood control for the downstream areas of the Yuanshui River. Based on the strike of rainstorm in the riverbasin, as for the same flood, the peak flood formed by rainstorm in the Youshui Riverbasin normally arrives at Yuanling earlier than that formed by rainfall on the main stream upstream of Pushi. In addition, as the flood in the Fengtan Reservoir is directly discharged into the Wuqiangxi Reservoir and regulated in the Wuqiangxi Reservoir, the flood volume, other than the flood peak, plays a key role in the flood hydrograph. Hence, joint flood control operation of the Fengtan and Wuqiangxi reservoirs should focus on making the flood peak in the Youshui River flow through the Fengtan dam as quickly as possible, and through the Wuqiangxi dam as soon as possible on the premise of that the discharge is allowed in the downstream river course; and earlier flood retention is not appropriate so as to evacuate the reservoir for accommodating the subsequent floods.

3. Effects of Lowering the Flood Control Level of the Wuqiangxi Reservoir on Power Generation

After the 1996 big flood, the relevant department of Hunan Provincial Government, in view of flood control, regulates that the flood control level of the Wuqiangxi Reservoir should be lowered from 98m (designed) to 94-96m, which would bring about limited benefits for flood control, and considerable negative impacts on power generation.

3.1 Analysis on the effectiveness of lowering the flood control level to increase the flood control capacity

The flood-release structures and the rated head of turbo-generator unit of Wuqiangxi Hydropower Station were all designed based on the flood control level of 98m. The spillway crest is at El.87.80m. If the flood control level is lowered, the flood-release capacity will also be lowered due to the head reduce. When the flood control level is lowered from 98m to 96m and 94m, the total flood release capacity will be considerably lowered by 3489 m3/s and 6495 m^3/s respectively (see Table 3).

The effects of different flood control level on max. initial water level have been analyzed for the Wuqiangxi Reservoir based on the max. control discharge (with the same peak clipping rate) for the one-in-20-year flood, one-in-50-year flood, and one-in-100-year flood. For the results, see Table 4.

As the flood in the Yuanshui River rushes down in a terrifying force, and the big flood is characterized with long duration (7 to 11 days), multiple peaks and large volume (the 7-day and 11-day max flood volumes were up to 11.9 billion m^3 and 15.4 billion m^3 respectively at Wuqiangxi based on the recorded data), the Wuqiangxi hydropower station should be used mainly for flood control, peak clipping, and peak staggering, rather than for flood retention. As restricted by insufficient flood release capacity at low water level, the flood control capacity obtained by lowering the flood control level during big flood is mostly ineffective flood retention (which may be safely discharged through the river channel), playing a very limited role in regulating the whole flood. The analysis indicates that lowering the flood control level to less than 98m will play some role in flood retention for small to medium-sized flood within a one-in-10-year flood, but very limited role in flood control for a one-in-20-year flood or above. If a big flood or catastrophic flood of one-in-20-year flood or above occurs, lowering the flood control level to 96-94m will have little impacts on the max. initial water level of the reservoir. For the small floods, lowering the flood control level to 96-94m can have some effect on flood control. However this flood control requirements can be met without lowering the flood control level of the reservoir.

3.2 Lowering the flood control level is unfavorable for the efficient and stable operation of the power generating unit

The designed rated head for the Wuqiangxi Hydropower Station is 44.5m. At the flood control level of 98m, the power generating unit can deliver the rated output. If the flood control level is lowered from 98m to 96m and 94m, the power generating unit can not deliver the rated output respectively by 80MW and 160MW, resulting in a big loss in power generation in flood season. In addition, due to the low flood control level, the power generating unit will operate under low head and high water consumption rate for a long period of time, which would not only result in power loss, but also cavitation, abrupt vibration, and unsafe and unsteady operation.

Table 3. Flood Release Capacity at the Lowered Flood Control Level

Flood control level (m)	9 surface bays + 5 low-level outlets + 1 high-level outlet (m^3/s)	Increase in flood control capacity (million m^3)
98	14716	0
96	11227	180
94	8221	348

Note: The power discharge has not been included in the table. The rated power discharge of each unit is 627 m^3/s (5 units in total). The flood routing calculation has given some allowance, 2500 m^3/s on average.

Table 4. Flood Routing Results

Flood standard	Max. control discharge (m^3/s)	Max. initial water level under different flood control level/m (difference form the designed value)			
		98m(designed)	97m	96m	94m
One-in-20-year flood	20,000	107.99	107.96	107.94	107.92
		0	-0.03	-0.05	-0.07
One-in-50-year flood	26,000	108.97	108.95	108.95	108.94
		0	-0.02	-0.02	-0.03
One-in-100-year flood	28,000	110.64	110.63	110.63	110.63
		0	-0.01	-0.01	-0.01

4. Dynamic Control of the Flood Control Level of Wuqiangxi Reservoir

4.1 Automatic water regime measuring and forecasting System

The automatic water regime measuring and forecasting systems of Wuqiangxi and Fengtan Reservoirs were established in 1995 and 1993 respectively, controlling a catchment area of 26,000km^2 from Anjiang to Wuqiangxi in the Yuanshui Riverbasin and 17,500km^2 in the Youshui Riverbasin, accounting for 52% of the total riverbasin area above Wuqiangxi damsite. The two systems comprise 70 telemetric precipitation stations, 13 gaging and precipitation stations, 8 relay stations, and two central stations, materializing the functions of telemetering the rainfall and water regime, forecasting the water regime through computer network, and formulating the reservoir operation scheme through computer network. The 10 -year actual operation of the two systems indicates that each system boasts an access rate greater than 96%, an acceptance rate greater than 92% for hydrological forecasting, and a response speed of 8 min., meeting the dispatching and operation requirements, playing a critical role in reservoir operation for the big floods in 1990s, and creating a favorable condition for water regime forecasting and reservoir operation as well as dynamic control of the flood control level for the Wuqiangxi Reservoir.

4.2 Analysis on forecasting and pre-discharging capacity

Due to the different distribution of rainstorm, the floods may be roughly divided into 4 types, namely the upper reaches, the upper and middle reaches, the lower and middle reaches, and the whole riverbasin. Analysis on the forecast period for the four types of floods and estimate on the pre-discharging capacity of the Wuqiangxi Reservoir are shown in Table 5. The calculation results show that the Wuqiangxi Reservoir is very capable in pre-discharging, and is capable of lowering the initial water level by 3 to 5m through forecasting and pre-discharging within the flood forecast period, achieving flood control by lowering the flood control level.

Therefore, with the help of the automatic water regime measuring and forecasting system, forecasting the big floods and pre-discharging within the forecast period may effectively lower the actual initial water level, obtaining considerable multipurpose benefits. However, high attention should be paid to the extent of lowering the flood control level based on the flood forecast results, so as not to bring about difficulty in restoring water in the reservoir. The following points should be highlighted:

[1] When catastrophic rainstorm is forecast to occur, pre-discharging to evacuate storage may be adopted so as to properly lower the reservoir water level (to below the design flood control level). Rigoriously lowering the reservoir water level should be avoided regardless of the weather conditions and tendency of water regime.

[2] As the wet season varies in the time of start and end each year, the reservoir operation level should be raised by stages in proper time based on the hydrologic and meteorological law of the riverbasin and long- and medium-term weather forecast, so as to retain the late flood.

[3] As the hydrological and meteorological characteristics of the riverbasin are quite different in time and space, the flood control level and time period should be determined based on the staged flood characteristics so as to achieve the dynamic control.

[4] As the rainfall and water regime varies greatly and quickly, levelled dispatching mechanism is suitable to be set up so as to improve the realtime and forecastibility of dispatch.

4.3 Reservoir control level in April and May

The recorded hydrological data since 1925 indicates that floods in May are mostly of medium to small size, and big floods are mostly in June and July (accounting for 85% and above). The historic catastrophic floods were all in June and July. Based on the reference to historic records and investigation on the historic floods as well as analysis on recorded data, since 1189, there were 5 recorded catastrophic floods of 40,000m^3/s order occurring in the Yuanshui River (in

Table 5 Estimates on the Forecasting and Pre-discharging Capacity of Wuqiangxi Reservoi

Type of Flood	Upper reaches	Upper and middle reaches	Lower and middle reaches	Whole riverbasin
Flood forecast period (h)	17	12	6	10
Average pre-discharge (m^3/s)	11400	12600	14400	13200
Base inflow (m^3/s)	3000	3000	3000	3000
Average possible pre-discharge (m^3/s)	8400	9600	11400	10200
Possible pre-discharge volume (10^6m^3)	514	415	247	367
Achievable initial water level (m)	92.6	93.6	95.4	94.1

1189, 1571, 1618, 1766 and 1996), all in the period from mid June to mid July, and mostly of whole river-basin type. Within 1900s, there were 10 big floods of 30,000m^3/s or above (in 1911, 1912, 1926, 1931, 1933, 1935, 1995, 1996, 1998 and 1999), all occurring in the period from mid June to mid July. Therefore, the flood control for the Yuanshui Riverbasin should focus on June and July.

Prior to the end of May, the floods in the Yuanshui River are mostly below 20,000m^3/s, less than the safe discharge in the downstream river course (20,500 to 24,000 m^3/s, and 23,000 to 26,600m^3/s after completion of the Dongting Lake Improvement Project Phase II). In April, flood occurrence is much less, with low magnitude. In this period, the flood in the Yuanshui River seldom meets with the flood in the Yangtze River, and the water level in the Dongting Lake has little impacts on that in the Yuanshui river course. Therefore, the river course boasts a big safe discharge and a relatively light flood control task. Sometimes, a long period of rainfall interval may occur. For example, in May of 1999 and 2000, a 15-day rainfall-free period occurred, which might affect the normal operation of the power station due to water shortage. In view of coordination between flood control and power generation, the reservoir water level may be raised properly. Hence, in the early wet season April, the reservoir may operate regardless of the flood control level (the design flood control level period excludes April), and the reservoir may operate at any water level below 108m so as to favor power generation and dispatch. In May, some flood control room should be considered for the reservoir, and the reservoir may operate at a water level of 102m or so. However, prior to the end of May, the reservoir water level should gradually draw-down to 98m. The reservoir operation may be made based on the meteorological and hydrological forecast. In case the water level at Chenglingji is high or big flood may occur, the unified dispatch from Hunan Provincial Flood Control Headquarter should be observed, and the reservoir water level should be lowered to 98m or below by means of forecasting and pre-discharging in a short period of time before the inflow flood comes.

4.4 Reservoir control level in June and July

The Yuanshui riverbasin lies in 26-30˚N and 107-112˚E , within the major rain band of the the Yangtze River plum rain frontal system, where wide-scope, high-intensity and long-lasting rainstorm often occurs. The common characteristics of plum rains cyclone is that there exists blocking in the 500hPa high latitude area, and a plum rains trough nearby 110˚E , with the west Pacific subtropical high pressure belt and Qing-Zang high pressure belt on its two sides. The stability and magnitude of the plum rains are closely related to the location of the west Pacific subtropical high pressure belt (crest line). Plum rains normally occur in mid June to mid July. The plum rains start in early June at the earliest and end in late July at the latest. Normally the last plum rain forms the biggest flood in the year. As shown on the 500hPa map, when the west Pacific subtropical high pressure belt (115-120˚E) is south of 25˚N , rainfall is plentiful. In June, the west Pacific subtropical high pressure belt is around 20˚N on average. When it first arrives at 20˚N, the plum rains begin, and when it is firmly out of 25˚N, the plum rains end, when the rain band moves to Huanghuai area and North China. One of the essential condition for occurrence of heavy rainstorm in a large area in the Yuanshui River-basin is that the west Pacific subtropical high pressure belt is located at 20-25˚N. At the transfer of spring and summer, the first arrival of the west Pacific subtropical high pressure belt is early to mid June. If the tropical cyclone is very strong or the high pressure of high latitude blocking is weak, the plum rains season will end ahead of time, or void plum rains may occur, i.e., no heavy rainstorm will occur and drought may come.

The most dangerous flood in the Yuanshui River-basin is the flood caused by plum rains. The Yangtze River plum rains frontal system is the main contributor. The flood is mostly characterized with high peak and large volume, often meeting with the flood in the main stream of the Yangtze River. The flood volume in the Yuanshui River in flood season accounts for 45 to 55% of the total in the four rivers of Xiangjiang, Zishui, Yuanshui and Lishui. The recent 6 catastrophic floods all met with the floods in the main stream of the Yangtze River.

The above analysis shows that June to July is the main flood season for the Yuanshui River, with high peak and large volume, often meeting with the floods in the Yangtze River and the Dongting Lake. As the flood control situation is rigorous, the reservoir should operate at the flood control level of 98m. In combination with flood forecasting and pre-discharging, the initial water level may be further lowered.

It should be stressed that the reservoir operation in July is very critical. The water regime in July is very complex and uncertain. July, on one hand, is the main period of big flood, and on the other hand, the transient month from wet season to dry season. In dry years, if the reservoir does not fill in July, the reservoir can not be filled to the normal pool level in August or no water is available for filling the reservoir, which would adversely affect the power output of the power station in dry season. So the reservoir filling opportunity should be well grasped. The above analysis indicates that once big flood occurs in the Yuanshui River in late July, it must be attributable to the extension of the plum rains period in the Yangtze Riverbasin, and direct meeting with the floods in the upper and middle reaches of the Yangtze River, as obviously shown from the water level and water regime in Yangtze River and the Dongting Lake. The crest line of west Pacific subtropical high pressure belt also lies south of 25˚N as shown from the 500hPa map. The big floods occurred in late July in 1931, 1954 and 1998 were all of this kind. This kind of flood may be basically identified or forecast in mid July based on the current water regime and weather situation. In some dry years, void plum rains occur or plum rains season ends in early July ahead of the time, such as the years 1941, 1956, 1959, 1961, 1972, 1978, 1981, and 2000. When the crest line of west Pacific subtropical high pressure belt lies north of 25°N as shown from the 500hPa map, the dry situation appears, so reservoir filling should start in mid July. As the situation in July is very complicated and special, July is the key month for coordination between flood control and power generation for the Wuqiangxi Reservoir. The meteorological, water regime and reservoir operation departments should analyze and communicate in the weather situation on daily basis, closely monitoring the weather tendency in a large scope, monitoring the rainfall and water regime in the river-basin, and strengthening communication. Once the plum rains period ends, reservoir filling should be decided to start immediately. It is not proper to keep the flood control level until the end of July regardless of the actual situation. Decision should be made based on the actual situation to achieve a dynamic control of reservoir water level so as to make use of advantages and avoid disadvantages, and maximize the multipurpose benefits of the power station.

4.5 Reservoir control level in and after August

The late flood period starts in August. Although floods may occur in this period, the floods are mostly characterized with low magnitude, short-duration, single peak, high peak, low volume, no direct contact with the flood in the Yangtze River, and minor harm. If period of flood control level is extended for this, the power station would be subject to considerable power loss, which is economically unreasonable. So the period of design flood control level excludes this period. Therefore, water regime forecasting should be strengthened when August comes so as to filling the reservoir in a timely manner. In view of the requirements on overall flood control situation of the Dongting Lake and reservoir filling for power generation, the comprehensive analysis results show that the water level of the Wuqiangxi Reservoir should be controlled as: 102m in early August, 104 to 106m in mid August, and 108m (NPL) in late August.

5. Conclusions

The design flood control level of the reservoir has been determined based on the available hydrological law over the years, which is a comprehensive parameter. The actual application of the design flood control level should be subject to dynamic control by time periods in combination with meteorological and hydrologic conditions, so as to better coordinate the conflicts between flood control and power generation, and bring the multipurpose benefits of power generation and flood control into full play. The actual operation of the Wuqiangxi Hydropower Station over the recent years indicates that dynamic control of the flood control level by time periods is fully feasible, with remarkable effects, which should be extended for application.

Reference

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Application of Gravel Soil Impermeable Material in High Earth-rockfill Dam

Yu Ting
Chengdu Hydroelectric Investigation & Design Institute of China
Hydropower Consulting Group, China

Abstract

Gravel soil used as impermeable material in high earth core rockfill dam has been the trend today. According to experiences from high gravel soil core rockfill dams designed in recent years in Sichuan province, China, measures for improving gravel soil quality, filter design for gravel soil as impermeable material and compactive efforts in gravel soil tests are discussed.

Keywords: *gravel soil; high earth-rockfill dam; coarse grain removing; filter design; compactive effort*

1. INTRODUCTION

In past dozens of years, earth core rockfill dam has been become one of the main dam types in high dam construction in the world. According to statistics in early nineties, earth core rockfill dams account for 55.5 % of the dams existing, under construction and higher than 230m in the world. From then on, high earth core rockfill dams have developed rapidly in China. In past few years, the "leading" reservoir projects planned in some large and medium rivers in Sichuan province, China are generally characterized with high dam (higher than 100m, some even up to 300m), deep overburden in river bed and natural impermeable materials available around the projects. Therefore, earth core rockfill dam is the main dam type researched for these projects. In the design of earth core rockill dam, it is very important to choose and design impermeable earth material. Practical experiences from these projects indicate that gravel soil used as impermeable material in high earth core rockfill dam has been the trend today. In the earth core rockfill dams higher than 200m,existing and under construction in the world, gravel soil is used as impervious core material mostly. In China, most high earth core rockfill dams under construction and design in recent years use gravel soil as impervious material. In Sichuan province, such projects include Pubugou, Qiaoqi, Shuiniujia under construction and Shiziping under design. Application of gravel soil in above-mentioned projects and research findings are presented in the paper.

2. APPLICATION OF GRAVEL SOIL IMPERMEABLE MATERIAL IN HIGH EARTH-ROCKFILL DAM

In general, comparing with pure cohesive soil, gravel soil can meet requirement of imperviousness. Besides, it also has following properties: (1) high deformation modulus and low compressibility which coincides with deformation of dam shell material, avoiding or decreasing core cracking and hydraulic splitting caused by arch effect; (2) in case of core cracking, coarse grains on joint plane will restrict crack scouring and extending and this is favourable to natural repair of crack; (3) not sensitive to water content in embankment material, making easier construction; (4) high bearing capacity, convenient for operation of heavy construction equipment. Several high gravel soil core rockfill dams are studied deeply as shown in Table 1. Some unique opinions are presented in aspects of engineering properties, adaptability of impermeable material in high earth and rockfill dam and earth material design for gravel soil.

2.1 Measures for Improving Gravel Soil Quality

From experiences, it is suitable to adopt 50% grains smaller than 5mm and 20% grains smaller than 0.1mm in gravel soil. However, some natural gravel soil includes less fine grains and imperviousness and impermeability can not be satisfied completely, while other natural gravel soil includes much more fine grains and strength can not meet the requirements of high dam. Therefore, engineering measures must be taken to these gravel soils to improve their quality. Quality improvement for the former gravel soil is described here mainly.

2.1.1 Pubugou Hydropower Project

At preliminary design stage, quality improvement measures are determined to be taken in diluvium soil area at Heima I Area after a large number of laboratory and field tests and in-situ compaction tests carried out, namely coarse grains larger than 80mm should be removed from earth material. Main test results before and after removing are shown in the Table 1. The test

Table 1. Properties of gravel soil impermeable material in some projects

Characteristics \ Project		Jinping I	Pubugou	Shiziping	Qiaoqi	Shuiniujia
Design Phase		Dam type comparing	construction	Feasibility study	construction	construction
Dam height（m）		295	186	136	123	110
Borrow Pit		Wali VI Area	Heima I Area	Changheba	Kari village	Shuiniujia Area B
Earth Material Genesis		talus	diluvium	moraine	Periglacial freezing-thawing	Residual and talus
Grain analysis（%）	<5mm	54.30	48.19	33.62	46.68	68.09
	<0.1mm	36.10	19.00	15.64	19.62	48.11
	<0.005mm	12.70	4.40	5.36	10.64	9.99
Main mechanical properties	ρ_d (g/cm^3)	1.99	2.25	2.21	2.14	2.02
	C(MPa)	0.02	0.02	0.05	0.03	0.05
	$\Phi(^o)$	20.18	32.90	36.00	24.50	23.60
	k_{20}	3.56×10^{-7}	4.40×10^{-5}	4.83×10^{-6}	3.68×10^{-7}	2.78×10^{-7}
	i_k	6.96	0.96	1.75	4.59	7.56
	i_f	12.56	4.04	4.75	9.86	13.42
Earth material quality improvement measures and results	measures	/	Grains >80mm are removed	Grains >60mm are removed	/	/
	<5mm		49.76	42.33		
	<0.1mm		21.77	19.69		
	<0.005mm		5.50	6.74		
	k_{20}		7×10^{-6}	2.8×10^{-7}		
	i_k		2.6～3.1	4.75		
	i_f		8.3～11.5	12.83		

Table 2. Test results of coarse grain removing from earth material in Heima 0 Area at Pubugou project

characteristics \ Earth condition		Natural gradation material	80mm grains are removed	60mm grains are removed
Grain analysis（%）	<5mm	37.11	45.32	49.94
	<0.075mm	12.85	15.73	17.29
	<0.005mm	2.33	2.88	3.15
Main mechanical properties	ρ_d (g/cm^3)	2.34	2.35	2.35
	C(MPa)	0.025	0.05	0.06
	$\Phi(^o)$	38.22	35.00	38.65
	k_{20}	1.2×10^{-4}	4.18×10^{-6}	1.3×10^{-6}
	i_k	0.88		
	i_f	3.99	4.19	5.30

results show that earth gradation is improved after removing and meets the requirements of grain size distribution of impermeable material from viewpoint of engineers. The earth material is classified as G_P before removing and changes into G_C after removing, thus its nature transforms from stone material into soil material and its mechanical property meets the requirements of impermeable material for high dam.

After construction began, in order to enlarge source of impermeable earth material, quality improvement measures (coarse grain removing method) are studied for earth material in Heima 0 Area. Due to coarse grain content in Heima 0 Area is higher than that in Heima I Area, coarse grain larger than 80mm removing scheme and 60mm removing scheme are studied respectively. The main test results for earth material before and after removing is shown in Table 2. The test results indicate that earth material screened off 60mm grains are better than that screened off 80mm grains from point of view of grain gradation and impermeability and easy to control construction quality. Therefore, 60mm grains removing scheme is adopted finally.

2.1.2 Shiziping Hydropower Project

In Changheba borrow pit at Shiziping project, earth material with natural grain sizes rather coarse can not be directly used as impermeable material for high dam according to practical engineering experiences. But a large number of test results indicate that its permeability coefficient is at magnitude of 10^{-6} after compaction. This may be caused by appropriate gradation of coarse and fine grains, higher content of cohesive grains in earth material (account for 16% in grains smaller than 5mm). In addition, earth material distribution is uniform and variation between upper and lower envelope curves for grain graduation is small (Fig.1).

In order to get further understanding of the earth material, ρ_{dmax}~P_5~ϖ_{op} curve for earth material is plotted against test results shown in Fig.2. For P_5>40%, compacted dry density of fine soil with grain size smaller than 5mm starts to decrease, indicating the first characteristic point of coarse grain content P_5=40%; For P_5=70%, dry density of earth material reaches the maximum value, but when coarse grain content increases from P_5=50% to P_5=70%, dry density of earth material increases slowly ; For P_5=60%,

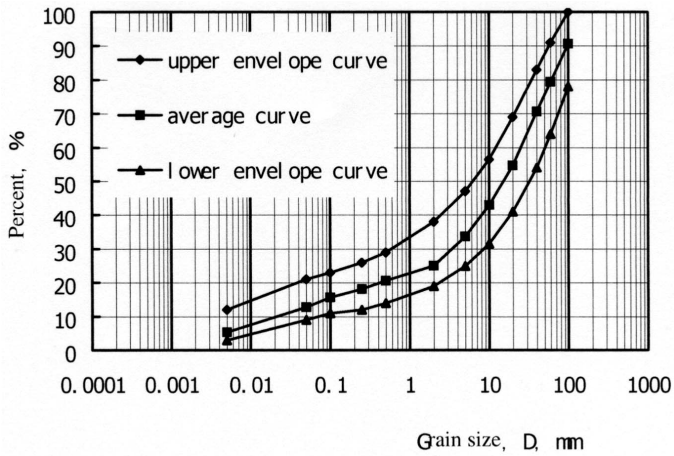

Fig.1 Envelope curves of grain gradation for whole borrow pit in Changheba debris area at Shiziping project

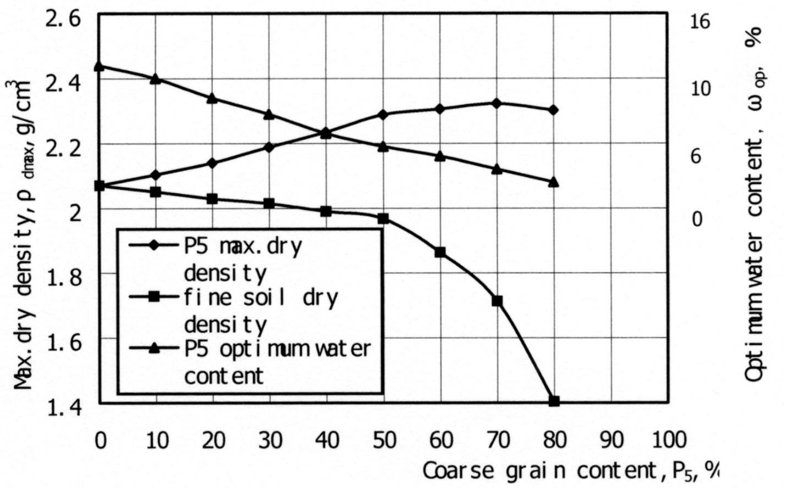

Fig.2 ρ_{dmax}~P_5~ϖ_{op} Curve for impermeable material in Changheba debris area at Shiziping project under comparative effort (2740kJ/m³) compacted dry density of fine soil is 10% lower than that for $P_5=0\%$. Fine grains can not fully fill into voids between coarse grains in earth material, indicating the second characteristic point of coarse grain content $P_5=60\%$; Because $P_5=66.38\%$ for natural grading earth material and impermeability of earth is lower, coarse grains larger than 60mm are removed from the earth material. $P_5=57.67\%$ is reached for earth after removing, thus imperviousness and impermeability improves remarkably (Table 1).

In order to verify laboratory test results, screening test is carried out in the field. Coarse grains larger than 60mm are removed from the earth material by SZ_2 vibration screen. Screening is carried out with good results and there is only very small portion of powder and cohesive grains in the removed material. Worries about this problem disappear. At the same time, we found the influence of water content in gravel soil passing on screening results is less, indicating that coarse grain removing method is an effective way to improve gradation and performance of gravel soil.

2.2 Filter Design for Gravel Soil Impermeable Material

Nonuniformity coefficient of gravel soil is large generally. Protection of fine grains(below 2mm normally) is considered mainly in filter design. Natural sand and gravel, crushed sand and stone or the mixture of two are adopted as filter material.According to test results of some projects in Table 3, filter design for gravel soil as impermeable material should meet design criterion. Furthermore, the maximum grain size of filter material should be limited below 40mm and average grain size should range from 1mm to 3mm. By reasonable filter design, percolation gradient of gravel soil with filter material protection increases 10 times as many as that without filter material.

2.3 Compactive Efforts in Gravel Soil Impermeable Material Tests

In Table 4, it is shown that earth material compacted density is improved by increasing its compactive efforts and its mechanical property is improved consequently. Gravel soil is characterized with higher strength and lower imperviousness and impermeability which is unfavorable,even can not meet design requirements, so it is necessary to improve its imperviousness and impermeability by increasing compactive efforts. On the other hand, some new roller compacted equipment with high power are used frequently in dam construction in recent years in order to improve dam placing intensity, thus shorten construction period. Some experiences from construction of high earth and rockfill dams show that compaction criterion determined by standard Proctor compactive effort

Table 3. Filter design for gravel soil impermeable material in some projects

characteristics \ Project		Pubugou	Shiziping	Qiaoqi	Shuiniujia
Filter material sources		Crushed sand and stone	Crushed sand and stone	Mixture	Natural sand and gravel
Filter material control size(mm)	D_{max}		20	40	40
	D_{50}		3	2	1
	D_{15}		0.7	0.47	0.22
Filter protection effect	i_k	16.44	22.50	55.7	27.73
	i_f	100	>139	>157	>100.29

Table 4. Influence of different compactive efforts on gravel soil property

characteristics \ Project	Jinping I		Shiziping		Shuiniujia	
Compactive effort (kJ/m^3)	604	2740	604	2740	604	2740
ρ_d (g/cm^3)	2.04	2.12	2.17	2.24	1.86	2.02
C(MPa)	0.035	0.04	0.04	0.08	0.05	0.05
Φ(°)	25.4	30.0	28.0	33.0	20.6	23.6
k_{20}	1.5×10^{-6}	3.3×10^{-7}	1.5×10^{-6}	2.8×10^{-7}		2.8×10^{-7}
i_k			3.00	4.75		7.56
i_f	12.4	>13.35	7.00	12.83		13.42

($Ec=604kJ/m^3$) sometimes leads to compaction factor more than 100% during construction. Therefore, it is necessary to adopt modified Proctor compactive effort ($Ec=2740kJ/m^3$) to analyze earth property and determine its compaction criterion in design of high gravel soil core rockfill dam.

3. Conclusions

Practical experiences indicate that earth core rockfill dam has been become one of the main dam types in high dam construction in the world. Gravel soil used as impermeable material in high earth core rockfill dam has been the trend today. This is well confirmed by high gravel soil core rockfill dams designed in recent years in Sichuan province, China.

Experimental studies on some projects indicate that removing coarse grains by screening is an effective way to improve gradation and performance of gravel soil when fine grain content is low in natural grading gravel soil or its imperviousness and impermeability can not meet requirements ; By reasonable design of filter material, percolation gradient of gravel soil with filter material protection increases greatly to protect core material from occurring seepage failure; It is necessary to adopt increased compactive efforts to analyze earth property and determine its compaction criterion in design of high gravel soil core rockfill dam.

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