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Historical Development and Some Experiences of Energy Dissipator at Multiple-Purpose Projects in Japan

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Synopsis

The Ministry of Construction, Japanese Government, has completed more than fifty multiple-purpose reservoir projects after the World War II. A historical view on the development of water-release equipments and their energy dissipators and the results of investigations on their performances after completion are described herein.

The contents of this paper are as follows; The first, a design development of the water-release equipments to fit the hydrological and geographical requirements peculiar to the rivers in this country is described. The second, conception on the release of the excess water is clarified. The third, general design procedures used at the energy dissipators of various types and a few design examples through the model test are presented. The fourth, an investigation of the experiences on outlet and stilling basin works conducted in 1963 is described.

General Remarks

The watershed development projects in Japan, initiated during the latter decade of 1930, have been successively progressed in the effective control of flood and the utilization of water resources, despite of their discontinuity in the World War II. Almost all the multi-purpose reservoirs completed have displayed a satisfactory performance in their operation and are making a real contribution to rapid economical growth in this country. The projects which were completed up to April 1964 under the supervision of the Ministry of Construction amount to 59, most of which were finished after 1950. In all the projects except two, concrete dams were constructed, because they are superior to the fill type ones in securing a higher safety for excess water during flood. In this country, frequent occurrences of severe earthquakes and floods had restricted the structural types of concrete high dam to the gravity one. But, since the former decade of 1950 the economical demand for construction cost has brought a remarkable progress both in design and construction technique of arch dams, and in consequence over half of the dams built after 1960 are of arch type.

The water-release equipments and energy dissipators at dams have common features specified by topographical and hydrological requirements in this country, which make a considerable effect on flood-control or water-utilization projects by reservoirs. The characteristic phases of the rivers in Japan, in relation to the multi-purpose reservoir projects, are in short described as follows; The first, since most rivers have their rise in mountaineous areas over 1,000 m above sea level and very small in length, the flood hydrgraph varies so abruptly with time that it is possible to control flood quite effectively even by the reservoir with a comparatively little storage capacity, while a quick and exact flood forecasting and a sufficient provision on reservoir capacity for flood control are needed. The second, since the average precipitations, most of which are due to the rainy front in June to July and typhoons in September to October, are over 1,600 mm a year, and maximum discharge during flood is so large that a great discharging capacity of the spillway is usually required in comparison with the small scale in catchment area. The third, since the river channel in mountaineous areas is usually formed by deeply eroded valleys and abrupt bends due to their green geologies, these topographical features restrict the arrangement of outlets and spillways to use the water-release devices operated under high head.

Development of Outlets and Spillways

To satisfy the requirements as mentioned in general remarks, it has become a recent trend in the outlet design after the latter decade of 1950 to provide a few high-pressure conduits of large scale, capable of partial opening in operation for flood control. In almost all the outlets, tainter or roller gates have been installed at the downstream end of conduits to regulate the flow to any degree of fineness, without a risk of cavitation. This type of gate with 5×4 m in size has shown a satisfactory performance under the head of 60 m by the the use of sealing of an eccentric trunnion type. Such types of high-pressure outlets for flood-control have been used at three-fifth of multiple-purpose dams completed after 1960, as shown in Table 1.

Almost all of the existing spillways are of crest-overflow type attached to the dam itself. As the maximum size of the crest gate was 10 m in height before the War, some spillways with large design flood discharge could not

TABLE 1.
High Pressure Outlet Conduits at Multi-purpose Dams in Japan.

Dam	Type	Date of Completion	Type of Gate (m)	Size of Gate (m^2) W × H	Max. Operation Head (m)	Sealing
Ayakita	C.D.A.	1959	Tainter	5.6×5.0	34.1	—
Futase	C.T.A.	1961	Tainter	5.0×3.4	69.0	E.T.
Ono	C.G.	1961	Tainter	3.6×4.3	34.7	E.T.
Muromaki	C.D.A.	1960	Roller	2.6×2.6	41.3	H.A.
Yuda	C.T.A.	1964	Tainter	4.6×3.2	54.4	E.T.
Amagase	C.D.A.	1964	Roller	3.5×4.6	37.8	E.T.
Kawamata	C.D.A.	1965	Roller	3.2×3.0	69.0	H.A.
Tsuruta	C.G.	1964	Tainter	4.3×4.2	45.4	E.T.
Sonohara	C.G.	1964	Tainter	5.0×4.0	54.5	E.T.
Kasabori	C.G.	1964	Tainter	2.8×2.3	44.0	—
Hamada	C.G.	1963	Tainter	2.8×1.9	42.2	—

Note; C.G. : concrete gravity, C.D.A. : concrete dome arch, C.T.A. : concrete thick arch, E.T. : sealing of eccentric trunnion type, H.A. : sealing of hydraulically actuated type.

help being provided by larger width than that of the river and the overflow section was contracted toward the apron, which made the occurrence of uneven flow entering into the stilling basin. Efficiency of energy dissipation due to the hydraulic jump was decreased and frequently the basin floors were eroded by circulating debris drawn in the basin. Since about 1950, the technical design of spillways was refined and available depth for flood-control was also increased by providing the higher crest gate with a size of 17 m in height.

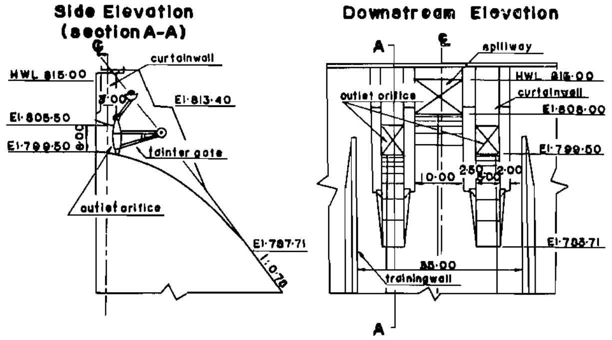

Fig. 1. Miwa Dam outlet and spillway works.

If the available depth for flood-control is under at most 25 m, more economical outlets with large rectangular cross-section, so-called an orifice as shown in Fig. 1, are used. This type of outlet consists of entrance opening formed in the bellmouth shape and hydraulic loads exerted on the curtain-wall above the opening are supported by piers on both sides. It makes the utilization of the storage capacity more effective and the cost of crest gates more economical. Since the flow through the orifice enters unevenly into the basin, it is necessary for the formation of effective jump to spread the flow uniformly by providing a sloping apron or separate wall at the upstream end of basin. These stilling devices were investigated through a scale model test of the Yokoyama project[1].

In earlier works of arch dams, a free overfall spillway with large overflow depth had not been used, because it had been afraid that deep break of arch action at the overflow section would give unfavourable influence on the dam body and the deep erosion at the base by falling water would threaten the safety of the dam. Therefore, a provision of another spillway for emergency was required with results of expensive construction of discharge equipments. But a number of model and field observations verify that a large overflow can also secure the safety of the dam itself by making reinforcement and providing a water pool. At present, the overfall spillway with 10 m depth can be used economically. The outlet conduits through the body made also the same problem in stress pattern of the body, and the structural model test certified that the large conduit could be safely installed[2].

Development of Conception on Release of Excess Water

Progress of design procedures on hydraulic works at multi-purpose dams has brought a wide variety of conceptions on released-water-disposition ; In earlier stage of the development in projects, in which the relase equipment of crest overflow type performed both functions of outlet and spillway, a hydraulic jump-type stilling basin was designed to make ar effective dissipation of energy in both service and emergency operations. In such a case, the cost of construction was inevitably expensive because heights of the auxiliary dam and the side wall were designed under the condition of emergency operation. Thereafter as the outlet and spillway have been equipped separately, a definite conception on the release of water has been gradually disclosed; The regulated outflow should be changed subcritical at the downstream end of the basin, to secure the channel maintenance of the downstream river. On the other hand, for emergency release, high security in the structural safety of dams must be proceeded prior to the maintenance of the downstream channel. The concept described herein is the basis to develop a better and more economical design of energy dissipators.

Development of Energy Dissipators

(1) General Description

Most of the multi-purpose dams in Japan are constructed in th mountaineous area due to the geographical and social requirements. Therefore, the stream in the vicinity of the dam site is commonly rapid, and the tailwater depth for any discharge is less than the sequent depth. Except a few, the stilling basin of hydraulic jump-type supplemented by a secondary dam is a common type of basin.

In almost all of earlier projects, the hydraulic jump-type stilling basin consisted of a simple horizontal concrete floor. As the hydraulic jump for smaller discharge than the design discharge would be drowned, a continuous end sill with sloping upstream face of 1 or 2 m in height was used to deflect a high velocity underflow. The relative length of the existing hydraulic jump-type stilling basins is shown in Fig. 2 with the Froude number of the incoming jet for the maximum controlled outflow. Satisfactory performance of the simple horizontal apron of l/h_j over 4.5 in both service and emergency operations has been verified by hydraulic model tests. For the aprons of l/h_j less than 4.5 used in the earlier project, the length was mainly determined by the expenditure of construction cost rather than by the hydraulic performance, without making a verification through the model tests. All of the basin with the appurtenances such as baffle block, abrupt rise and trapezoidal weir, have become effective after 1960.

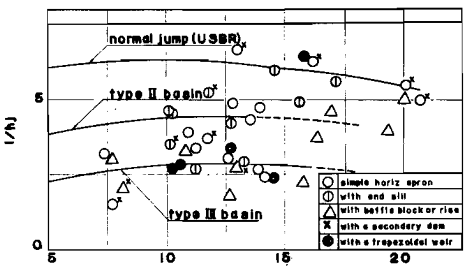

Fig. 2. Relative length of existing jump-type stilling basins.

For the center overflow spillways of thin arch dams, a water cushion-type of energy dissipator is commonly used. The main cause of the bed erosion by the falling nappe may be considered to be that the nappe of high energy penetrated through the crack of the foundation rock exerts an uplift force on a lump of rock and lifts it up. Since the joints of foundation rock develop in complicated phase, to maintain the uniformity of the bed, the concrete protection is made in the reach, where a large dynamic pressure is indicated on the experimental model investigation. The necessary water depth in the pool is usually determined on the basis of the pressure measurement data obtained by the hydraulic model test. As the possibility of damage or erosion of the concrete floor depends on the magnitude of the variation and distribution of bed pressure, the water depth is chosen in such a way that the maximum dynamic pressure p_a, the deviation of the maximum bed

TABLE 2.
Water-Release Equipment and Stilling Device at Multi-purpose Thin Arch Dams in Japan.

Dimension of spillway at existing arch dams

Dam	Spillway				Outlet		Max. heigh of tail water (m)	Stilling pool			
								End sill		Secondary-dam	
	Type	Discharge capacity (m^3/s.)	Crest length (m)	Max. over flow depth	Quantities	Dimensions of the cross section (m) W H		location D.C. + m	height (m)	location D.C. + m	height (m)
Ayakita	Center over flow	500	27.6	4.6	2	5.60×4.5	68.1	45.6	2.0	109.0	13.0
Muromaki		773	24.0	6.3	2	2 62×2.62	79.0	46.5	2.0	119.5	9.7
Okura		1400	32.4	7.3	—	—	75.7	45.0	4.0	122.0	13.0
Kawamata		1240	36.0	6.5	2	3.22×2.85	117.0	60.0	2.0	145.0	13.1
Amagase		640	40.0	4.0	3	3.42×4.56	61.5	58.0	2.0	93.6	7.0

Dam	Continued				Data for determination of pool depth						
	Apron			Height of wall (m)	Scale of model	Discharge per unit width (m^3/s/m)	Design water depth in pool h(m)	Equivalent sequent depth h_j(m)	Max. dynamic pressure p_a(m) (m in water col.)	Distance showing p_a s(m)	Coeff. of pressure concentration p_a/s
	Mean width (m)	Thickness (m)	Length (m)								
Ayakita	30.0	1.5	45.6	17.0	1 : 40	21.3	14.0	11.6	26.0	17.5	1.21
Muromaki	9.3	1.5–3.5	46.5	26.5	1 : 40	32.2	18.0	14.8	24.8	26.0	1.22
Okura	9.5	4.0	65.0	36.0	1 : 80	43.2	19.5	16.9	29.0	33.6	1.15
Kawamata	25.0	1.0–1.5	90.0	28.0	1 : 50	33.9	20.0	16.9	45.0	25.0	1.18
Amagase	42.0	1.0–3.0	93.6	20.0	1 : 50	30.5	13.0	13.6	32.5	25.0	0.96

pressure from the pressure corresponding to the tailwater depth, becomes less than 30 m in head in the prototype, or that the coefficient of pressure distribution, the ratio of p_a to the distance s in the flow direction which p_a is recorded, becomes less than unity. However, they are values of the provisional design criterion, which are not based on the dynamic analysis of behavior of the falling nappe. In order to develop a more reasonable design of the stilling pool of this type, further investigations on air-entrainment mechanism of the falling nappe and dynamic behavior of the concrete floor during operation must be conducted on both the prototype and the scale models. It is usually taken into consideration to make the discharge per unit width as small as possible and to fix the concrete mat to the foundation by anchor bars. The dimensions of spillway and stilling basin works of the existing thin arch dams determined by the hydraulic model test are shown in Table 2.

The released flow through the outlet which has the regulating gate at the downstream end is not mixed with the flow from the overflow spillway. As usual, the gravity and thick arch dams provide the outlet relase along the downstream face of the dam and dissipate the energy of the flow by the use of hydraulic jump. The sloping apron or large bucket curve is provided at the upstream end of basin. The separate wall can be also provided not to occur the surface disturbance due to the interference of the flows through the adjacent outlets.

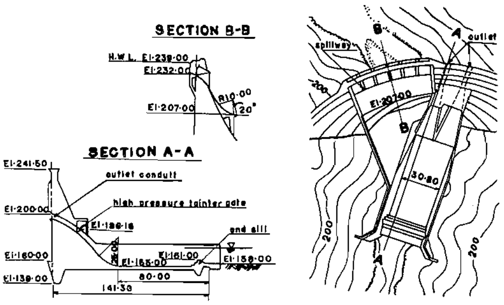

Fig. 3. Spillway and stilling basin works for Yuda Dam.

A recent trend for the discharge flow through the spillway is to use an energy dissipator of trajectory type. The alignment of this kind of dissipator used at the Yuda Dam is shown in Fig. 3[3].

As the discharged flow through a high pressure outlet equipped at the thin arch dam falls apart from the toe of dam, one stilling device is used for both outlet and spillway operations. It is necessary for the falling jet not to impinge the valley and the secondary dam. The deflector to spread the jet is often provided at the exit of the conduit, or the outlets are designed in a manner that the released jets through two outlets intermix each other in the air. A few types of the energy dissipators designed recently through the model tests are described in the following.

(2) Laterally Curved Basin

When a dam is located at an abrupt bend of a valley, a hydraulic jump-type stilling basin with laterally curved walls will be designed. The use of this type of the basin should be avoided if possible, because the supercritical flow makes irregular standing waves across the channel and further the subcritical flow deviates to the outer side and consequently the back current will be produced along the inner wall with results of less efficiency in energy dissipation. One solution made under this condition is to use the roller bucket with a very small basin in length, which was first equipped at the Ikari Dam.

For the Sarutani Dam which was inevitably provided by the curved basin from the beginning of the apron, the multiple guide-baffle, as shown in Fig. 4, is provided to prevent the flow concentrating in outer side and to secure an uniform toe of the jump by dividing the incoming flow into four parts[4]. In September, 1959, the spillway experienced the released flow of approximately maximum design discharge, 1,935 m^3/sec., and the hydraulic performance of the baffle was verified in close conformity with the results of a model test except the occurrence of extensive erosion at the downstream portion of the apron.

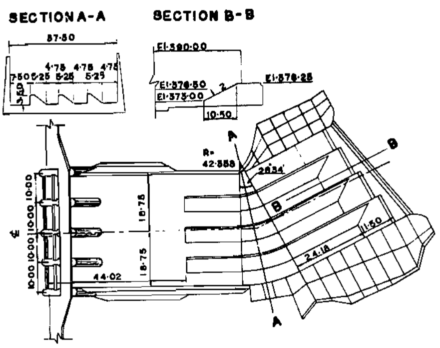

Fig. 4. Sarutani Dam laterally curved apron.

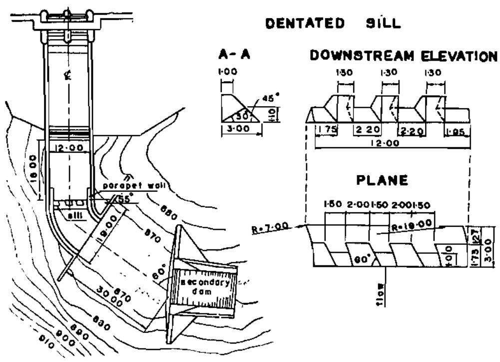

Fig. 5. Shinaki Dam spillway works and dentated sill.

Fig. 5 shows the curved basin of the 44 m high Shinaki Gravity Dam with overflow type spillways designed for the maximum discharge of 310 m^3/sec.. Though the normal jump length for the design discharge is 51.4 m, the river channel bends 20 m downstream from the toe of dam, resulting in the use of laterally curved apron. To shorten the length of the basin, it was tried to provide the dentated sill with 2 m height as shown in Fig. 5, at the beginning of the curved portion. It has also expected for the dentated sill to disperse the undercurrent, prevent the bed erosion by the violent mixing and direct the deflected flow in the direction of the center of downstream channel. The surface profiles and velocity distributions in the basin along the outer and inner walls when the sill exists and does not exist are shown in Fig. 6.

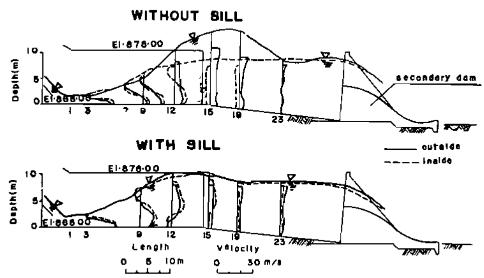

Fig. 6. Surface profile and velocity distribution in basin.

It is found in the figures that the sill has a considerable effect on the elimination of concentration of the flow to the outer side. The velocity distribution shows that the efficiency of dissipation in energy by the bed roller behind the sill is large. To prevent the occurrence of the subatmospheric pressure on the face of the sill, the corner edge was rounded by an elliptical curve and verified by the pressure measurement on the scale model to be cavitation free.

(3) *Trapezoidal Weir*

When the tailwater depth is much less than the subcritical sequent depth, a provision of a secondary dam or a trajectory bucket is usually considered. If the channel structures or bed would be feared to be subject to severe damage or erosion by the falling nappe through the bucket, and if the suitable site to provide a secondary dam could not be found, these devices can not be used. As the discharge capacity of outlets is usually much less than that of the spillways, it becomes an important problem to develop the economical and safe design of the stilling device, when one stilling basin is used to dissipate the excess energy in both service and emergency operations.

Thus, the energy dissipator with a trapezoidal weir of a relatively large height at the end of horizontal floor basin has been developed and used at five multi-purpose projects; Miwa, Meya, Yokoyama, Tsuruta and Ozegawa projects. The weir has a performance to secure the sufficient tailwater to form a jump in service operation and to prevent the scour just downstream of it by splashing the flow under supercritical state in emergency operation. Such a design can reduce the basin length and the wall height considerably, compared with the common secondary dam with a vertical upstream face. The Tsuruta Dam has three outlet conduits of partly pressure type with the maximum discharge capacity of 1,400 m^3/sec., located in the portion of the overflow spillway of which the design discharge is 5,000 m^3/sec.. As a suitable site to the secondary dam was not found and the river channel turned to the left about 150 m downstream from the toe of apron, the stilling basin with a trapezoidal weir was used. The length of the basin was selected equal to 70 m for securing the economical requirement and the suitable direction of splashed flow, while the length of the normal jump computed for the maximum controlled release, 2,300 m^3/sec., becomes to be 105 m.

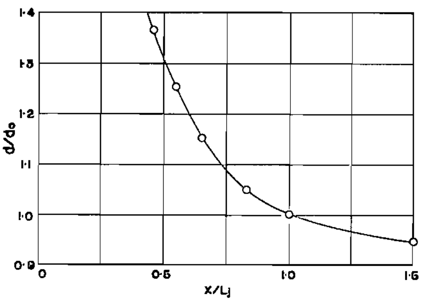

Fig. 7. Relation curve between height of weir and position of jump.

The stilling basin was designed to make a hydraulic jump for a discharge of 2,500 m^3/sec., considering the allowance for non-uniformity of the incoming flow. The relative height of the free overflow weir necessary to form a jump with the relative jump position obtained by the experiment, is shown in Fig. 7. In Fig. 7, X represents the distance from the toe of jump to the weir, L_j the length of a normal jump, d the necessary height of the weir, and d_0 the height of the weir computed, based on the assumption that the flow is critical on the crest. In this case, X/L_j is 0.67 and d_0 10.0 m, then d becomes 12.0 m. Through the

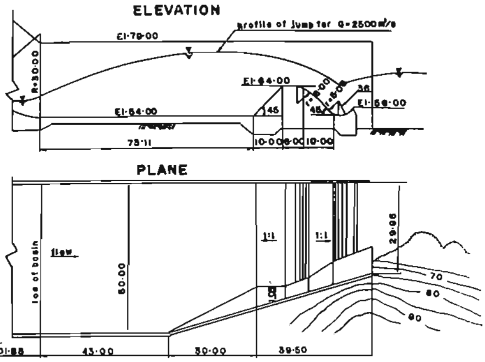

Fig. 8. Tsuruta Dam stilling basin works.

test, it has been indicated that the contraction of the apron toward downstream had an effect on shooting the flow along the curved river channel in emergency operation and on reducing the weir height to 10 m due to increase of efficiency of back-water[5]. The final design of the stilling basin and the spraying flow condition in emergency operation observed in model test are shown in Figs. 8 and 9, respectively.


Fig. 9. Deflected flow condition in emergency operation (1 : 50 scale model).

(4) Water Cushion-type Energy Dissipator

The Amagase Dam was designed as a concrete arch dam of dome type with 72 m height. Three outlet conduits are provided to discharge a total of 1,000 m^3/sec. under 33.5 m head and to control the flow by high-pressure roller gates placed at the downstream end of conduits. The overflow spillways with the net crest length of 40 m in total, capable of discharging 680 m^3/sec., were also made. The water pool was provided by a secondary dam to dissipate the excess energy of falling nappes through both outlets and spillways.

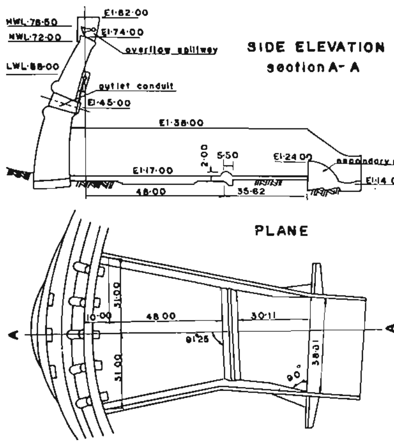

Fig. 10. Amagase Dam spillway works and stilling pool.

The depth of the pool of 13 m was determined by the bed pressure measurement obtained on a scale model when outlets were operated in full opening at the maximum water level in the reservoir[6]. It was verified by the test that the maximum bed pressure at both operations of outlets and spillways was smaller than that at single operation of outlets under the same operating conditin.

The secondary dam of 7 m high was provided at 93.6 m downstream from the axis to secure the required depth and the deflector with 2 m height was set at the middle portion of the floor across the basin to disperse a high velocity undercurrent and to eliminate the difference in water level between both sides. The horizontal concrete mat was constructed on the foundation rock in the whole reach of pool as shown in Fig. 10.

Operation Experiences of Outlet and Stilling Basin Works

(1) General Description

The oldest multiple-purpose project in Japan is the Kodo Dam in Yamaguchi Prefecture, which was completed in 1935. The dams with the flood-control purpose which have been completed before April, 1963, amount to fifty-nine, five of which were built before 1950, 17 early in 1950's, 23 in the latter decade of 1950, and the others after 1960. Since multi-purpose reservoir projects bave been in operation less than 10 years, there are few spillways which have experienced large flood releases, while water releases from the outlet works have been frequent.

Though the actual performances of the water-releasing and stilling devices have hardly investigated except when the severe damages of the basin floor or river channel were caused by the operation, a provision of the outlet structures operated under high head requires closer inspection and maintenance of the hydraulic structures. Then, the first investigation on the water-releasing devices at the multi-purpose dams in Japan, was conducted by the Public Works Research Institute, the Ministry of Construction, in 1963. The main purpose of the investigation was to obtain the data available to make the practical evidence in the design as well as the future maintenance of the works constructed and to verify the model indication with the observed results of the prototype. At first, to facilitate future investigations, the data of 49 dams were collected in a tabular form on the subjects shown in the following :

1) Types and characteristics of dam ; height, crest length, volume of concrete, and commenced and completed year.

2) Hydrological data ; design flood discharge for spillway and outlet, specification of flood control, and water level and storage capacity for reservoir utilization.

3) Outlet details ; type, use, dimensions, designed capacity and details of regulating devices.

4) Spillway details ; type, dimensions, designed capacity and details of gate.

5) Appurtenant outlets ; sand removal gate and emergency outlet.

6) Stilling basin detail ; type, dimensions, maximum height of fall, and details of appurtenances and secondary dam.

7) Characteristics of outlet channel ; lateral bend, presence of landslide or talus, depth of bed deposit and quality of foundation rock.

8) Condition of construction ; method of diversion, period of apron placing, frequency of flooding during construction and presence of drawn debris in basin.

9) Hydraulic model test; model scale, executor, investigated items, and discrepancy between recommended design and existing one.

10) Actual records of operation; date of flood occurrence, maximum inflow and outflow, period of operation, maximum reservoir level during flood, gates used for releasing and type of flood.

11) Scheduled field test; purpose, object of test, date of test and summary of test result.

12) Occurrence of unexpected flow behavior during operation; date, place and cause of occurrence, difference from expected performance and damaged state.

13) Field inspection; date, portion and items of inspection, and its result.

14) Actual result of repair; date, portion and method of repair, and its efficiency.

15) What kinds of data are necessary to operate the reservoir more effectively?

A summary of the results investigted is given in Table 3. Field inspections have indicated that the stilling basins or exit channels for 12 dams were damaged during flood releases, and that the damage of the apron is mainly attributed to the erosion by recirculated debris which are drawn by lateral vortices, while the erosion of the river channel is caused by incomplete stilling action at a large release. It is obviously recognized that the scale of damages increases with an increase in frequency of operation and discharged capacity, by the fact that almost all the structures which experienced the discharge equal to or more than the maximum design one have been noticed to be different from that predicted by the model studies.

Several experiences of high head gates indicate that a severe vibration or water-leakage occurs for a certain discharge due to improper design of guide plates or sealing. Prototype data of a few structures at which the damage or the unsatisfactory performance was found will be discussed later.

(1) Ayakita Dam

The outlet works for the Ayakita Arch Dam consist of two rectangular conduits, capable of discharging 1,060 m^3/sec., under 29.1 m head. The releas-

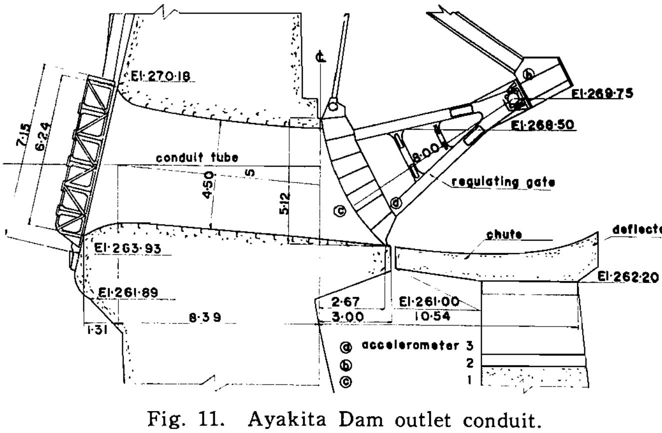

Fig. 11. Ayakita Dam outlet conduit.

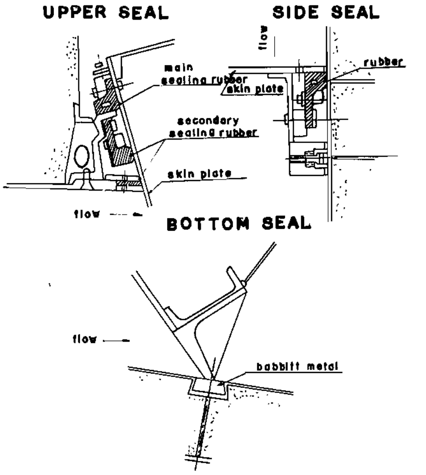

Fig. 12. Sealing mechanism of outlet tainter gate.

ed water is regulated by 5.60-m-wide by 4.98-m-high tainter gates, equipped at the downstream end as shown in Fig. 11. After completion of the dam, prototype gate tests were made under 25.5 m operating head in November, 1960, to obtain data of vibration and bending stress during operation. The vibration characteristics were measured by using six accerelometers and the stresses of lower arm by four strain gauges. The measurement was done for the left side gate at the opening of 0.5, 1.0, 1.5 and 2.0 m, while another gate was held at 1.0 m opening during operation. The sealing mechanism shown in Fig. 12 indicated a satisfactory performance of water-stop at any opening during tests. But, during floods of 1961, the upper secondary seal sustained severe wear, resulting in considerable water-leakage, and then was exchanged by brass-lined rubber to prevent abrasion. A part of repaired rubber has been removed by released water during operation in the next year. The water-drawing through the bottom at closed position has necessitated to hold water-tightness by the emergency coaster gate, with the respect of maintenance of gate and concrete chute.

An undesirable phenomenon on the guide-wall of the chute during tests was observed to occur a periodic rising of issuing jet, at 1.0- to 1.5-m-opening, which pounded the anchor beam with results of intense impact force. This was also verified by vibration data recorded maximum vertical amplitude of 0.3 mm for anchorage and the maximum longitudinal amplitude of 0.1 mm for gate, at 1.0 m opening. The cause of occurrence of oscillating nappe was concluded that the high velocity flow discharged under the gate spreaded laterally in the opening which was formed at the bottom corners of joining section between conduit and chute, and impinged on the guide wall. The bottom corners of the conduit is rounded in a circular form with 1.3 m radius so as to prevent the stress concentration, while the concrete chute has the rectangular section. As the subsequent operations experienced a severe vibration of gates, the vibration-damper was equipped at the side parts of the gates in 1962. During the model tests, these unfavorable phenomena had not been recognized because of a small scale in the model and because of the fact that the dispersing efficiency of the deflector had been mainly investigated at full gate opening. Thereafter, the design of rectangular outlet conduits with regulating tainter gate has been refined to diminish the rounded corner toward the outlet portal.

(2) Tase Dam

The dam is a straight, concrete gravity structure 81.5 m high, completed in October, 1954. The spillway is composed of a gravity ogee weir controlled by 6 tainter gates, 10-m wide by 8.5-m high. When the reservoir level is is below the spillway crest, flood regulation is accomplished by 4 outlet conduits in the middle of the spillway, capable of discharging 500 m^3/sec. at 42.5 m maximum operating head. The stilling basin consists of a level apron 70-m wide and 59-m long, provided by 2-m high stepped side sills with end baffle blocks and a 1-m high sloping end sill in the middle part.

The basin was dewatered and inspected in November, 1959. Desposition of about 583 m^3 sand and gravel and erosion with a depth of 10 cm to 100 cm in a ring form were found in the middle part of the basin floor, as shown in Fig. 13. 16 large floods whose maximum released discharges were about 110 m^3/sec. on the average had occurred up to 1959 after completion. Operation of one outlet conduit had been made for regulating almost all floods, since the pressure type conduit with the gate provided in the body should not be operated at half opening to prevent occurrence of cavitation. It was found during operation that the released flow through the conduit indicated the submerged condition in the basin with little stilling action and splashed over the end baffle with high standing wave, with a result of erosion to side banks of the outlet channel. Asymmetric release to the center line of the basin caused the lateral eddy current stretched to the downstream channel, resulting in an abrasion of the concrete floor by dragged debris, shown in Fig. 14. Moreover, the cofferdam left just downstream of the apron checked the development of the bed

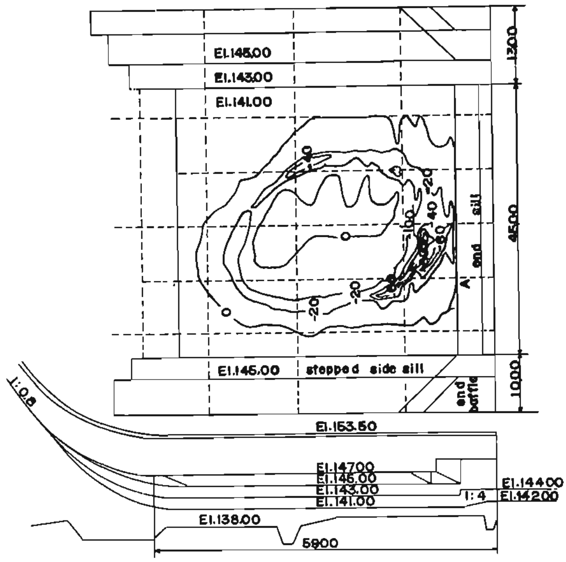

Fig. 13. Erosion pattern in apron for Tase Dam.

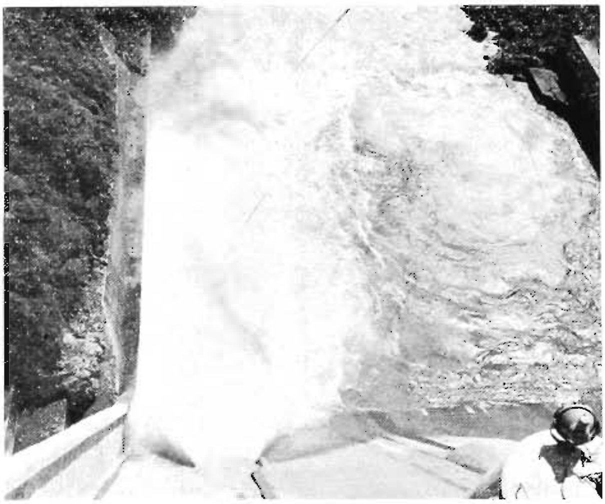

Fig. 14. Laterally back current at Tase Dam during one outlot operation.

roller and promoted occurrences of bed scour and lateral eddy.

The design of stilling basin was made after the model test conducted in 1953[7]. It was very regrettable that the stilling action for only two kinds of discharges, 500 m^3/sec. and 1,000 m^3/sec., was investigated in the model test, while the performance of stilling basin was not certified for a relatively small release as frequently appeared in the prototype.

The second inspection made in August 1960 did not find further progression of erosion than that in the previous inspection, probably for lack of experience of water-release in this period. At the same time, the eroded part of the basin floor was repaired by the gannet method and the sequent outlet operation during 4 floods of 1962 indicated that the repaired basin was not damaged, though a little deposit was found in the basin.

(3) Nagayasuguchi Dam

The spillway for this multi-purpose project consists of a concrete gravity ogee weir topped with six 10.0-m wide by 14.5-m high roller gates, used for both service and emergency operations. The height of the crest above the apron floor is 64.5 m and the 77.5-m long spillway section on the crest is contracted toward the toe to be 54.0-m long at the stilling basin section. The basin is composed of unusually short level floor with 2.5-m high curved end sill to deflect the flow far downstream at large releases. The spillway works were designed in conjunction with a series of model tests, containing the movable bed test, of which the model scale was 1 : 50. The model test indicated that the front of hydraulic jump reached downstream from the apron and the deflected flow fell on the base of the protection wall of the left side bank, with deep erosion, for the discharge over 1,500 m^3/sec., while the design discharge was 5,400 m^3/sec.. So the 4-m heightening of the cofferdam located at 130 m downstream from the apron was recommended by the Public Works Research Institute, to secure the formation of a jump, though this work was not realized for deficit of the cost of construction.

Fig. 15. Erosion at Nagayasuguchi Dam.

The first operation of the spillway was made during the flood in September, 1956, after completion of the project in 1955. Six floods of which the maximum released discharges ranged from 1,000 m^3/sec. and the average period of operation was 300 hours passed over the spillway during the year of 1956 to 1957. An extensive damage was noticed at the central part of the apron and on the top of the end sill, when the basin was dewatered for inspection in January, 1958, as shown in Fig. 15. The maximum depth of erosion on the concrete floor was about 2 m and the face of guide wall on the left side was roughened by abrasion. The severe erosion on the top of the end sill would result from the cavitation, where the reinforcing steel was bent toward upstream by bed rollers.

The subsequent investigation made in 1960 after 10 flood releases with the maximum rates of 300 m^3/sec. to 2,000 m^3/sec. did not find the further erosion of the basin while the scour of the foundation just upstream of the cofferdam was considerably progressed. During the flood in September 1960, 4 spillway gates except 2 on both sides were operated to release water of 4,500 m^3/sec., with results of failure of the part of the coefferdam and the protection wall of the downstream channel.

Causes of erosion and failure were thought to be the following: In design, though the front of jump is secured in the basin for discharges less than 1,500 m^3/sec., the length of the apron is so short the bed erosion between the apron and the cofferdam is severe, and that the lateral eddy current which occurs due to the uneven distribution of the incoming flow and curvature of the exit channel draws the scoured debris into the basin, with results of severe abrasion of the concrete floor. For discharges over than 1,500 m^3/sec., the tailwater depth held by the cofferdam is so insufficient to form a hydraulic jump that the flow deflected by the end sill causes a deep erosion of the channel bank. As to construction, the erosion patterns of the apron which are different between three blocks may be verified by the fact that the placing period, the quality and strength of concrete were varied each other. Cavitation damages on the sill may be accelerated by a rough concrete surface placed at the curved section. As to actual operation, when all spillway gates are operated at uniform opening, the flow concentrating to the central part of the basin will occur and result in reduction of the stilling action. For the topographical condition, the abrupt bend of the exit channel will cause a lateral back roller at the upstream portion of the cofferdam.

In 1962, the eroded parts of the apron and protection-wall were repaired by replacing the concrete, and the damaged river bank was protected by derrick stones, which have indicated a satisfactory performance against flood releases thereafter.

(4) Miwa Dam

The 69-m high Miwa Gravity Dam has an overflow spillway with 10-m wide by 7-m high tainter gate and two outlet orifices with 5-m wide by 7-m high tainter gates, capable of discharging 1,400 m^3/sec in total. The stilling basin consists of a level apron 35-m wide and 69-m long, provided by a trapezoidal weir of 6.5-m in height at the end of the basin.

The basin was designed with the aid of model tests to form a hydraulic jump up to the discharge of 500 m^3/sec. which was released through two outlet orifices operated at equal gate opening. It was verified by the model tests that orifice operation gave the severest requirement for stilling action, whereas the actual regulation was mainly conducted by crest gate operation and the design controlled release was 300 m^3/sec..

Over a period of 4 years until 1961, 4 releases with the maximum discharges ranging from 100 m^3/sec. to 400 m^3/sec. were experienced. Observed performance of the prototype stilling basin during these floods indicated a good agreement with that of model tests.

During the flood in July 1961, a flow of 742 m^3/sec. was released from the outlet and spillway, resulting in a splashed flow condition over the weir. The deflected flow spread in the downstream channel, and the side bank of the river fell for 100-m reach by turbulent back rollers. When the released water reached to 350 m^3/sec., the toe of jump moved abruptly toward downstream and began to sweep out of the weir, as shown in Fig. 16, whereas the model tests had shown that a normal jump should be held at this discharge rate. The cause in which this unpredicted flow condition appeared is believed to be the occurrence of lateral eddy current in the basin due to a slightly difference of discharge rate between two orifices.

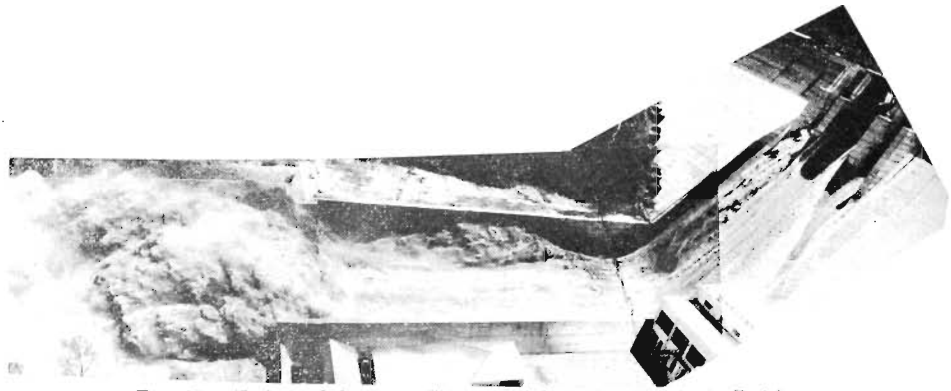

Fig. 16. Deflected flow condition at Miwa Dam with 350 m^3/sec..

Conclusion

The hydrological, topographical and social features in Japan have brought about the development of several peculiar types of energy dissipation works for multiple-purpose dams in this country. The multi-guide-baffle and horizontally skewed dentated sill have been used effectively for laterally curved apron. To be suitable to flood-control requirements in this country, the stilling basin of trapezoidal weir type has been developed and verified to perform a sufficient stilling action and to satisfy the economical requirement. The characteristics of air entrained in the falling nappe and the mechanism of scour occurred at the base should be further investigated to develop more economical design of water cushion-type energy dissipators.

The operating experience with spillway and outlet works at these dams gives summaries as followings ;

(1) The high pressure tainter or roller gate operated under over 30-m head should provide a particular sealing apparatus such as an eccentric trunnion, to prevent leakage and vibration during operation.

(2) Design and operation of spillway or outlet should be conducted to make the released flow symmetric to the center line of the basin, and not to cause the laterally back roller which might draw the eroded debris into the basin.

(3) Choice of the apron length and the height of secondary dam insufficient to a complete stilling action will induce severe erosions in the whole reach of the basin without exception.

(4) To certify the satisfactory performance of the stilling basin which is commonly used to spillway and outlet works, the model test should be made in such a manner that a perfect stilling action can be obtained for a large variety of combinations of outlet and spillway discharges to be expected.

Acknowledgment

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Table 3. Prototype Data of Existing Spillway and Outlet Works.

No.	Dam	Date of completion	Dam		Flood-control		Outlet works				Spillway works			Energy dissipator			Scale of model test 1 : n	Frequency of flood release			Field test			Field inspection			
			Type	Height (m)	Design flood (m^3/sec.)	Max. controled release Q_0(m^3/sec.)	Type	Number	Max. operation head (m)	Design discharge (m^3/sec.)	Type	Number of bays	Design discharge (m^3/sec.)	Type		Appurtenance		Discharge over Q_0	Discharge over $1/2\,Q_0$	Discharge over $1/4\,Q_0$	Year	Object	Items	Year	Object	Items	With damage *
1	Ishibuchi	1954	R	53.0	1,200	900					G	6	1,440	Tr	Sj	—	—										
2	Tase	1954	C S	81.5	2,700	500	C	4	42.6	484	G	6	2,715	J	H	Es	30			3				1959	Stilling basin	Basin deposit	
																								1960	Stilling basin	Floor erosion	*
3	Ikari	1956	C G	112.0	2,000	1,000	C	1	41.0	100	G	3	3,000	J	R6	Sd	40		2	2	1960	Stilling basin	Floor erosion				
4	Narugo	1957	C A	94.5	1,600	900	T	1	19.0	750	U G	8	1,250	J, Tr	H	Tw	40, 60										
5	Fujiwara	1958	C G	94.5	1,950	1,100					G	3	3,600	J	Hc	Sd	—		1	1				1960	Crest gate	Inspection	*
6	Aimata	1956	C G	67.0	650	330					G	2	780	J	H	Sd	—										
7	Futase	1961	C A	95.0	1,440	800	C	2	67.0	840	G	4	1,800	J	H	B, Sd	40, 35										
														Tr	Sj												
8	Miwa	1959	C G	69.1	1,200	300	O	2	15.5	300	G	1	1,140	J, Tr	H	Tw	35	2	4	3	1058	Outlet gate	Vibration	1961	Outlet channel	Scour	*
9	Maruyama	1956	C G	98.2	6,600	4,800					G	5	8,200	J	H	Sd	—	1	9		1954	Crest gate	Stress pattern	1961	River channel	Scour	*
10	Sarutani	1957	C G	74.0	2,060	—					G	4	2,060	J	Hc	B, Sd	25		3	6				1961	Stilling basin	Scour and erosion	*
11	Hmadaa	1963	C G	59.7	430	130	C	2	33.5	270	G	1	300	J	H	B	40				1963	Outlet gate	Vibration				
																						Stilling basin	Stilling action				
12	Nagase	1956	C G	77.0	3,300	2,300	C	2	44.6	20	G	5	4,770	J	Hc	Sd	30			2							
13	Miyakawa	1957	C G	88.5	2,500	1,500					G	3	3,300	J	Rs	Sd	35	1	5	6							
14	Ichifusa	1906	C G	80.0	1,300	650	C	1	78.6	25	G	2	1,560	J	Hc	Sd	30			6				1960	Crest gate	Inspection	*
15	Muromaki	1961	C A	80.5	595	265	C	2	40.0	365	G	2	957	Wc	—	Es, Sd	40				1961	Outlet gate	Vibration				
16	Hiugami	1960	C G	79.5	940	350	C	2	65.0, 50.0	175	G	2	1,200	J	H	Sd	—		3	1							
17	Yubara	1955	C G	70.0	1,424	600					G	6	1,700	J	H	Es	—										
18	Sasaogawa	1957	C G	76.0	470	140					G	3	605	J	H	—			2	1							
19	Kanogawa	1959	C G	61.4	2,750	1,500					G	4	3,320	J	H	Es	55		1	2				1963	Stilling basin	Scour and erosion	*
																									Outlet channel		
20	Hikihara	1958	C G	66.0	470	100					G	2	564	Tr	—	Es	50		1	2				1959	River bank	Scour	*
21	Shikamori	1963	C G	57.5	505	345	C	1	35.1	206	G	1	910	J	Hc	Sd	50			1	1963	Outlet gate	Flow velocity				
														Tr	—												
22	Kitagawa	1962	C A	82.0	1,800	1,300					G	5	2,100	Wc	—	—	60										
23	Yoroibata	1957	C G	58.5	1,100	500	O	2	17.5	500	G	3	820	J	H	Es	—		2	2							
24	Sugano	1954	C G	42.5	500	265					G	2	990	J	H	Sd	60		1					1962	Stilling basin	Erosion	*
25	Sagami	1947	C G	53.7	4,100	—					G	5	4,100	J	H	—	—		5	1				1956–1962	River channel	Scour	*
26	Kijiyama	1960	C H G	46.0	500	265					G	2	1,034	Tr	F	—	50										
27	Kayase	1963	C G	51.0	310	130	C	2	21.0	200	G	1	168	Tr	F	—	50	1			1963	Outlet gate	Vibration	1963	Outlet gate	Water leakage	*
																						Air vent	Air demand				
28	Arita	1961	C G	27.5	52	52	C	1	17.9	14	U G	2	60	J	H		—		1	1							
29	Hanayama	1958	C G	47.8	1,440	455					G	2	1,640	J	H	Tw	50		3	3							
														Tr													
30	Okura	1961	C A	82.0	1,200	400					G	2	1,400	Wc	—	Es, Sd	80				1963	Water release	Flood propagation				
31	Dogawa	1955	C G	66.0	1,300	550					G	3	1,600	J	H	—	—										
32	Matsuo	1950	C G	68.0	3,096	2,550					G	10	3,700	—	—	—	—										
33	Serigawa	1956	C G	52.2	1,100	550					G	3	1,320	J	H	—	—		1	3							
34	Nanakawa	1956	C G	58.5	1,380	320	C	1	28.2	108	G	2	1,920	J	H	—	—	5	2					1959–1962	River channel	Scour	
35	Takashiba	1961	C G	57.5	2,300	1,750	C	1	19.7	16	G	3	2,800	J	R6	—	50										
36	Arasawa	1955	C G	61.0	1,200	360	C	2	15.5	19	G	2	1,870	J	H	Es	50	1			1955	Crest gate	Operation test				
37	Katsurazawa	1957	C G	63.6	550	70					G	3	660	J	H	Es	35			1	1957	Crest gate	Flood propagation				
38	Ayaminami	1958	C G	64.0	790	560					G	2	1,134	J	H	Es	—			3							
39	Ono	1961	C G	61.0	2,400	1,400	C	3	26.7	900	G	3	1,800	J	H	B, Sd	30			1	1962	Outlet gate	Vibration				
																						Air vent	Air demand				
40	Gomei	1963	C G	27.5	150	90	C	2	24.5	90	G	3	100	Tr	—	B	40				1963	Outlet valve	Vibration				
41	Yanase	1953	C G	55.0	1,700	1,100					G	4	2,106	J	H	—	50	2	4	1				1961	Outlet channel	Scour	*
42	Chuzenji	1959	C G	6.4	94	94					G	2	113	J	H	—	—		1					1962	Stilling basin	Erosion	*
																									Outlet channel	Scour	
43	Ayakita	1960	C A	75.3	1,330	1,090	C	2	29.1	1,060	U G	2	809	Wc	—	Es, Sd	30, 40		1	2	1960	Outlet gate	Vibration				
44	Nikyu	1943	C G	29.5	840	—					G	4	1,162	J	H	—	—										
45	Meya	1960	C G	58.0	500	200	O	2	15.5	860	G	1	440	J	H	Tw	60	4		7							
														Tr													
46	Miomote	1953	G C	87.5	1,500	700					G	4	1,870	J	H	—	50	4	2					1958	Stilling basin	Floor erosion	*
47	Koyagawa	1955	C G	41.0	705	182					G	3	710	J	H	—	—	5	1					1959	Outlet channel	Scour	*
48	Nagayasuguchi	1955	C G	85.5	6,400	5,400					G	6	7,680	J	H	Es	50		3	6				1958–1962	Stilling basin	Erosion and scour	*
																									Outlet channel		
49	Okiura	1945	C G	40.0	344	254	C	5	21.5	372	G	4	284	J	H	Es	—		8	2							

Note ; CG : Concrete gravity dam, R : Rockfill dam, CA : Concrete arch dam, CHG : Concrete hollow grvity dam, C : High pressure conduit, T : Tunnel, O : Orifice, G : Gated spillway, UG : Ungated spillway, J : Hydraulic jump-type, Tr : Trajectory type, Wc : Water cushion-type, Sj : Ski-jump type, Rb : Roller bucket type, H : Simple horizontal apron, Hc : Laterally curved basin, Rs : Reversely sloping apron, F : Flip bucket, Tw : Trapezoidal weir, Es : End sill, Sd : Secondary dam, B : Baffle pier.