Part 3

Feasibility Study on Conventional Hydropower Projects

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Objectives and Flow of Feasibility Study

Chapter 8 Objectives and Flow of Feasibility Study

8.1 Objectives of Feasibility Study

(1) A feasibility study is conducted to objectively determine the viability of the project from the standpoint of technical, economic, financial, and social and natural environment.

A feasibility study report is used for the nations' policy makers to determine whether to implement the project. It is also used for international financial institutions to examine and determine the viability of the project.

(2) The feasibility study is broadly classified into pre-feasibility study and feasibility study. The difference depends on the accuracy and scope of the study, however is not clearly defined.

8.2 Process and Outline of Feasibility Study

Figure 5-1 in Chapter 5 describes the process in relation to the reconnaissance study in Part 2 and the feasibility study in Part 3. When the reconnaissance study concludes that the project is good enough to proceed to the next step, the feasibility study will be conducted. The flow of feasibility study is described in Figure 8-1 which content is described in Chapters 9 through 19. Although the study items vary slightly depending on the power generation type such as run-of-river type or reservoir type, and the power development scale, the general content is only described in this Manual. Therefore, it is necessary to consider the individual conditions of each project when conducting the feasibility study.

Feasibility study is outlined hereafter.

(1) Demand and supply plan

The electric power supply plan is prepared on the basis of the forecasted future power demand. Power development plan (PDP) for long period is studied taking into account the capacity of existing power plants and candidate projects. Where a shortage of supply power is expected or where an appropriate reserve margin is not ensured after the demand and supply balance is examined, new power supply sources are required. The projects studied in the feasibility study are selected as a candidate project for the additional power source and its timing of development is examined accordingly. This is described in 9.2.

(2) Meteorology, hydrology, topography and geology

Data regarding meteorology, runoff, flood discharge, sedimentation, topography, geology, construction materials, and seismic data are required for the electric power planning, structure designing, and in the construction planning of a hydropower plant. These data are first collected and analyzed in the feasibility study. Meteorology and hydrology data including rainfall, river flow, flood discharge, evaporation, suspended load, etc., are observed. The rainfall and evaporation are observed at a meteorological station. River flow, flood discharge, suspended load are observed at a flow gauging station. Data collected from satellite images is used to produce topographic maps. Regarding the geology and construction material, ground survey, aerial photograph interpretation, physical prospecting, drilling and exploratory adit methods are used. The methods of measurement and analysis of the above described data are outlined in 9.3 and 9.4.

(3) Preparation of development plan

The power generation type (run-of-river, reservoir, pondage, or pumped storage) is determined by taking into account the future power demand-and-supply, the topography and the geology of the project site. Then the layout is determined. When the environmental impact of the project is confirmed, the optimum development scale is examined comparing the economics of the different maximum outputs, dam heights, etc. When the optimum plan is selected, the basic specifications are determined for the structures such as dam height, inner diameter of headrace tunnel, maximum plant discharge, effective head, maximum output and energy. These are explained in Chapter 10.

(4) Design of civil structures, electro-mechanical equipment, and transmission and transformation facilities.

The design is carried out based on the specifications and layout determined in (3).

The civil structures include the dam, intake weir, spillway, intake, settling basin, headrace, head tank, surge tank, penstock, powerhouse, tailrace channel, and tailrace. These structures are designed based on the meteorological and hydrological data and topographic and geological data described in (2). For instance, the dam type is determined according to the topographical and geologic conditions and the construction materials available at the dam site. The construction cost estimated according to the structure design is an important factor in determining the economic viability of the project. The design is, therefore, carried out with the accuracy expected in the feasibility study stage. The basic design concept is described in Chapter 11.

The electro-mechanical equipment includes the turbines, generators and transformers, etc. The optimum turbine and generator types are determined considering the power plant operation, turbine discharge fluctuation, and other factors. The design is carried out with the accuracy expected in the feasibility study. The efficiency of the generating equipment is the fundamental value to calculate the power and electric energy. It is as important as the cost of electro-mechanical equipment to determine the economic viability of the project. These are described in Chapter 12.

The transmission plan is produced considering supply area of the hydropower plants. A power system analysis and plan of transmission line are conducted. These are described in Chapter 13.

(5) Construction plan and construction cost

The construction plan and construction schedule are prepared taking into account the project site conditions and based on the construction method according to prevailing technical level. The transportation conditions, construction conditions (labor force, power for construction work, water, and land) are naturally considered in addition to the data described in (2) and (3). The project construction cost is calculated for each item based on the design in (4) to arrive at the total project cost. The construction cost includes the cost of civil structures and electro-mechanical equipment, cost of construction administration, engineering cost for preparation of definite design and tendering of various project components, contingencies and interest during construction. To produce a financial schedule, the funds required each year are calculated for each item of work in both local and foreign currencies. These are described in Chapter 14.

(6) Environmental and social considerations

An environmental impact caused by hydropower development can be classified into physical, ecological and social impacts. Basic concept on environmental impact study, and the concrete measure to cope with these matters at the master plan study and feasibility study stages are described in Chapter 15.

(7) Economic analysis and financial analysis

The economic feasibility of the project is analyzed based on benefit and cost. The benefit is acquired from the power and energy given in (3). The cost is acquired from the construction cost given in (5). The economic analysis and financial analysis are conducted. These are described in Chapter 16.

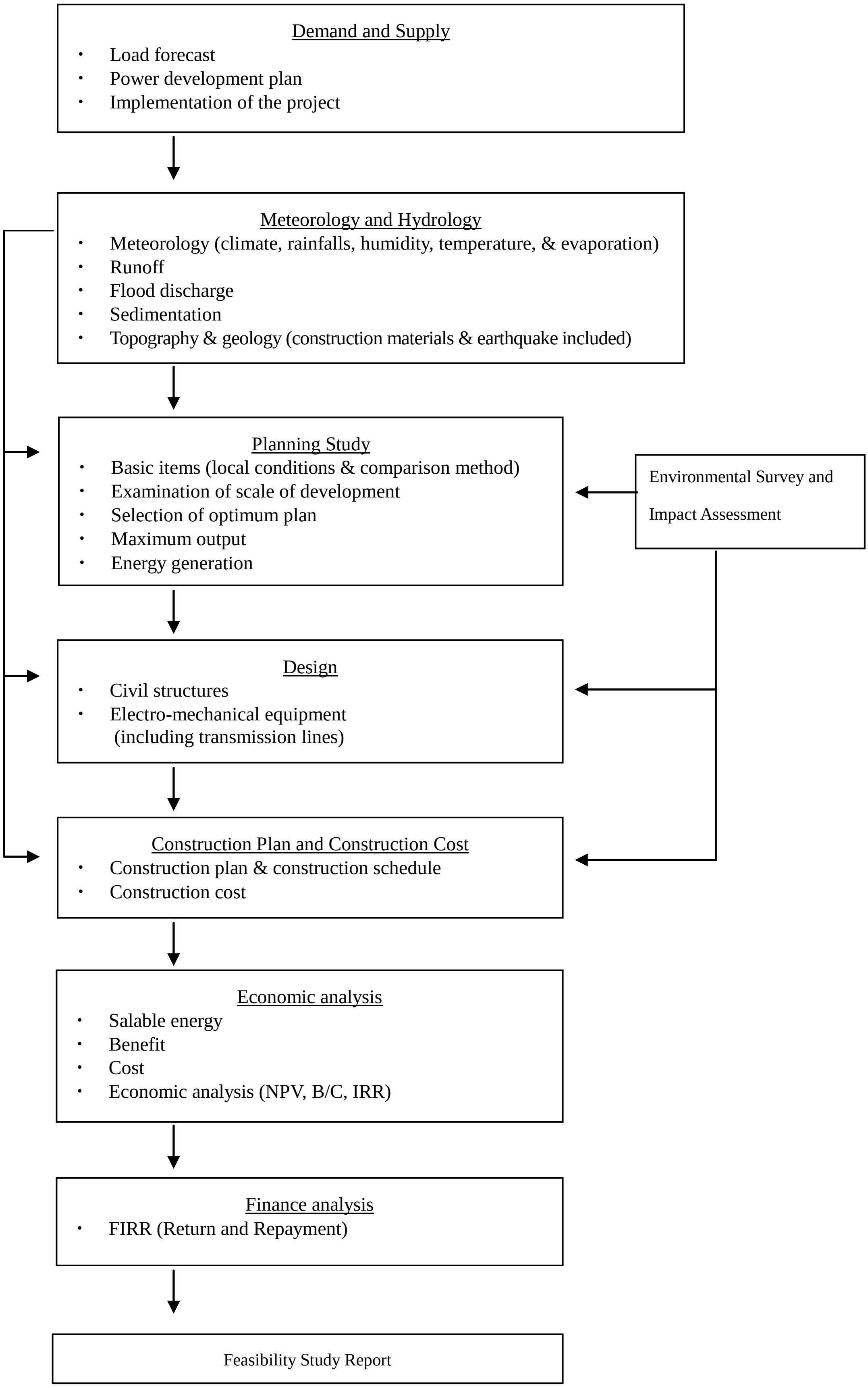


Figure 8-1 Flow of Feasibility Study

Reference of Chapter 8

[1] Guide Manual for Development Aid Programs and Studies of Hydroelectric Power Projects, New Energy Foundation, 1996

Chapter 9

Power Demand Forecast, Geological and Hydrological Studies

Chapter 9 Power Demand Forecast, Geological and Hydrological Studies

9.1 Power Demand Forecast

9.1.1 Method of Electricity Demand Forecast

(1) Models for building power development plan

Various econometric models are developed by academies and research institutions as a tool to project future electricity demand, to prepare power development plan and so on. These models are generally composed of threes engines, namely, the macro-economy model, the electricity demand forecasting model, and the power development plan model as shown in Figure 9-1.

The macro-economy model is a tool to forecast economic outlook based on investigation and analysis on national and international economic circumstance and socio-economic policies. Economic variables forecasted there such as economic growth rate and economic structure shall be input into the electricity demand forecasting model as given assumptions. The electricity demand forecasting model is a tool to forecast electricity demand based on the economic variables and incorporating investigation and analysis on electricity system, domestic and international electricity circumstance and electricity policies. The power development plan model is a tool to derive the optimum electricity supply pattern against the forecasted electricity demand incorporating supply possibilities and price movements of various energy sources such as oil, gas, coal, and hydro, energy policies and other preconditions. In general, these three engines are used to examine effects of various policies and to construct an optimized energy plan under the forecasted world energy movement and political and economic circumstance inside and outside the country.

However, building of the macro-economic model requires enormous efforts. Therefore, the macro-economic model is not built and the economic plan (target) of the country is often used as the economic variables for the demand forecasting model. The power development plan model seeks optimization using a minimum cost or a maximum profit as target function by the linear programming. The model known well in developing countries is Wien Automatic System Planning (WASP) developed by International Atomic Energy Agency. In this section, the electricity demand forecasting model is described in detail.

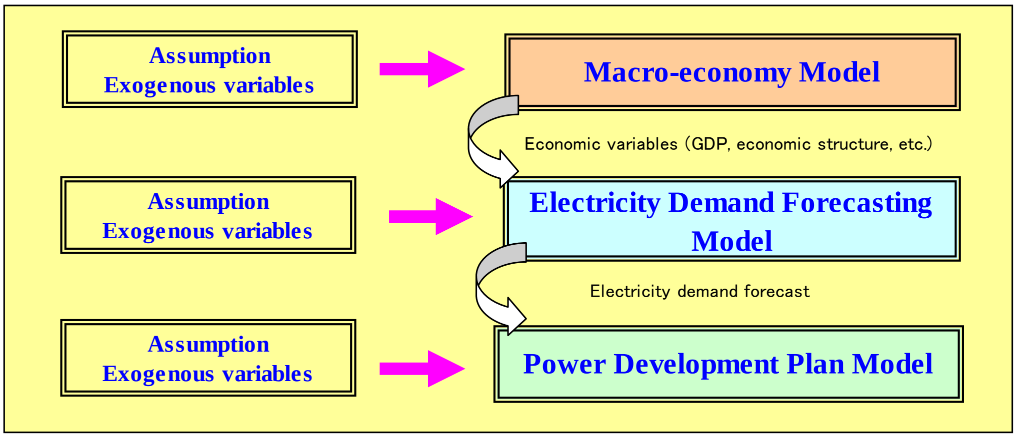


Figure 9-1 Models for Building Power Development Plan

(2) Structure of electricity demand forecasting model

The electricity demand forecasting model is composed of the two blocks as shown in Figure 9-2: the macro economic block and the electricity demand block.

As explained in the previous section, economic indicators are in principle given to the model referring to those announced by the government and/or projected by relevant offices as exogenous variables. Thus, the model reflects various economic and industrial policies stipulated under the Socio-economic Development Plan and other plans. The necessary time-series data (past statistics) for the electricity demand forecasting model are actual electricity demand in the past and prospects of the population, the number of households, GDP, the electricity tariff, the electrification rate, etc. If these data cannot be collected in the time-series, it is not possible to use it as a variable of the forecasting model. As for the period of the necessary time-series data, if you want to forecast for ten years, it is preferable to collect data for the past ten years.

In building the model, as much data as generally available are used to make model maintenance and operation easier. In line with the continued economic development, the energy demand of developing countries will be affected greatly by impacts of changes in industrial energy consumption pattern, changes in life style, energy conservation policies and so on. Therefore, it is strongly recommended to establish a data collection system to continuously watch indicators of these trends.

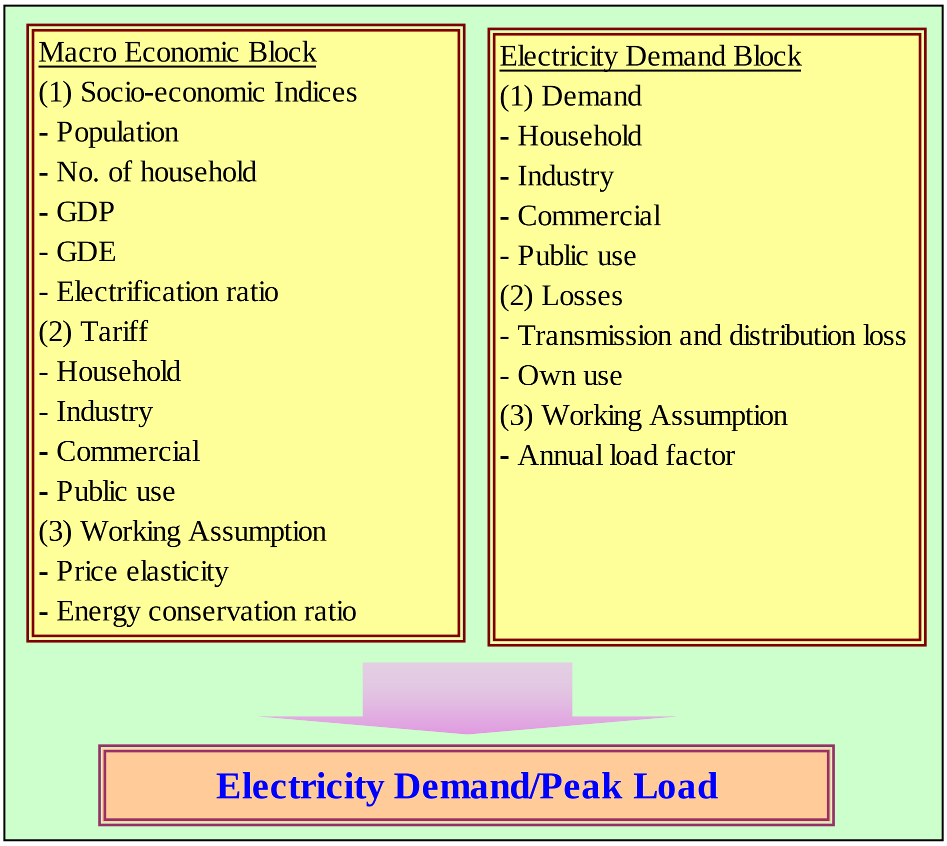


Figure 9-2 Outline of the Electricity Demand Forecasting Model

(3) Model equation

The model equation to estimate the electricity demand in the future is estimated from the correlation of the variables such as GDP, number of household, population, electricity tariff, income, electrification ratio, and so on that can explain the past electricity demand and the electricity demand by the regression analysis. There is a lot of commercially available software for regression analysis tool. The analytical result (correlation) is the same as any software. In case of the electricity demand forecasting model, coefficient of determination[1] (R^2) that show the correlation are generally high and is 0.9 or more because the accuracy of power statistics is higher than that of other energy statistics and the substantial change is generally a little in electricity demand every year.

If GDP and electricity tariff are used as variables, it should be converted from nominal prices to real prices. The variables with a comparatively high correlation that controls the electricity demand are shown below.

1) Electricity demand for industry

Major variables: GDP of industry, Indices of industrial production, Electricity tariff

Of the above-mentioned variables, it is thought that industry GDP and indices of industrial

[1] The coefficient of determination is a calculation of the accuracy of the model in explaining the data.

production have a strong correlation. When using two variables that have such a strong correlation, the analysis might be impossible. Even if the result is obtained, the reliability is low. In this case, it is desirable to use either variable (high coefficients of determination). It is said the multicollinearity when there are two variables or more with a high correlation in one equation. The following is example of model equation.

$y = f$ (industrial GDP, electricity tariff)

$y = a \times$ industrial GDP – $b \times$ electricity tariff + c

2) Electricity demand for commercial

Major variables: GDP of commercial, Floor space, Electricity tariff

In general, a floor space has a high correlation with electricity demand for commercial sector. However, in developing countries, there is no statistics of floor space for commercial buildings. In this case, commercial GDP is used instead of floor space as variable.

$y = f$ (commercial GDP, electricity tariff)

$y = a \times$ commercial GDP – $b \times$ electricity tariff + c

3) Electricity demand for residential

Major variables: Population, GDP, Income, Private consumption expenditure, Electrification ratio, Electricity tariff

Of the above-mentioned variables, it is thought that GDP, income, and private consumption expenditure have a strong correlation each other and multiocollinearity exists. It is necessary to choose the variable with a high coefficient of determination.

$y = f$ (income, electrification ratio, electricity tariff)

$y = a \times$ income + b $\times$ electrification ratio – $c \times$ electricity tariff + d

As above-mentioned, three model equations were given to the examples. It is noticed that the sign of the coefficient of the price (electricity tariff) variable is a minus of all equations. This is an economic theory. It means when the electricity tariff increases, the electricity demand decreases. Positive coefficient of the price shows if the electricity tariff increases, people will use electricity more than before. But this is not real world. As the results of regression analysis, sometimes the coefficient of the price becomes a plus. In this case, the electricity tariff cannot be used as a variable.

(4) Test and evaluation of model equations

The demand forecasting model is an econometric model and is formulated with regression equations and arithmetic equations. For selection and evaluation of regression equations, there are several kinds of testing methods. In this model, the following tests are conducted for selection of the regression equations.

1) Evaluation of demand forecasting equations

- Determination coefficient (target: more than 0.85)
- T-value[2] test of parameters (target: more than 2.0)
- Durbin Watson ratio[3] test (target: 1<DW<3)
- Sign test of the regression coefficient (logical according to economic theory)

2) Evaluation for the result of the electricity demand forecast

- Is the electricity elasticity to GDP[4] in the future realistic?

  In general, the electricity elasticity to GDP changes by about one. Though the electricity elasticity might temporarily show 1.5 or more in the developing country in the short period of time, it is usual between 1-1.5 on an average. It is necessary to review the model equation if the elasticity is too large.

- Is the electricity consumption per capita realistic?

  The electricity consumption per capita at the present and the future is compared. Moreover, it compares estimated electricity consumption with that in the surrounding countries and verifies whether the estimated consumption is appropriate.

  Here, we should note that, regression equations are calculated on the past statistical figures, that is, on the past trend, but our future should not be a simple copy of the past. In developing countries, experiencing a rapid developing stage now, economic structure and people's life style will change quickly. We could not foresee the future simply extrapolating the past trend. We need to carry out versatile analysis with regard to shift of the economic development stage, change in the economic structure, life cycle of popular commodities, etc.

9.1.2 Example of Electricity Demand Forecast

Here, the electricity demand in the future is actually forecasted by using the past Japanese statistics.

(1) Time-series data

Time-series data are the electricity demand by sectors, the electricity tariff (nominal price), GDP by sectors (nominal price), GDP deflator, the floor space for commercial buildings, the private consumption expenditure (nominal) from 1980 to 2000 as shown in Table 9-1. The electricity tariff at real price in Japan from 1980 to 2000 has gradually decreased as shown in Figure 9-3. On the other hand, the electricity demand for residential and commercial sectors, the private

[2] T-value measures the statistical significance of an independent variable. Generally, any t-value greater than +2 or less than - 2 is acceptable.

[3] The Durbin-Watson ratio is a test statistic used to detect the presence of autocorrelation in the residuals from a regression analysis. The value of d always lies between 0 and 4. If the Durbin–Watson ratio is substantially less than 2, there is evidence of positive serial correlation. If the Durbin–Watson ratio is substantially more than 2, there is evidence of negative serial correlation.

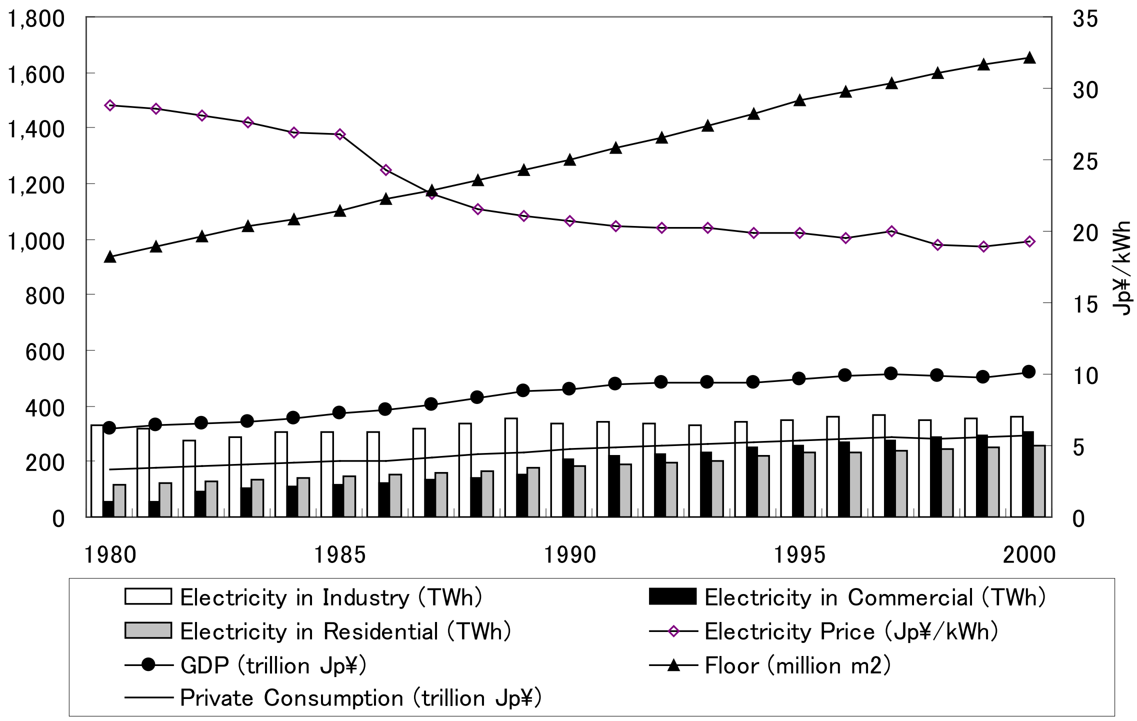
[4] Electricity elasticity to GDP = Growth rate of electricity demand/Growth rate of GDP

consumption expenditure, the floor space gradually increase and the electricity demand for industry sector remained at the same level.

Table 9-1 Time-Series Data for Electricity Demand Forecast

	Unit	1980	1981	1999	2000
1. Electricity Demand					
(1) Industry Sector	GWh	327,791	320,512	354,733	361,628
(2) Commercial	GWh	52,953	57,000	296,174	303,721
(3) Residential Sector	GWh	116,093	121,267	251,698	257,849
(4) Grand Total	GWh	496,837	498,779	902,605	923,198
2. Electricity Price (nominal)	Jp¥/kWh	22.49	23.14	18.66	18.65
3. Gross Domestic Product (Nominal)					
(1) Industries	billon Jp¥	228,618	244,098	474,262	475,943
1) Agriculture ,forestry and fishing	billon Jp¥	8,778	9,030	9,279	8,896
2) Mining	billon Jp¥	1,363	1,276	645	627
3) Manufacturing	billon Jp¥	68,093	72,612	110,125	111,439
4) Construction	billon Jp¥	22,228	24,200	38,133	37,130
5) Electricity ,gas and water supply	billon Jp¥	6,576	7,381	14,068	13,576
6) Wholesale and retail trade	billon Jp¥	36,780	38,481	73,066	70,661
7) Finance and insurance	billon Jp¥	12,034	11,706	30,218	30,445
8) Real estate	billon Jp¥	22,644	24,391	65,130	66,342
9) Transport and communications	billon Jp¥	15,522	16,868	34,947	34,821
10) Service activities	billon Jp¥	34,602	38,154	100,726	103,752
(2) Producers of government services	billon Jp¥	19,076	20,606	43,827	44,539
(3) Producers of private non-profit services to households	billon Jp¥	3,286	3,542	9,977	9,343
(4) Total GDP	billon Jp¥	250,980	268,246	497,629	502,990
4. GDP Deflator	1995=100	78.2	81.1	98.56374	96.85544
5. Total Floor Space for Commercial Sector	million m2	936	972	1,630	1,655
6. Private Consumption Expenditure (Nominal)	billon Jp¥	134,233	142,604	283,880	282,772

Source: Handbook of Energy & Economic Statistics in Japan, IEEJ



Source: Handbook of Energy & Economic Statistics in Japan, IEEJ

Figure 9-3 Relationship Between Electricity Demand and Major Variables

(2) Model equation for electricity demand forecast

The model equation of the electricity demand forecast estimated by the regression analysis using the past time-series data is as follows.

1) Electricity demand in industry sector

INEL = 506,213-25.745 × INGDP/DEF - 647,614 × ELP/DEF

(4.14) (-0.376) (-2.89) ------------------------- t-value

INEL: Electricity demand in industry, INGDP: GDP of manufacturing, DEF: Deflator, ELP: Electricity tariff

Determination coefficient: 0.679, Durbin Watson ratio: 1.23

Dividing GDP of manufacturing in industry sector and the electricity tariff by deflator in the above-mentioned model equation means a nominal price has been converted into the real price. Seeing the absolute value of t-value, the coefficient of GDP is considerably low with 0.376. This means GDP doesn't influence the electricity demand. Moreover, GDP should not be used as a variable because the coefficient of GDP is a negative. This means when the GDP increases, the electricity consumption decreases. This contradicts the economic theory. On the other hand, the absolute value of t-value for the electricity tariff coefficient is 2.89 and the electricity demand in industry sector is sensitively reactive to the electricity tariff. Coefficient of determination is lower than 0.85 of the targets as mentioned on (4), 1) in this section and it is necessary to examine a further variable to improve coefficient of determination. The Durbin–Watson ratio shows a positive serial correlation of 1.23. From above reason, the estimated equation using the price (electricity tariff) variable only is as follows.

INEL = 460,841 - 570,897 × ELP/DEF ------------------------------- i)

Determination coefficient: 0.676, Durbin Watson ratio: 1.16

2) Electricity demand in commercial sector

As for the model equation of the electricity demand for commercial sector, two pairs of variables (GDP and electricity tariff, floor space and electricity tariff) were analyzed. The model equation and the evaluation results of the regression analysis are as follows.

(a) Variables: GDP and electricity tariff

COEL = 13,499 + 509.8 × COGDP/DEF - 624,630 × ELP/DEF

(0.128) (6.39) (-2.37) ------------------------- t-value

COEL: Electricity demand in commercial sector, COGDP: GDP of wholesale and retail trade, DEF: Deflator, ELP: Electricity tariff

Determination coefficient: 0.953, Durbin Watson ratio: 0.69

(b) Variables: Floor space and electricity tariff

COEL = -211,670 + 334.12 × FLR – 159,575 × ELP/DEF ------------------------- ii)

(-2.45) (10.4) (-0.776) ------------------------- t-value

COEL: Electricity demand in commercial sector, FLR: Floor space, DEF: Deflator, ELP: Electricity tariff

Determination coefficient: 0.978, Durbin Watson ratio: 1.1

Comparing with two model equations, as for the correlation between the electricity demand for commercial sector and variables, the floor space is stronger than GDP because determination coefficient and Durbin Watson ratio of (b) equation are better than those of (a) equation. On the other hand, the electricity tariff doesn't influence the electricity demand so much because the absolute value of t- value for coefficient of electricity tariff is low. However, there is no problem even if the electricity tariff is left as the variable to show that the price is considered in this equation. In the sign test of the coefficient, the floor space is a positive and the electricity tariff is a negative. This means that the electricity demand is increasing according to increase of floor space and the electricity demand is decreasing according to increase of the electricity tariff. This agrees with the economic theory.

3) Electricity demand in residential sector

As for the model equation of the electricity demand for residential sector, two pairs of variables (GDP and electricity tariff, private consumption expenditure and electricity tariff) were analyzed. The model equation and the evaluation results of the regression analysis are as follows.

(a) Variables: GDP and electricity tariff

REEL = -413,889 + 99.98 × GDP/DEF + 720,887 × ELP/DEF

(-2.82) (5.91) (2.2) ------------------------- t-value

REEL: Electricity demand in residential sector, GDP: Total GDP, DEF: Deflator, ELP: Electricity tariff

Determination coefficient: 0.945, Durbin Watson ratio: 0.76

(b) Variables: Private consumption expenditure and electricity tariff

REEL = -244,568 + 143.86 × PCE/DEF + 396,061 × ELP/DEF

(-5.73) (16.5) (3.97) ------------------------- t-value

REEL: Electricity demand in residential sector, PCE: Private consumption expenditure, DEF: Deflator, ELP: Electricity tariff

Determination coefficient: 0.99, Durbin Watson ratio: 1.2

Comparing with two model equations, as for the correlation between the electricity demand for residential sector and variables, the private consumption expenditure is stronger than GDP because determination coefficient and Durbin Watson ratio of (b) equation are better than those of (a) equation. Moreover, the absolute value of t- value for coefficient of all variables shows 2 and more. This means that all variables have the correlation with the electricity demand. In the sign test of the coefficient, however, the coefficient of the electricity tariff is a positive. This means that the electricity demand is increasing according to increase of the electricity tariff. This does not agree with the economic theory. In this case, we cannot use the variable of the electricity tariff. The following is the equation that removes the variable of the electricity tariff.

REEL = -76,799 + 110.79 × PCE/DEF ------------------------- iii)

Determination coefficient: 0.981, Durbin Watson ratio: 0.39

(3) Comparison between actual and results of model equation

Figure 9-4 to 9-6 show the comparison between actual electricity demand and the result of above-mentioned model equations, i), ii), and iii). The solid line is actual electricity demand in the past, the dotted line is the result from the estimated equation of the model, and the line with triangular marker is the residual between actual electricity demand and the result of model equation (difference between actual and the model result). As is evident in the figures, the result of the model equation for residential electricity demand that the determination coefficient is the highest is almost corresponding to actual demand.

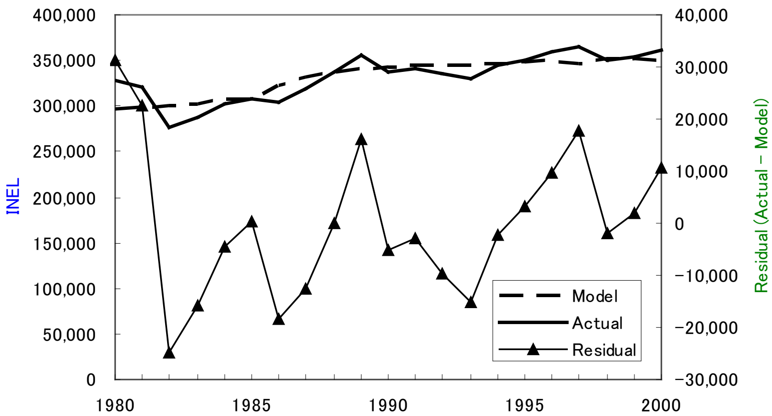


Figure 9-4 Comparison Between Actual and Results of Model Equation for Industry Sector

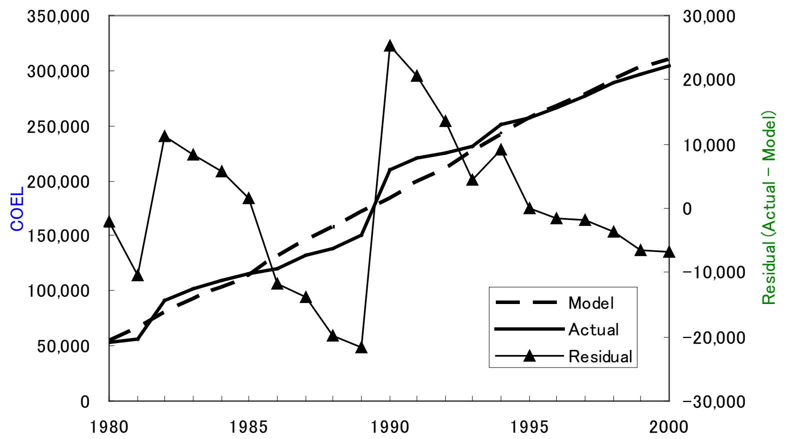


Figure 9-5 Comparison Between Actual and Results of Model Equation for Commercial Sector

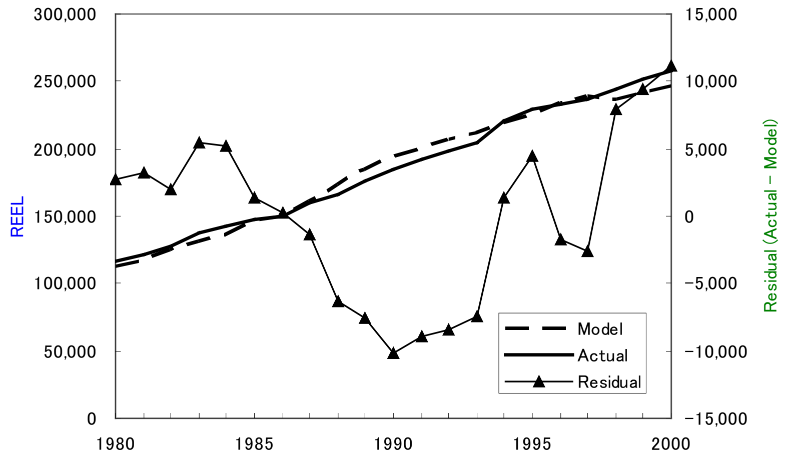


Figure 9-6 Comparison Between Actual and Results of Model Equation for Residential Sector

(4) Electricity demand forecast

Figure 9-7 to 9-9 show the electricity demand forecast from 2001 to 2006 that is estimated by the model and actual demand. Actual electricity demand in industry and commercial sectors from 2001 to 2006 is slightly lower than the result of model equation. It is considered that the energy efficiency of electric appliances was promoted. Thus when estimating the electricity demand in the future, it is necessary to consider the government policy for energy efficiency and conservation.

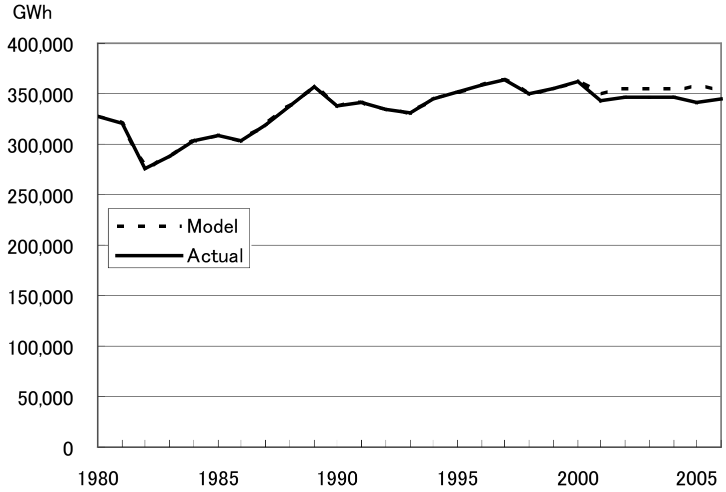


Figure 9-7 Comparison Between Actual and Model Forecast for Industry Sector

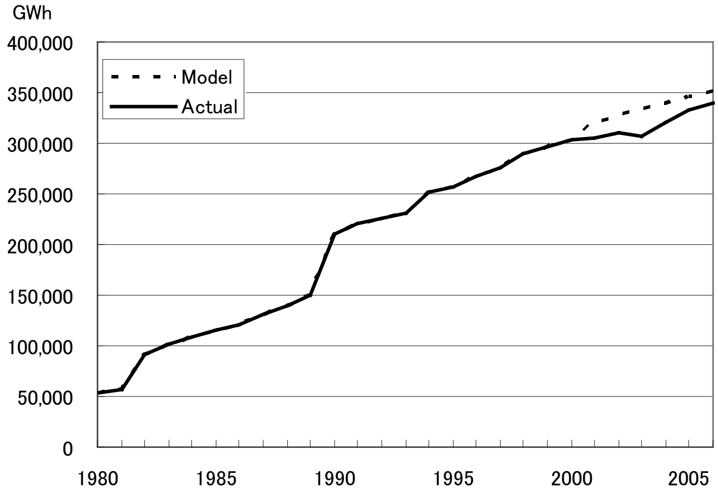


Figure 9-8 Comparison Between Actual and Model Forecast for Commercial Sector

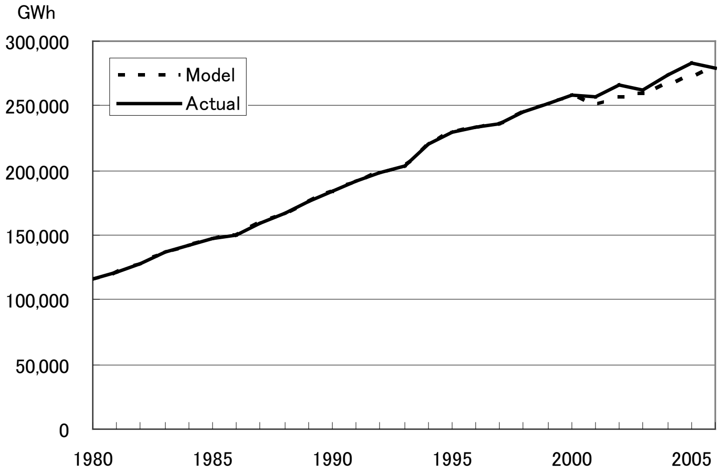
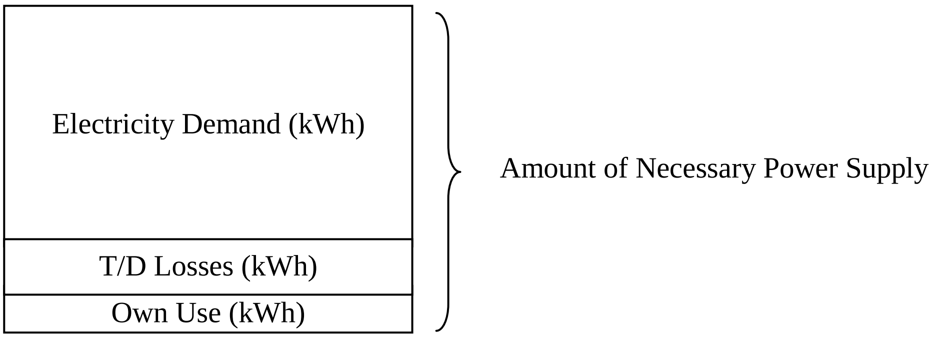


Figure 9-9 Comparison Between Actual and Model Forecast for Residential Sector

(5) Amount of necessary power supply

Previously, the electricity demand consumed by the end user has been forecasted. But as for amount of necessary supply for the power plant, it is necessary to consider own use in the power plant and transmission and distribution (T/D) losses. Usually, own use and T/D losses in developed countries are about 5% respectively. However, those of developing countries trend to be higher than those of developed countries. Amount of necessary supply for the power plant is defined by the following equation.

$$\text{Amount of Necessary Power Supply} = \frac{\text{Electricity Demand}}{1 - (\text{T/D Losses \%} + \text{Own Use \%})/100}$$



(6) Required power supply capacity

What the power plant is required is to supply necessary electricity. However, amount of necessary power supply fluctuates over time. In the developed country, the peak load appears in daytime that needs air-conditioning and the factory demand. On the other hand, in the developing country, the peak load appears in nighttime for lighting demand. Anyway, the electric power company has data for the peak load. The ratio of the peak load and the average load is called a load factor and it is shown in the following equations.

$$\text{Annual Load Factor (\%)} = \frac{\text{Annual Average Load (kW)}}{\text{Annual Peak Load (kW)}} \times 100$$

$$\text{Annual Average Load (kW)} = \ = \frac{\text{Annual Demand (kWh)}}{365\ \text{days} \times 24\ \text{hours}}$$

From the above-mentioned two equations, required power supply capacity is defined by the following equation. Actually, several percent of reserve capacity must be added for maintenance and accident of the power plant.

$$\text{Required Power Supply Capacity} = \frac{\text{Annual Demand (kWh)}}{365\ \text{days} \times 24\ \text{hours}} \times \frac{100}{\text{Annual Load Factor (\%)}}$$

9.1.3 Power Development Plan

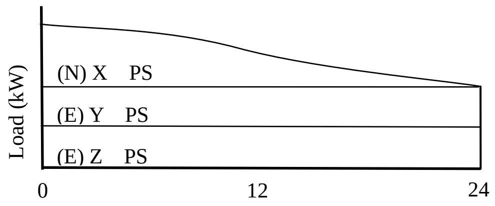
A power development plan shows an introduction plan of various kinds of new power projects and renewal plan of existing power plants based on a long term, more than 10 years, power demand forecast. A power development plant shall be established so as to minimize total investment costs and operation and maintenance costs of all power plants taking into account a development schedule.

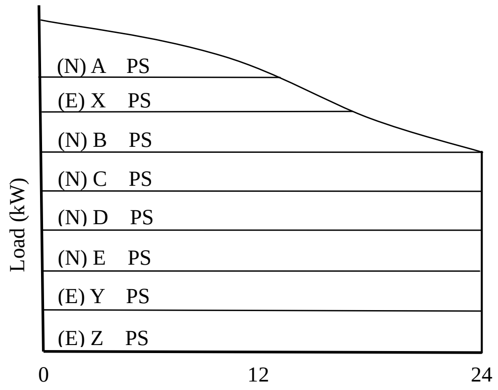
WASP (Wien Automatic System Planning Package), which was published by IAEA (International Atomic Energy Agency in 1974, is widely used in the world as a tool to minimize a total cost of a power development plan. There exists the version 4 of WASP so far.

(1) Principle of WASP

A dynamic programming, which is commonly applied for solving an optimization of nonlinear combinational problem, is used in WASP. Skipping a detailed explanation of a dynamic programming, it is a classical method which is applied to optimization problems such as maximization or minimization of an objective function. The objective function is minimization of total investment costs and operation and maintenance costs of all power plants, here.

It is possible to find a optimal development plan, that is minimization of total costs, for instance among A, B, C, D and E candidate projects, to satisfy the power demand 10 years later by using WASP.





Note: (N) means new project and (E) means existing plant

Figure 9-10 Present Power Load Curve Figure 9-11 Power Load Curve 10 Years Later

(2) Main input and output data of WASP

WASP is a tool to find an optimal power development plan which satisfies a power demand in the future, and required input data for simulation and indicated output data are as followings.

1) Input data

- Power demand (load curve)
- Data of existing power plants (such as firm capacity, power loss for station use, forced outage, etc., for hydropower)
- Data of candidate power plants (such as construction cost, O & M costs, fuel costs, etc.)
- Constrains of power development plan (such as margin rate, LOLP (loss of load probability), etc.)

2) Output data

- Power development plan with minimum costs
- Total power energy
- Total costs
- Power supply reliability (such as margin rate, LOLP, etc.)

An example of optimized power development plan derived from WASP is shown in Table 9-2.

Table 9-2 An Example of Optimal Power Development Plan

Year	New Project (Hydro)	New Project (Thermal)	Retirement	LOLP (%)
2011	A Hydro			0.12
2012				0.88
2013		C Thermal		0.02
2014				0.26
2015	B Hydro			0.02
2016				0.67
2017		E Thermal		0.11
2018				0.54
2019		D thermal		0.08
2020				0.37
Total Cost (NPV) up to 2020 : 9,920 million US$				

3) Sensitive analysis

Since various assumptions and unknown factors in the future are included in the process of deriving an optimal power development plan, reasonableness of the plan shall be checked in a comprehensive manner based on sensitive analysis against following parameters.

- Power demand forecast (high, medium and low scenarios)
- Discount rate
- Price escalation
- Fuel price

9.2 Topographical and Geological Studies

9.2.1 Topographic Survey

(1) Topographic Maps Required

For a reconnaissance study of Chapter 5 through Chapter 7, topographical maps of 1:50,000 are sufficient. In the subsequent feasibility study for the optimization of electrical power planning of a project, maps of 1:5,000 should be used. The topographical maps should cover areas of major civil structures such as dam, waterway, and powerhouse, and reservoir. The maps should include access roads for construction, borrow areas, and for temporary facilities.

The scale of topographical maps should be 1:5,000 or 1:10,000 for small reservoirs, and the same 1:10,000 or 1:25,000 for large ones. Maps of 1:1,000 to 1:2,000 should cover the areas of major structures like dam and powerhouse.

(2) Preparation of Topographical Maps and Survey on Altitude of Intake and Tailrace

Satellite pictures are useful for preparation of topographical maps for feasibility study. In developing countries access to the planned project site is likely to face difficulty due to road conditions and other restrictions. The use of satellite images eliminates the needs for site access and enables to develop topographical maps of 1:5,000. The maps of this scale can be enlarged to 1:2,000 for use in feasibility study. Where site access is possible, the accuracy of survey can be improved by control point measurements (coordinate and altitude) using the GPS. Static measurements using GPS should improve accuracy to within a few centimeters of errors. The gross head between intake and tailrace is one of the major factors to determine power plant output. The accuracy of the head can be accomplished by leveling data with GPS measurements.

(3) Cross Section of River

Rating curves at water intake point prepared by cross sectional survey of a river enable evaluation of the output of hydropower plant.

9.2.2 Geological Survey

Figure 9-12 shows a flowchart of the geological survey which is commonly applied to a hydroelectric power project on each stage from its planning to the construction.

Geological survey is carried out in the feasibility study to investigate the geologic structure, the rock type and the physical/mechanical properties of the bed rock in the project area. The data is used to prepare geological maps for the design of the civil structures. These surveys are also to determine the feasibility of the project from the geological engineering aspect.

Generally, geological survey on the F/S stage involves geologic mapping, aerial photo interpretation, geophysical prospecting and exploratory drilling, and in case of necessity, exploratory adit may be excavated.

It is also necessary to study the distribution, available quantity, and physical/mechanical characteristics of available materials for collecting concrete aggregates and fill materials for embankment dam.

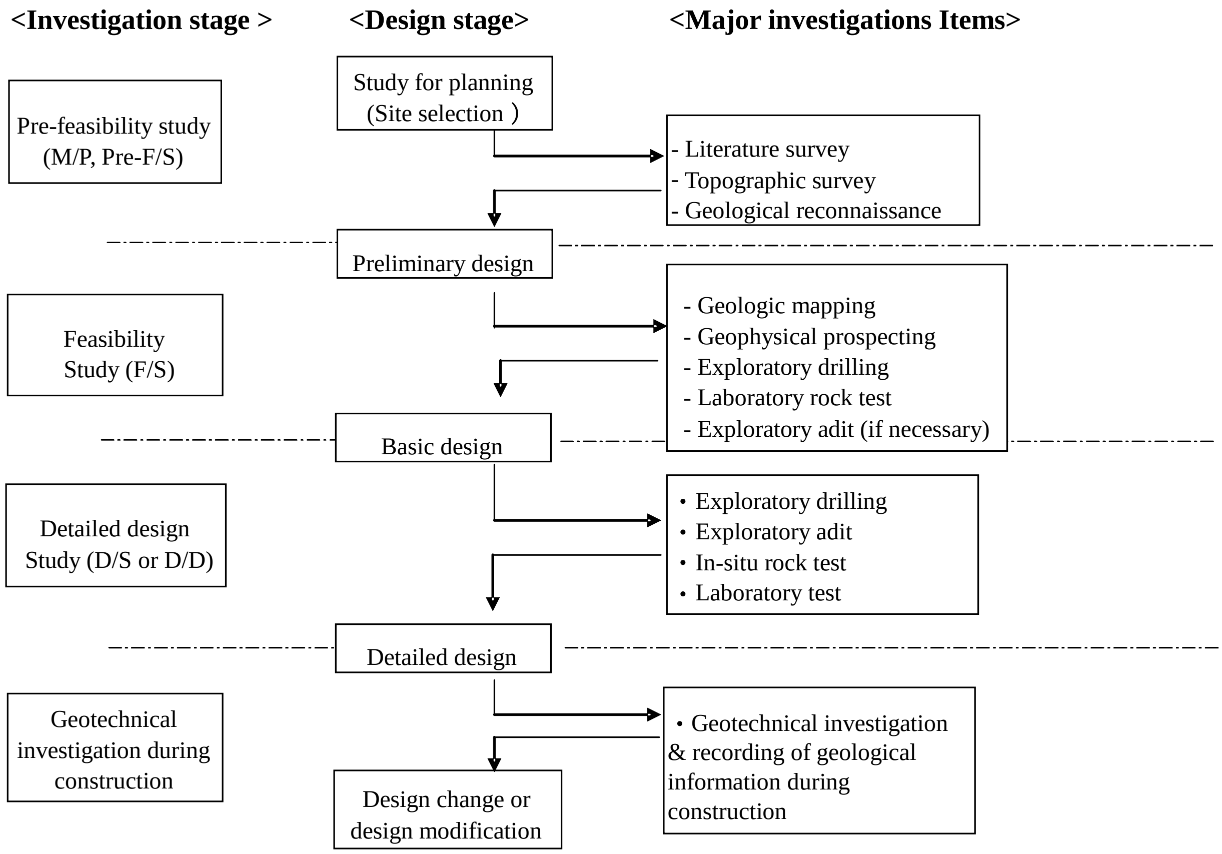


Figure 9-12 Flowchart of Geological Study & Survey on Hydropower Project

(1) Geologic mapping

In the case of conducting geologic mapping on F/S stage, 1:5,000 to 1:25,000 scale topographic maps are commonly used for the reservoir area and the headrace (waterway) route(s), and 1:1,000 to 1:2,000 scale maps are for the major structure sites, such as dam, powerhouse, etc.

Prior to conducting the geologic mapping, it is necessary to gain an understanding of general geologic features of the project area using the existing geologic maps/data concerned with the area. Further, the interpretation of geographical features by the aerial photo and the large scale topographical maps of, around 1:50,000 will serve the understanding of the topographic features which are indicative of the rough geologic structures, patterns of faults and regional joint-sets, distributions of talus, alluvial-fan, lava plateau, karst topography, landslide, etc.

When conducting surface geologic survey, the survey route is selected on the map to prepare a rough survey plan. The most effective survey routes are river valleys and roads where the geologic formation is clearly exposed. However, a route crossing the geologic structure should be selected when a river flows along a strike of sedimentary rock since only the same horizon is surveyed in some cases.

Additionally, the geologic mapping routes and spheres are to be decided so as to confirm important information acquired by the existing geologic maps/data and aerial photographs.

The main observation items for geological survey are described below.

1) Observation items on geological outcrops

(a) Lithofacies

- Type of bedrocks (consolidation degree, genesis and component substances for unconsolidated sedimentary deposits)
- Distribution of the stratum (conformity, unconformity, strike & dip)
- Joint pattern (type, strike & dip, continuity, intervals)
- Characteristics of bedding and schistosed planes (strike & dip, thickness of beddings, characters of fissility/exfoliation)
- Geologic age (fossil)

(b) Lithologic characters

- Hardness of bedrocks
- Conditions of Weathering and alteration
- Conditions of crack/joint (loosened degree, type of plugging/filling materials)
- Remarkable phenomena (impregnation, eruption, secondary deposits, cavity)

(c) Geologic structure

- Folding structure (minor fold, syncline and anticline)
- Fault (strike & dip, width and conditions of sheared zone, continuity, branch fault)
- Outcrops of surface and underground water (spring/outflow channel, water quantity, quality, temperature, underground flow and extinction)

2) Observation items on overburden (deposits) where there are no outcrops of bedrocks

- Type of deposit (top soil, talus, humus, sand and gravel)
- Genesis (fluviatile, aeolian, residual soil, colluvial soil, volcanic ash, debris flow, alluvial fan and terrace)
- Characteristics (tightness, grading, moisture content)
- Distribution (thickness, spread, configuration, stratification, and continuity)
- Vegetation (plant species, age, planted or natural)

3) Observation items on topography

- Valley configuration (width, water depth, river gradient and hillside slope angle)
- Flat surface configuration (elevated peneplain, volcanic plateau and terrace)
- Specific topography (Karst, landslide, failure, fault valley)

The observation results are entered onto the topographic map along the survey route and a route map is provided. Locations of photos and collected samples are noted on the route map, and the detailed sketches of outcrops are recorded as required.

Geologic plans and sections summarize the geologic route maps recorded by the geologic mappings and contribute to make up engineering geologic maps, which could offer important geotechnical features for the basic designs, together with underground information obtained from the geophysical explorations and the exploratory drillings which are described later.

(2) Aerial photo interpretation

Aerial photographs are used not only for drawing up the topographic maps, but also as a supplementary means for the geological survey. The reduced scales of aerial photographs vary according to the purposes, however, from one to several thousands to 1: 40,000, commonly.

Aerial photo interpretation is done with a pair of photographs and a stereoscope. Although portable stereoscopes are available, a reflecting stereoscope is used for indoor work.

Aerial photographs provide clear topographic relief through the stereoscope. Also, specific color tones and darkness can be emphasized by selecting the appropriate film and exposure conditions. Aerial photographs, therefore, enable easier interpretation than topographic maps or on-site topographic interpretation, and larger scale geologic structures such as rock distribution, faults and joint systems, occurrence of surface deposits, landslides/slope collapses etc. can also be interpreted through analysis of the identification elements combined with color tones.

Moreover, landslides and slope collapses can be detected their in-situ progressive conditions by the aerial photos taken at different times.

(3) Geophysical prospecting

Geophysical prospecting is a method to sound the underground geologic and hydrologic structures since the bedrocks with their overburdens forming the ground indicate different geophysical characteristics according to their types, weathering and fracturing degrees and in-situ local conditions. As a rule, geophysical prospecting includes seismic prospecting (velocity and penetration of elastic wave propagation), electric prospecting (electric characteristics including resistivity and spontaneous potential), gravity prospecting (density), and magnetic prospecting (geomagnetic field). Of geophysical prospecting, a method conducted in one drillhole is called as geophysical logging which includes “velocity logging” and “electric logging”. And, other geophysical methods using one or more drillholes and/or exploratory adits are called as geo-tomography which includes “Seismic tomography”, “Resistivity tomography” and etc.

Here, seismic prospecting is described as it is the most generally used method for the hydroelectric power projects. The propagating velocity of elastic wave varies in the ground according as its geologic conditions. Therefore, elastic waves are generated by an explosion or by hammering on the ground surface or underground, and the arrival time of the primary or longitudinal wave is measured to assume the underground structures that are characterized by the elastic wave velocities.

Elastic wave or seismic wave prospecting form the ground surface is divided into the refraction and the reflection methods, and the former one is commonly applied on the project sites. The outline of the refraction method, which is also called as Primary- or, simply, P-wave refraction prospecting, is as follows:

The subjects of the prospecting are to estimate the thicknesses of riverbed and/or talus deposits covering the bedrocks, weathering degrees of the bedrocks, locations/widths of any of fractured/ faulted zones in the bedrocks, and so on.

Figure 9-13 shows the principle of the refraction prospecting and Time- Distance curve. The diagram shows an underground structure model in which the upper layer provides velocity V_1 and where the lower layer provides velocity V_2. ($V_2 > V_1$). When the seismic wave generated at Point A is received at Point P, and the distance AP is shorter than x, the wave passing through the upper layer arrives at Point P first. When the distance is farther than x, the wave passing the lower layer arrives at Point P first after being refracted at the boundary line of velocity. Therefore, the relation between the distance and arrival time shows a polygonal line. The inverse of the gradient at each straight line indicates the velocity. This graph is called the travel time curve. The thickness (h) of the upper layer with V_1 can be acquired by the following equation;

$$h = \frac{x}{2}\sqrt{\frac{V_2 - V_1}{V_2 + V_1}}$$

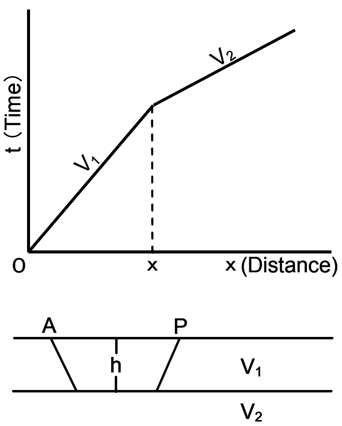


Figure 9-13 Principle of Refraction Prospecting and Time-Distance Curve.

In the case of refraction prospecting, some several to some tens of prospecting lines are located over the surveyed area. The length of these prospecting lines must be at least five times longer than the depth to be prospected. From the plane view, it is necessary to extend the lines slightly to the outside (approximately 1-2 times the prospecting depths) of the survey coverage area.

It is generally expected that the prospecting precision by the refraction prospecting is higher where the velocities of the bedrocks are larger and/or their geologic structures are simpler than these are lower and/or more complex. For instance, the precision of the prospecting should be the most excellent to estimate the thicknesses of overburdens/surface deposits (riverbed deposits and/or scree, etc.) with low elastic-wave velocities where they overlie bedrocks (Paleozoic beddings and/or massive granites) with high elastic-wave velocities. Following this, the prospecting is expected to estimate subsurface weathered conditions of the bedrocks and/or locations/sizes of fractured/faulted zones, rather precisely.

The expected precision of the prospecting is, however, rather low in the following cases; to estimate the thicknesses of overburdens/surface deposits overlying bedrocks showing rather low elastic-wave velocities such as sedimentary rocks of Neogene period (sandstone, mudstone, etc.), to figure locations of small scaled fissures/ faulted zones in the bedrocks, and so on.

It should be reminded that, in principle, distributions of the elastic-wave velocity patterns in the subsurface ground are of the results of the refraction prospecting, that is, they do not indicate directly the stratigraphic layers and lithologic types/difference of geologic origin of rocks, even if their elastic-wave velocities are same.

In order to analyze the results of the refraction prospecting, effectively and precisely, it is necessary that they should be judged by comparison with geologic cores which are recovered from drillholes located on intersecting points of the prospecting lines and/or along the lines.

(4) Drilling investigation

1) General feature of investigation

Drilling exploration purposes to acquire information regarding the geologic conditions of the subsurface ground by geologic observation and laboratory tests of the recovered cores and, also, by various in-situ tests in the drillholes.

To a certain degree, subsurface geologic conditions can be assumed by surface geological survey and geophysical prospecting, it is, however, insufficient to grasp the practical geologic conditions. Accordingly, investigations by drillhole and exploratory adit are required to confirm the subsurface geologic conditions, more in detail. Investigation of drillhole is commonly superior to that of exploratory adit on the project site, from the view point of economical performance, construction period, safety control, and environmental protection.

On the other side, only one drillhole investigation is imperfect to assume the direction or continuity of fault and/or bedding plane. Moreover, in case of poor core recovery because of encountering loosened, disturbed, and/or sheared portions, correct judgment of the geologic conditions may not be possible. Therefore, it is very important that core drilling should keep utmost technical efforts in order to get full of core recovery.

Advantage of core drilling regarding to its applicability for the project site is as follows:

- Short investigation period and minimal cost to examine underground geologic conditions
- Possible to drill in any direction
- Recovered core enables a variety of laboratory tests
- Drillholes enable a variety of tests including permeability test, varied logging tests, seismic prospecting, ground water test, horizontal load test and standard penetration test.

Cores recovered from drillhole are stored directly from the core tube into a specified core box. They are stored according to the depth described on the core box. Where the core is not recovered or removed from the box for laboratory test, the vacant portion must be filled with a stick of the same length together with a note described the reason clearly. Simultaneously, the recovered cores should be put into core box provided partitions to prevent their mixture.

2) Geologic inspection of drilled core

The results of drilled core observation from the engineering geologic view point are described in the specified drillhole log, as shown in Figure 9-14. The basic items for the drillhole log are described hereinafter.

(a) Geologic name

Described by rock name and specified code. The stratum name is also described if it is evident. The surface deposit is expressed by, for example, river deposit, extremely weathered granite, etc. Its component, gradation and characteristics are clearly described in the remarks column.

(b) Rock color

The color of core saturated with water is described. A saturated core allows clear observation of the component particles and crystals, so the observation of the core should be carried out under wet condition of the core. When its color changes significantly, depending on a wet or dry condition, colors of both conditions are to be observed and noted.

(c) Weathering degree, hardness and joint conditions

Since this item is directly related to the engineering geologic properties of the rocks, the content is described based on the classification of the foundation bedrocks. Regarding joints, especially, its freshness, contamination state, existence of slip surface, and type and conditions of impurities are described. Also, the phenomena regarding solubility (soluble rocks including limestone), and swelling by water (mudstone, etc.) is described.

The core conditions may be expressed by Rock Quality Designation (RQD) which is a method to express the minimum 10cm long core recovery in each unit length.

(d) Drilling conditions and actual records

Actual drilling records with measured data in the drillhole are utilized for the reference data to understand subsurface geologic conditions. Such records/data recorded in daily drilling reports are of drilling speed, supply/drain water rate during drilling, location of cementation, diameters of drilling bit/drillhole, location of casing pipe with its diameter, locations of flowing-out of drilling water and/or flow-in ground water with their flowing rate (l/min), monitoring water level with date, etc. In a case that slimes are only recovered by drilling, this fact should be noted clearly in the drilling daily report, and it be reminded that the drilled section recovered only slimes are excluded from core recovery log of the drillhole.

3) Permeability test and monitoring groundwater level

Permeable characters of subsurface ground are generally shown by Lugeon value and/or Permeability coefficient, and the former is applied for bedrocks developing, more or less with cracks/joints and the latter for unconsolidated deposits and/or soft rocks in which Darcy' law for water flow is applicable, respectively.

A Lugeon value is calculated by results of Lugeon test which injects pressured water against a drillhole section (common length 5m) and by the relations of the pressures and injected water volumes. The Lugeon value is defined as injected water volume per minute (l/min) and per one (1) meter length of test section under injected water pressures of 0.98MPa(10kgf/cm^2).

$$Lu=10Q/PL$$

where,

Lu	: Lugeon value
Q	: Injected water volume per one (1) minute (l/min)
P	: Injecting pressure (kgf / cm^2)
L	: Length of test section (m)

There are two methods for the permeability test for unconsolidated deposits and/or soft rocks, one is Packer method and another is Open-end method. The testing procedures of the Packer method is as quite same as of the Lugeon test. The permeability coefficient of Packer test is calculated by the following formula:

$$k = (Q/2\pi LH) \ln(L/r_0) \qquad : L \geqq 10\, r_0$$

where,

k	: Permeability coefficient (cm/sec)
Q	: Injected water volume into drillhole (cm^3/sec)
L	: Length of test section in drillhole (cm)
H	: Hydraulic head (cm)
r_0	: Radius of drillhole (cm)

During actual drilling works of a drillhole, the temporal water levels in the drillhole are measured everyday. When the drillhole reaches to the planed depth and the drilling works are ended, and then, after waiting one or two days, the stable groundwater level in the drillhole is to be measured. It is well known that the groundwater level is unstable in seasons, especially, in rainy and dry seasons. Accordingly, it is very necessary to conduct annual monitoring of the groundwater levels in the investigation drillholes in some places where the groundwater levels are very important factors for design such as the dam site. Such annual monitoring works are done periodically with manual water level meters or indicators or continuously with telemeter water level indicators.

(5) Laboratory rock test

Laboratory rock tests use rock core samples recovered by drillhole and make clear the physical and mechanical properties of the rock which are useful for evaluation of geotechnical features of the bedrocks. On the F/S stage of the hydroelectric power project the following laboratory tests are commonly conducted:

[Physical tests] : Specific gravity・absorption test, ultra sonic test
[Mechanical test] : Unconfined compression test, Splitting tensile strength test, triaxial compression test (at need)

Of the standards of laboratory rock tests, ASTM (American Society for Testing and Materials), JIS (Japanese Industrial Standard), JGS (Japanese Geotechnical Society) and BS (British Standard) are rather typical.

(6) Exploratory adit

Exploratory adit is the most effective measure for geological survey because it enables direct observation of the underground geology, such in-situ exact conditions as, strike & dip, continuity, sheared width of faults and/or other discontinuities, and to evaluate rock-classification on exposing rocks of adit walls. Although sufficient data is acquired from drilling investigation, the exploratory adit is the most reliable to precisely know the geology where core recovery is low, or geologic factors such as the characteristics and direction of faults and joints. Also, the exploratory adit is used when measuring the seismic wave velocity, or conducting the in-situ rock deformation/shear test. The cross section of the adit for observation and various tests must be approximately 1.8m high and 1.5m wide at the base. The following are the observation items.

- Type, characteristics and distribution of surface deposit
- Type of rock, quality, its distribution, and rock mass classification
- Geologic structure (bedding, schistosity, fault, fracture zone, seam, joint, etc.)
- Weathering and alteration of bed rock
- Rockmass classification/Rockmass evaluation
- Ground water conditions (humidity, spring, and spring locations in the adit are also recorded. Regarding a large quantity of springing water, its temperature, quantity, quality, pressure and spurt change in time sequence are measured as references to check the spring path.)

Although there are several methods to express geological development of exploratory adit, an example is shown in Figure 9-15. It is generally produced in 1:100 scale maps. The name, elevation of the adit entrance, excavation length and direction must be entered in the diagram.

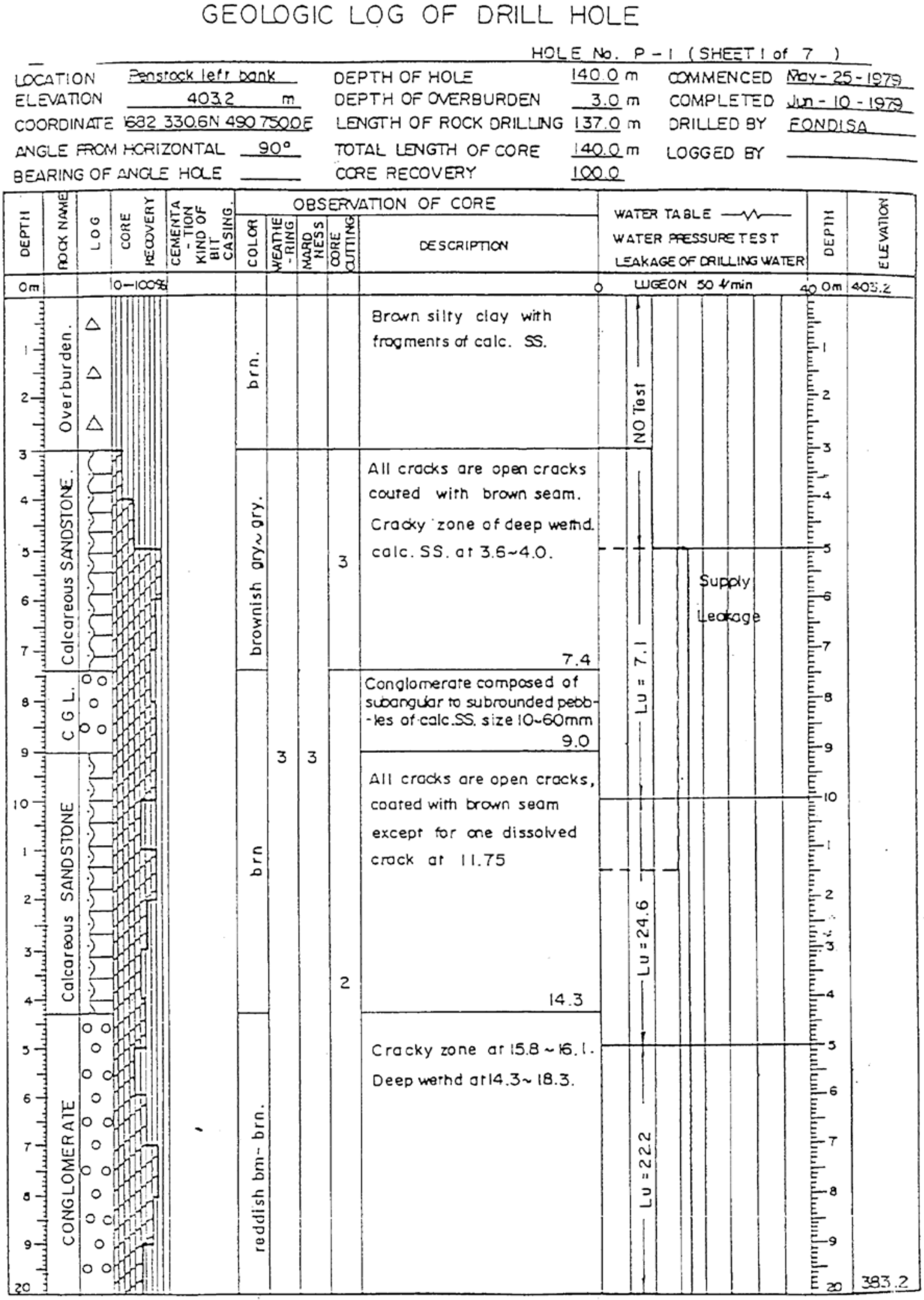


Figure 9-14 Example of Drillhole Log

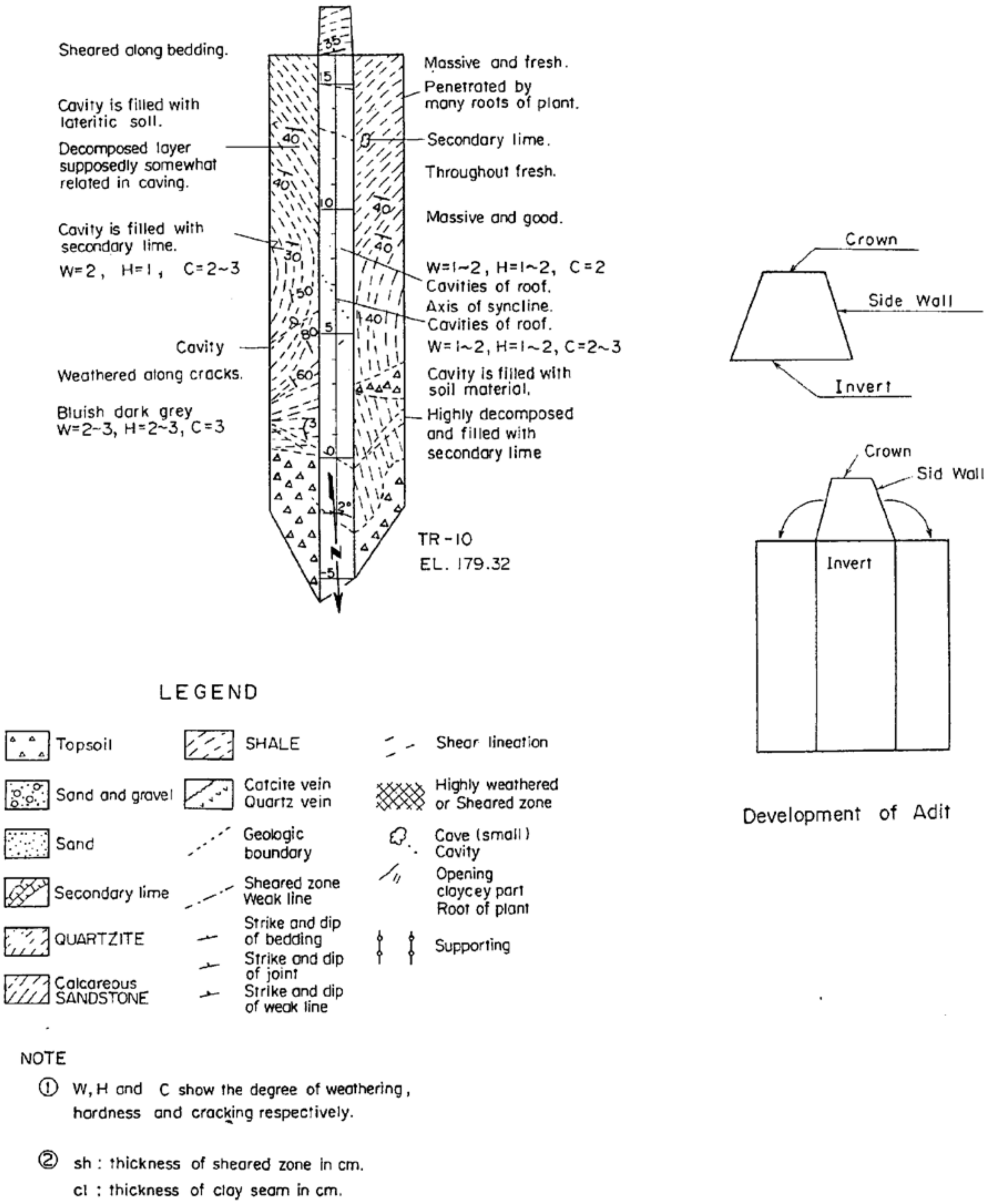


Figure 9-15 Geologic Development of Exploratory Adit

9.2.3 Construction Materials

(1) Dam types and materials

Dams are categorized into concrete dams and fill dams. The former is further divided into gravity dams, which include Roller Compacted Concrete (RCC) and Roller Compacted Dam Concrete (RCD) types of dams, and arc dams. The latter consists of impervious clay core dams, concrete facing and asphalt facing types of dams. Selection of a type of dam for a particular site depends on the construction costs of dam and spillway, which are estimated taking account of the topography and geology of the site, and availability of construction materials from site vicinity.

(2) Surveys of dam body materials

Surveys of dam body materials consist of material quality survey and material abundance survey. In the quality survey, materials available from site vicinity are evaluated for the properties required for dam construction. The survey for abundance estimates the volumes of materials obtainable from nearby borrow areas in order to evaluate sufficiency for dam construction.

1) Material survey for concrete dams

The material survey in this section primarily describes concrete dams and also covers intake weirs, concrete impervious facing of fill dams, and other concrete structures.

Consumption of aggregates for concrete of dam body takes places in a short span t of time and in large volumes. This makes production of aggregates at the project site a standard approach. They are usually obtained from quarries. Riverbed gavels may be used, but it is quite rare that the volumes obtainable from one location are sufficient to meet the need for aggregates.

(a) Survey for quality

Strength and specific density are the two properties required for raw materials for aggregates. Many kinds of rocks can satisfy the requirements, provided they are free of deleterious elements. Sandstones are a preferred selection in many instances because it is known to be problem-free. Metamorphic rocks and clay stones are not usually chosen since the former aggregate tends to be flat and the latter aggregate lacks strength.

Quality requirements for aggregates include the following.

- Sufficiency of hardness and strength, and durability
- Adequacy of gravel diameter and size distributions
- Inclusion of less-than-permissible levels of deleterious materials

These properties and characteristics are verified by specific gravity measurements, and tests which include water absorption test, and Los Angeles and alkali-aggregate-reaction tests.

(b) Survey for quantity of material available from quarry sites

Survey for quantity usually follows the next procedures.

- Carry out grid-by-grid elastic wave exploration of candidate quarries selected based on existing geologic maps and outcrop surveys
- Carry out boring at grid intersections. For positive results, conduct visual checks of rocks from test adit, and prepare samples for abundance survey and confirm the volumes obtainable from quarries

2) Material survey for fill dams

Material survey for quarries for rocks basically follows the procedures described for aggregates for concrete. Materials subject to survey for core include weathered rocks, colluvial soils, mudflow deposits, and loam soils.

(a) Survey for quality

Materials for fill dams are divided into pervious materials (rocks) , impervious materials, and semi-pervious materials (filter materials). Pervious materials provide structural stability. Sand gravels and rocks are chosen since they combine required strength and drainability. Impervious materials provide impermeability for dams to store water, and clayey geologic materials with relatively high contents of coarse grains are selected. For semi-pervious materials, sand and gravels are chosen. They have properties which are in between pervious and impervious materials. Asphalt or concrete is used when impervious materials are not obtainable.

Surveys for quality for pervious and semi-pervious materials include specific gravity measurement, water absorption coefficient test, grain size distribution measurement and uniaxial compression test. Those for impervious materials include the tests such as natural water content, specific gravity and measurements, liquidity limit and inelastic limit, grain size distribution, compaction permeability, and triaxial compression. Abundance survey is done by reconnaissance of ground surfaces and geophysical and boring explorations. Test pits and boring survey are added for impervious materials.

(b) Survey for quantity of material available from quarry sites

The survey basically follows the procedures described for the aggregates for concrete, which include reconnaissance of ground surfaces, geophysical exploration, and boring. For impervious materials, survey by test pits and augur boring are added.

9.2.4 Earthquakes

The objective of seismic design is to design structures which are economical and structurally safe against anticipated earthquake loads at planned site of a project. In the case a developing country in which the project site locates already has seismic load modeling, mode of analysis, and safety standards for seismic design in order, the feasibility study is conducted according to these tools of the country. Conversely when they are non-existent, they need to be prepared in advance for feasibility study. This section considers the latter case and presents how seismic loads are determined.

First earthquake survey is necessary in order to determine the seismic loads for seismic design. Items for the survey and study are shown in Table 9-2. Those from (1) to (3) identify the active faults likely to cause maximum damage at the site, and the item (4) clarifies the characteristics. Based on the proceeding findings and results, earthquake hazard evaluation is done in item (5). If earthquake zone maps, which record intensity of seismic motions for wider areas covering the planned site, are available, they can replaces item (5). Even in that case the items from (1) to (3) are necessary in order to validate the available zone maps.

Table 9-3 Investigation for Evaluation of Design Seismic Load

(1) Review of earthquake tectonics surrounding area of the country
(2) Review of neotectonics
(3) Survey of historical earthquakes and earthquake damage
(4) Survey of faults likely to influence regions centered on planned site
(5) Evaluation of earthquake hazard
(6) Design Seismic Coefficient

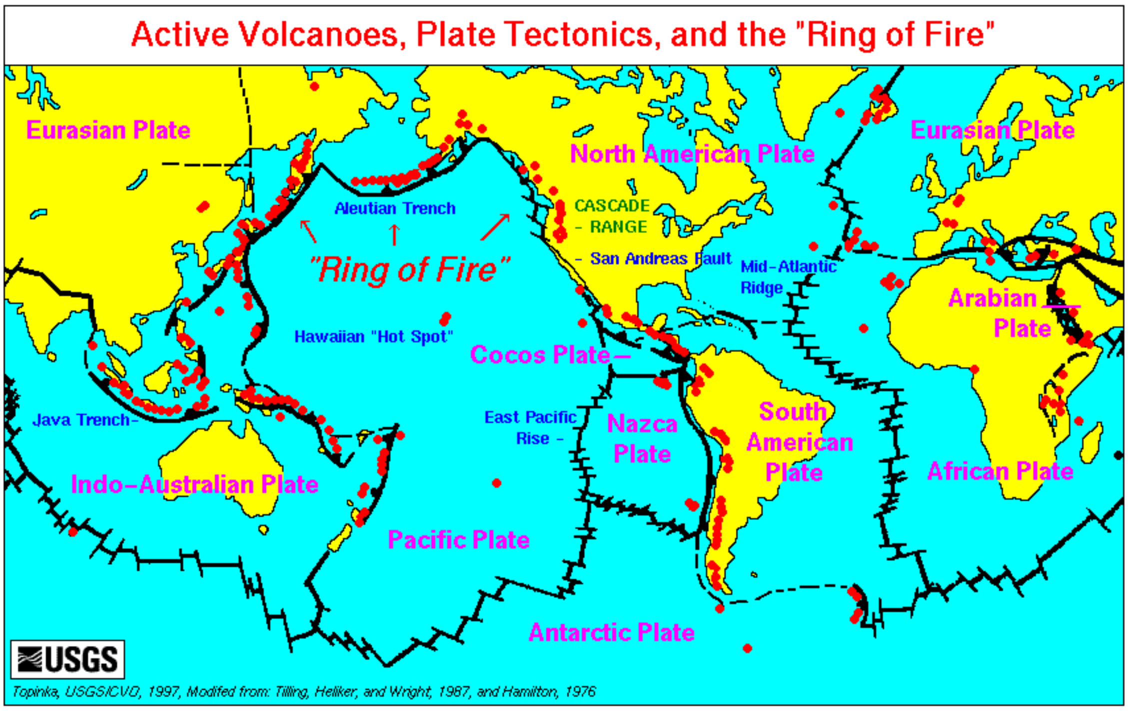
Seismic design considers two kinds of earthquakes. One is an earthquake which is anticipated to occur a few to several times during the life of the power plant. Another is an earthquake that evaluates seismic qualification of equipment in relation to the disaster to the public that could be caused by the earthquake. The former is called Operating Basis Earthquake (OBE), the latter Maximum Credible Earthquake (MCE), at planned site.

During the feasibility study seismic loads are necessary for both OBE and MCE. Seismic loads from the two earthquakes are evaluated at different stages of construction of structures, including immediately after the start of construction, after completion of construction, and when the reservoir is empty. Different safety standards and rules apply to each of these conditions. Dam and other important facilities of hydropower plants are designed to the OBE with 10 percent of probability of exceedance and with a return period of 475 years during 50 years of plant life. MCE needs to have much longer probability of exceedance, and in some instances the period ranges from 5,000 years to 10,000 years.

(1) Earthquake tectonics (Seismotectonics)

Earthquakes are categorized into plate boundary earthquakes, inland earthquakes which occur inside the plates, volcanic earthquakes, and induced earthquakes by artificial causes. Of these earthquakes, the plate boundary and inland earthquakes, which are both caused by earth's structures, are important for seismic design.

First step of earthquake survey starts from collection and review, from global perspective, of plate maps and incidences of earthquakes in and around a nation in interest. This is done using existing research papers and documents. If the nation resides in plate boundary areas or in earthquake-prone regions, the seismic design plays progressively more important roles. The degree and extent of survey should be consistent with the importance of seismic design. (Refer to Figure 9-16).



Source: US Geological Survey

Figure 9-16　Hypocenter Distribution and Plate

(2) Review of Neotectonics

Neotectonics-faults and folds active in the Quaternary period-in areas centered on the planned site, is important in relation to inland earthquakes. Their distributions should be surveyed using existing geological data for the areas. Where a fault or faults are confirmed in the vicinity of planned site, evaluate their seismicity in the Quaternary period and their causal relationship with earthquakes. Topographic maps and aerial photos, when available, should be used for identification of faults.

(3) Survey of Historical Earthquakes and Earthquake Damage

Domestic seismic catalogues and historical records of earthquakes should be collected when earthquake observation is in place in the nation in interest. The data thus obtained should be used in reviewing historical and regional-wise distributions of earthquakes and their magnitudes related to the number of occurrences, and the resultant damage. When such data is not available in the nation, it is effective and efficient to rely on earthquake catalogues and data bases obtainable from the global research institutions shown in Table 9-3.

History of damage can be obtained from literatures and from interviews of residents. In cases where damage was actually caused, the magnitude of earthquake can be estimated from the extent of damage, provided the design earthquake is available for the structure in interest.

Table 9-4 Famous Earthquake Catalogue

➢Seismicity of Earth（Gutenberg & Richter, 1954）	Major earthquakes in the world 1904-1952
➢Regional Catalogue of ISC(International Seismological Center, UK)	Monthly report of earthquake observation and earthquake catalogue in the world
➢Preliminary Determination of Epicenter （USGS, USA）	Earthquake Data Report
➢ Earthquake data file: NOAA: National Oceanic and Atmospheric Administration）	Including update observation http://www.ngdc.noaa.gov/hazard/earthqk.shtml
➢US Geological Survey (USGS)	Including update observation http://earthquake.usgs.gov/

(4) Survey of faults likely to influence regions centered on planned project site

Faults, identified in the preceding survey as likely to influence regions around planned site, require evaluation of their characteristics. They are termed earthquake source faults, and the earthquakes caused by the faults at planned site are termed as an anticipated earthquake by earthquake source faults. Basic characteristics of earthquake source fault depend on its geometrical parameters such as length, width and dip, and on hypo-central distance from planned site location.

Earthquake hazard evaluation by semi-empirical mode of analysis stated in the next section requires a large volumes data on the relationships between earthquake source faults and resultant earthquakes. For a feasibility phase of study, fault data similar to one in interest may be used for evaluation.

(5) Evaluation of earthquake hazard

1) General

Evaluation of earthquake hazard is to estimate, based on the above survey, the magnitudes and characteristics of earthquakes at the planned project site. An earthquake characteristics observed at the site is considered to be dependent on earthquake source fault characteristics (epicenter characteristics), changes that the source earthquake undergoes during propagation from its origin to the site (propagation characteristics), and the amplification or attenuation characteristics (ground characteristics) at the site.

In many cases there is only a limited earthquake data, and this does not allow appropriate earthquake hazard evaluation. To provide for such situations, there is another method that does not rely on earthquake characteristics but regard earthquakes as statistical events and determines the magnitudes of earthquake at planned site by statistical processing of past earthquake data. The following section provides an outline of the method.

The anticipated earthquakes are divided into plate boundary and inland earthquakes. Errors in the estimation of earthquake characteristics by statistical method are considered to be dependent on the return period of these two types of earthquakes. This necessitates the need to consider inherent bias such errors could cause in the result of evaluation.

(a) Anticipated earthquakes are caused by plate boundary earthquakes

A reasonable amount of information is usually available since earthquakes are believed to occur at a return period from 100 to 200 years, judging from the relative velocities at plate boundaries. Sufficient earthquake data thus derived allows evaluation of earthquake hazard at a reasonable level of accuracy.

(b) Anticipated earthquakes are caused by inland earthquakes

Evaluation of earthquake hazard based on historical earthquake data is not always appropriate because fairly long return periods precludes collection of any adequate amount of data and information. Evaluation of earthquake hazard in a situation like this sometimes depends on peak acceleration. The acceleration on the surface of the good quality bedrocks of dams in Japan has been estimated to range between 200 and 250 gals. It is based on the earthquake data collected from dams near the epicenters of recent strong earthquakes. The peak acceleration can also be estimated from attenuation relationships determined from seismic ground motions monitored at bedrocks similar to those of the planned site.

2) Earthquake hazard analysis by probabilistic method

Probabilistic method of evaluation relies on historical earthquake data and estimates the magnitudes of an earthquake at planned site according to a probabilistic approach. Application of the method includes an analytical mean that calculates statistically expected values and an application of the third asymptotic distribution of the extreme value theory (Gumbel, 1958). Results obtainable include peak acceleration and peak velocity at the site.

The radial distance and duration for analysis are determined considering the seismicity and the records of historical earthquakes. In earthquake-prone countries, a distance of 500km and a duration of 200 years can be regarded a standard option. Since the accuracy of estimation depends on the number of earthquakes, use of the seismic date in Table 9-4 is preferable, in addition to the data available from the record of earthquake observation on and in regions along the pathways of earthquakes.

3) Seismic risk analysis by deterministic method (Empirical method)

An empirical method relies on the use of attenuation relationship for evaluation. It determines, based on historical earthquake data, the relationships between the magnitudes of an earthquake at its epicenter and the distances of its pathway to the location for evaluation. Hydropower plants are placed on firm bedrocks, and if the earthquake data for evaluation is obtained on the bedrocks of comparable characteristics as those for the site in consideration, the attenuation relationship can be interpreted as having already factored in the bedrock characteristics both of epicenter and site location.

The following is an example of attenuation relationship[5,6].and is obtained based on the earthquake records at dam site and on the bedrocks whose elastic wave velocity, Vs, ranges from 700 to 1,500m/sec. The equation determines response spectrums of earthquake on the bedrocks, and the peak acceleration is obtained as short-period components in the analysis. In application coefficients C (T) are necessary.

$$\log S_A(T) = C_m(T) \times M + C_h(T) \times H_c - C_d(T) \times X_{eq} - \log X_{eq} + C_o(T)$$

where,

T = natural period

S_A (T) = average response spectrums of two horizontal components

M = magnitude of earthquake at fault (Magnitude by Meteorological Agency: Mj)

H_C = depth from fault center to ground surface (100km for a case greater than 100km)

Xeq = equivalent hypocentral distance

Cm (T), Ch (T), Cd (T), Co (T) = regression coefficient determined on type of earthquake and on great number of historical earthquake records

4) Seismic risk analysis by deterministic method (Semi empirical method)

Semi empirical method first evaluates seismic motions at earthquake source fault, taking into account its rupture processes, and then evaluates seismic motions at planned site factoring in both the propagation and site bedrock characteristics. Figure 9-17 shows a concept of the earthquake motions at epicenter, where a rupture starts at a point (★).

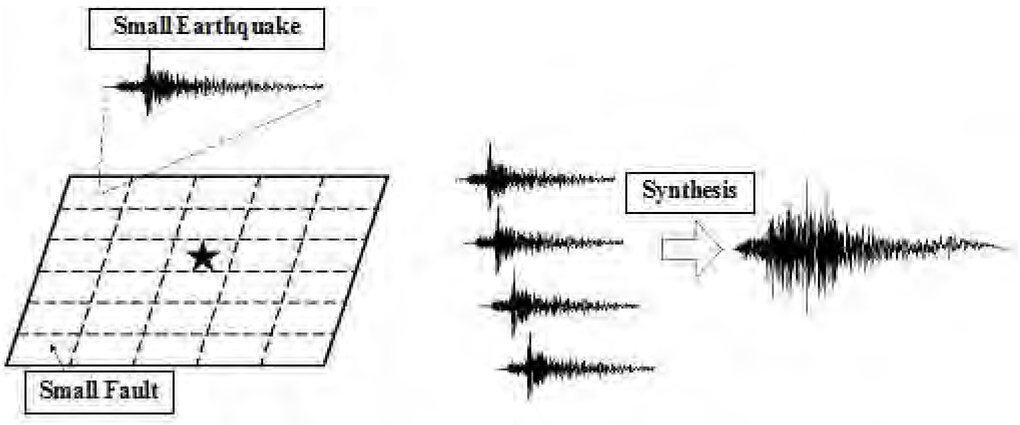


Figure 9-17 Concept of Processes of Initiation of Seismic Motions at Earthquake Source

5 S. Mitsuishi, K. Shimamoto: Seismic Performance Evaluation of Dams against Large Earthquake (in Japanese), Engineering for Dams, No.274, 2009.7

6 http://www.nilim.go.jp/lab/fdg/12/12/html

(6) Design Seismic Coefficient

Design of structures by design seismic coefficient requires determination of seismic coefficient, which is design seismic load, at planned project site.

A peak acceleration at a planned site is determined first from earthquake zone maps of the nation of the site or from the above seismic risk analyses. The peak acceleration thus obtained is used to determine the seismic coefficients applicable for the site. Seismic coefficients, if recorded in the maps, can be used for design.

The relationships between peak acceleration and seismic coefficient are obtainable from several research papers[5]. The following equation is known to apply to the relationships.

$$k_h = R \times (A_{max} / 980)$$

where,

k_h = design horizontal seismic coefficient
R = conversion constant (≈ 0.4 to 0.6)
A_{max} = peak acceleration (gal)

The conversion constant, R, depends upon the hardness of ground and period. The constant is small for hard bedrocks and for earthquakes with seismic motions whose periods are dominant toward shorter side. A value of 0.4 could be used in such application. Conversely, for soft ground and when longer periods are dominant, the constant is larger and a value of 0.6 should be applied.

9.3 Hydrological Study

9.3.1 Study Items and Purposes

The hydrological data required for the feasibility study and its purposes are described hereinafter.

(1) Runoff data

1) Daily flow and monthly flow

The data is used to calculate the output and energy generation. Runoff data is the one of the most important data for the calculation.

2) Flood flow and hydrograph

The data is used to calculate the design flood discharge by using river flow and hydrograph flowing into a reservoir. And the data is used for flood routing considering storage effect in a reservoir.

(2) Meteorological data

1) Rainfall (amount of rainfall and snowfall)

In the case runoff data is not long enough for hydropower planning, various runoff models should be made in order to construct runoff data for the necessary period. The data is also used for flood estimate, construction planning, etc.

2) Atmospheric temperature, humidity, wind velocity, and wind direction

The data is used for PMF calculation, construction planning, etc.

3) Evaporation data

The flow data regulated in a reservoir or a pond is used to calculate generated energy. On the other hand, projects located in arid region or semi-arid region, available flow data for hydropower plant is affected by the evaporation from reservoir.

(3) Sediment yield data

The data is used to set high water level and low water level for planning of a reservoir. Sediment volume is necessary to determine the sedimentation level in the reservoir. Also the data is used for design for settling basins or sand flash gates.

(4) Others

The data on roughness coefficient of river is used to study the backwater effect to the upstream after the dam completion, and to study of tailwater level for power generation. The data on vegetation is used to study the runoff conditions in the river basin, evapotranspiration, roughness coefficient etc.

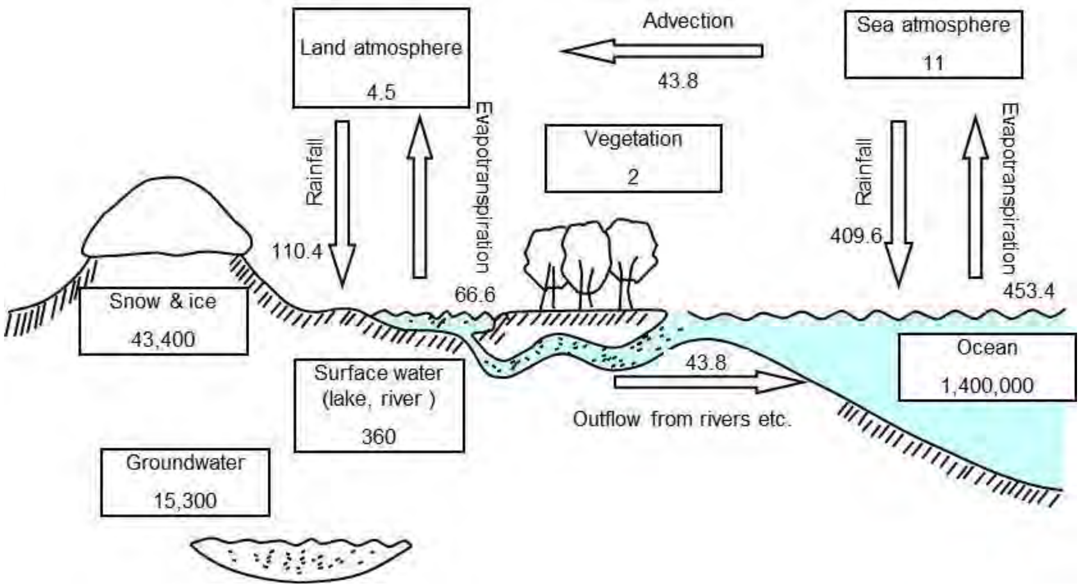
9.3.2 Hydrological Investigation Method

(1) Water cycle

Figure 9-18 shows a schematic of Water Cycle which consists of rainfall, outflow from rivers, outflow from ground water, evaporation from oceans and lakes, transpiration from vegetation. The raindrops fall from clouds (atmosphere) and reach to the earth, and then the water returns to the atmosphere under the following process.

- Certain portion of the water flows on the earth as rivers, and then flows into the ocean.
- Certain portion of the water forms free water surface on the earth and evaporates, and is absorbed from root system of vegetation and returns to atmosphere by transpiration from stoma of plant's leaves. These processes are called evapotranspiration.
- Certain portion of the water penetrates into the earth from the earth surface, and outflows to the earth surface.
- A phenomenon of rainfall and evaporation occurs on the ocean, a part of the evaporation is transferred on earth.

Numerical value in Figure 9-18 indicates water volume, and phenomena of evaporation and rainfall from the ocean stand out.



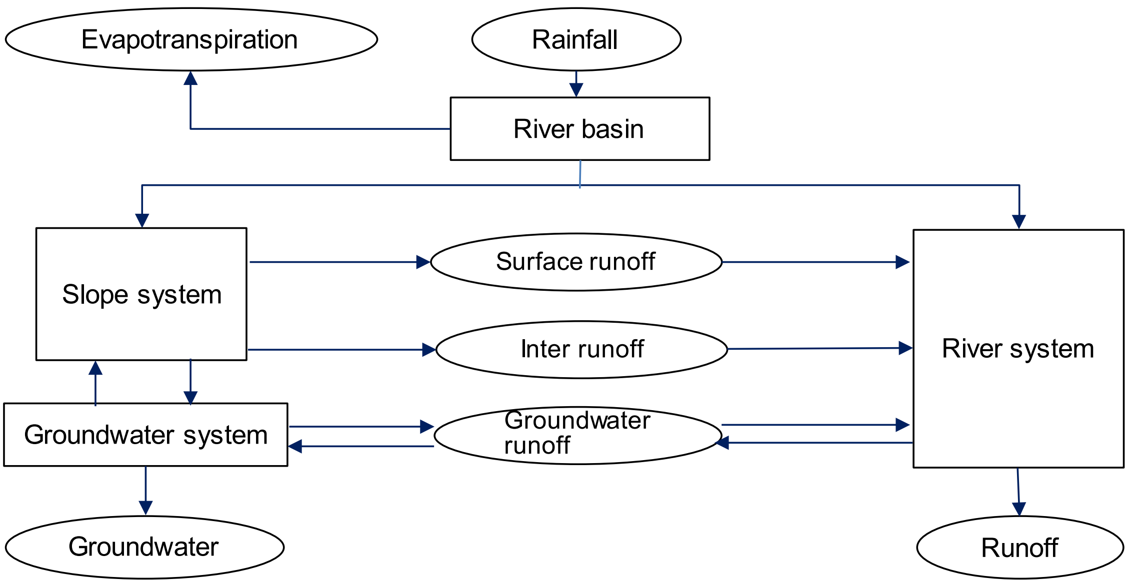
Note: numerical value : 10^{12} m³

Source: Takeda, Ueda, Yasuda, Fujiyoshi, 1992

Figure 9-18 Water Cycle

The rain water falling over a basin appears in a river after following outflow forms which merge and become river runoff. The runoff system expressing this phenomenon is shown in Figure 9-19.

- Water flow on the earth surface is called surface flow, and an outflow to river system is called "surface runoff".
- Flow penetrating to the earth and flowing along the slope is called "inter runoff".
- Flow penetrating to the earth and becomes ground water, and finally comes out to a river. It is called "ground water runoff".



Source: T.Takasao, S.Kanamaru: Hydrology, 1975

Figure 9-19 Runoff System

(2) Rainfall

1) Measurement of rainfall

Rainfall is measured with a non-recording rain gage or recording rain gage including robot rain gage and radar rain gage system. Non-recording rain gages measure the rainfall of one day. Recording rain gages radar rain gage systems are suitable for river basin management which requires to measure rainfall intensity (rainfall per hour) and the rainfall duration. However since the radar rain gage system might not be appropriate facility for developing countries due to its cost, it is not explained precisely in this Manual.

(a) Recording rain gage

Recording rain gage enable automatic measurement of the rainfall for shorter durations together with the rainfall intensity. The measured rainfall is also automatically recorded onto a paper chart.

The recording rain gages are generally a tipping-bucket type and storage type. Also, by different categorization, there are robot rain gages that feature automatic measurement and transmission. A robot rain gage provides accurate, long term measurement in mountainous areas where constant attended observation is difficult. Furthermore, since it provides rainfall data which changes momentarily, this gage is suitable for river basin management.

An example of tipping-bucket type is shown in Figure 9-20 Rain water and snow enter a receiver, and then is received by funnel and enter a tipping-bucket. The tipping-bucket is designed to be as seesaw so that when the water of 0.5mm stores in the bucket, it falls to the opposite side and the water is discharged. Rainfall depth can be measured by counting the number of fallings.

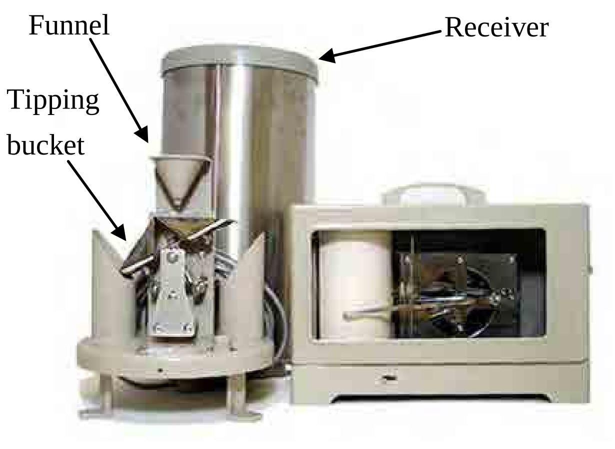


Figure 9-20 Example of Tipping-Bucket Type

(b) Non-recording rain gage

Measurement with a non-recording rain gage is conducted daily at a scheduled time. When the measurement is scheduled at 09:00 a.m., the rainfall measured at 09:00 a.m. indicates the rainfall between 09:00 a.m. of the previous day to 09:00 a.m. on the day. When measuring rainfall at specified intervals, for instance, rainfall may be measured every 6 hours, 3 hours, or 1 hour with a non-recording rain gage.

(c) Installation of rain gage

The best location for a rain gage requires the following conditions; easy access for observers, and flat topography where the wind blows parallel to the ground. The locations such as mountain and hill tops, hillsides, cliff edges, and where sheltered by objects or trees must be avoided.

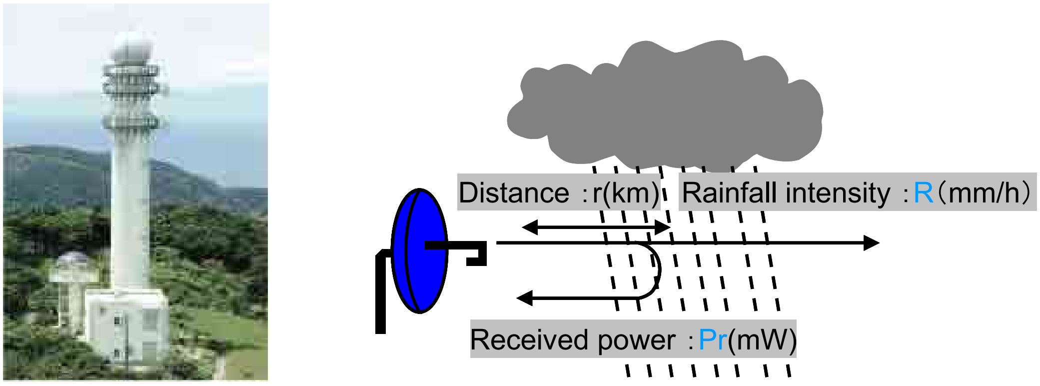
Regarding installation density, it is recommended that the measurement area be divided into several sub-areas, each of which indicates an almost uniform rainfall. A rain gage should then be installed in each of these sub-areas. Where it is difficult to divide the total area into sub-areas with an uniform rainfall, it is recommended that rain gage be installed every 20 to 50km^2.

Figure 9-21 Example of Rain Gage Installation

(d) Radar rain gage system

Radio wave has the characteristics to go straight and carom when it runs into obstacles. Radar rain gage system emits pulsed radio wave having directivity from turning antenna. When the radio wave hits raindrops and returns back, the system receives it by the antenna and measures the rainfall intensity and distribution as follows.

- Distance is measured from the travel time
- Azimuth direction is measured from an antenna direction.
- Rainfall intensity is measured from strength of returning radio wave



Source: MLIT of Government of Japan

Figure 9-22 Rader Rain Gage Site and Concept of Radar Rain Gage System

(3) River flow

River flow is classified to that of a normal condition and a flood condition. The latter is called flood flow (discharge). The former flow is mainly used for hydropower planning and plant operation, and the latter is used for design of dams etc. In this Manual, the term of "river flow/flow" means the runoff of normal condition and the term of "flood flow "is used for analysis under flood condition.

1) Flow observation at gauging station

(a) Measurement method of river flow

In order to measure river flow, flow velocity measurement and survey of river section are necessary. However, the work cannot be done for 24 hours every day in a year because of enormous work volume and cost. Therefore following method is used.

i) A rating curve as described later in Figure 9-28 is prepared by measuring actual flow discharge and the water level at the same time. Frequent observations of the flow are better although it is costly. In Japan, the measurement frequency is 3 times a month (minimum) in principle.

ii) The water level should be observed daily or hourly or continuously although velocity measurement is not done.

iii) The flow when the measurement is not conducted is obtained by using the rating curve and the water level mentioned above. The detail is explained below.

(b) Location of flow gauging station

The following conditions are considered in selecting a flow gauging station (hereinafter, gauging station). An example of the gauging station is shown in Figure 9-23.

➢ River direction should be almost straight, so curved part and sharp change in cross section area should be avoided.

- River flow should not be excessively fast or slow.
- Stream flow direction and river bed are less changeable.
- Flow velocity and water level should be measurable regardless of the volume of flow.

Figure 9-23 Flow Gauging Station

(c) Survey of river cross section at gauging station

The river cross section is surveyed and drawn at the gauging station site. When the cross section changes due to flooding or other reasons, it is necessary to conduct another survey to correct the drawing.

(d) Flow velocity measurement with current meter

a) Propeller current meter

Figure 9-24 (left) shows the current meter in which water pressure leads rotating body. Generally this type is used for rivers having flow with certain water depth shown in Figure 9-25.

b) Electromagnetic current meter

Figure 9-24 (right) shows the current meter which does not have rotating parts. This type is especially useful for small tributaries where Propeller current meter is not applicable but to shallow water depth.

Figure 9-24 Propeller Current Meter and Electromagnetic Current Meter

c) Measurement of Velocity

When the velocity is measured at many points in the vertical line as shown in Figure 9-25, the mean velocity on the measurement line should be calculated by trapezoid formula for all points against the depth (See Figure 9-25 (right)).

It is, however, calculated in the following simplified method in case the flow measurement faces difficulty due to certain river conditions. The two point method has higher priority to one point method and surface method.

Surface method

Measure the velocity V_s at 5cm below the surface. The mean velocity V_m is obtained by multiply V_s by 0.8.

$$V_m \text{ (m/sec)} = 0.8V_s$$

One point method

Measure the velocity at 0.6d (d: water depth below the surface) as shown in Figure 9-25. The result is taken as the mean velocity.

$$V_m = V_{0.6}$$

Two point method

Measure the velocity at 0.2d and 0.8d below the water surface.

Their average value is taken as the mean velocity.

$$V_m = \frac{V_{0.2} + V_{0.8}}{2}$$

Three point method

Add a doubled value of the velocity at 0.6d to the velocity at 0.2d and 0.8d below the water surface and divide by 4.

This is regarded as the mean velocity.

$$V_m = \frac{V_{0.2} + 2V_{0.6} + V_{0.8}}{4}$$

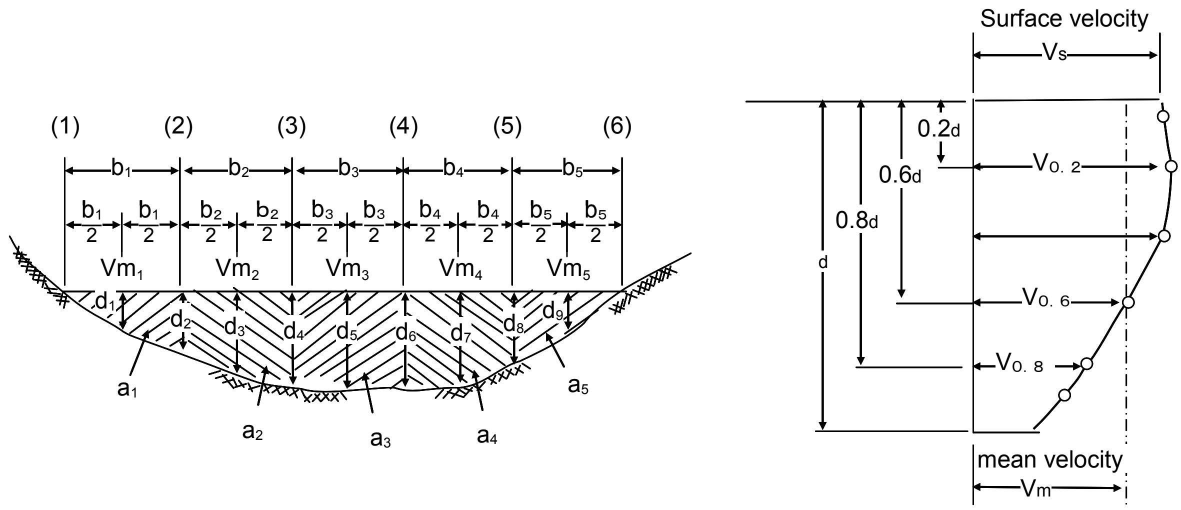
When the mean velocity Vm (m/sec) is acquired for each line from the above methods, the river flow is calculated by the following equation and shown in Figure 9-25.

$$Q(\text{m}^3/\text{sec}) = a_1 \bullet V_{m1} + a_2 \bullet V_{m2} + a_3 \bullet V_{m3} + a_4 \bullet V_{m4} + a_5 \bullet V_{m5}$$

where,

a_i : average area for each section (m^2), See Figure 9-25

Vm_1 : the mean value of velocities of section (1) and (2)



Note: $b_1 = b_2 = b_3 = b_4 = b_5$

Figure 9-25 River Cross Section and Vertical Flow Velocity Curve

(d) Rating curve

By plotting the water level on the ordinate and the flow on the abscissa, a rating curve is acquired by the least square method as shown in Figure 9-26 The rating curve is generally expressed by the following quadratic parabola equation;

$$Q=a+bh+ch^2$$

where,

Q	: flow (m^3/sec)
h	: water level (m)
a, b, c	: constant

When the relation between the water level and flow cannot be expressed in one equation, it is recommended that it be expressed in several equations according to the water level. An example of a rating curve is shown in Figure 9-26.

It is required to prepare a new rating curve when the relation between the water level and the flow changes due to river bed change by flood events occur.

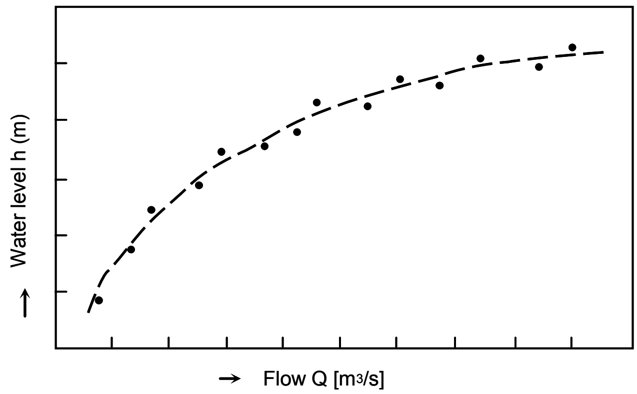


Figure 9-26 Rating Curve

(e) Water level measurement

There are several methods to measure the water level as follows.

a) Method by using staff gage

A staff gage should be installed to measure the water level from the lowest to the highest levels. When one staff gage is not enough to measure, more than two staff gages are used. A minimum overlap of 50cm with the next staff gage is desirable as shown in Figure 9-27.

b) Method by using float

When a float is used with a water stage recorder, a stilling well is installed to prevent water level change due to waves. The vertical motion of the float placed in the well is converted to pulley revolution by a wire, and the revolution is reduced and recorded as shown in Figure 9-28.

c) Method by measuring pressure

Two methods are available to measure the pressure such as installing a pressure sensor in the water, emitting gas into the water and measuring the pressure required to emit the gas as shown in Figure 9-29.

d) Others such as employing ultrasonic wave

Figure 9-27 Staff Gage Tape

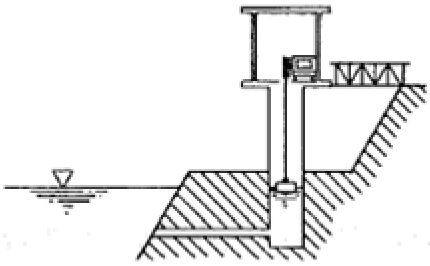


Figure 9-28 Float Type

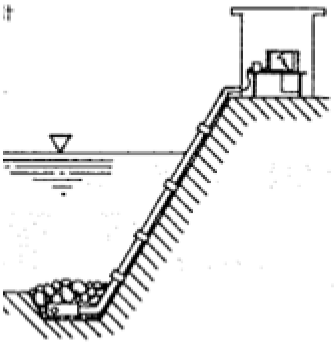


Figure 9-29 Pressure Type

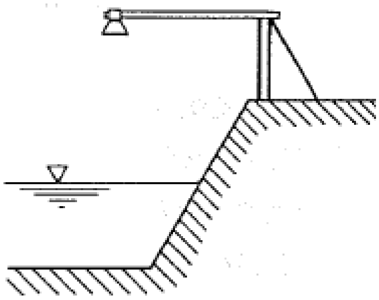


Figure 9-30 Ultrasonic Wave Type

2) Flow measurement with weir

When measurement with a current meter is not practicable due to small flow of a small river, a weir is installed as shown in Figure 9-31 to obtain the flow by measuring the overflow depth. The weir is installed at right angles to the stream center line. A stable location with a slow current is selected for its upstream. There are many methods to calculate the flow, but in this Manual the following equation is used.

Range of application is $0.5m \le B \le 6.3m$, $0.15m \le b \le 5m.$, $0.15m \le D \le 3.5m$, $bD/B^2 \ge 0.06m.$ and $0.03m \le h \le 0.45\sqrt{b}$ m.

$$Q = Cbh^{3/2}$$

$$C = 1.785 + \frac{0.00295}{h} + 0.237\frac{h}{D} - 0.428\sqrt{\frac{(B-b)\,h}{BD}} + 0.034\sqrt{\frac{B}{D}}$$

where,

Q	:	flow (m^3/sec)
h	:	overflow depth (m)
B	:	weir width (m)
b	:	weir opening width (m)
D	:	height from the channel bottom to the weir edge (m)
C	:	flow coefficient

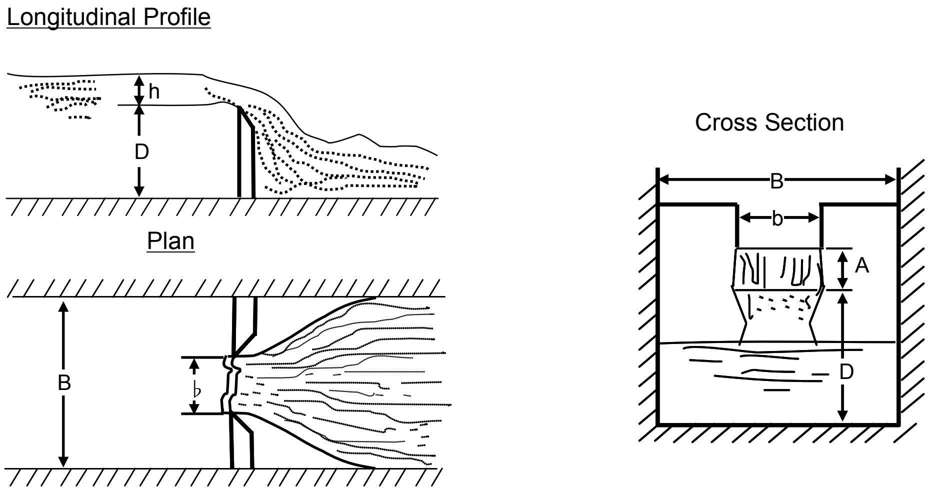


Figure 9-31 Rectangular Weir

A fixed weir with a complete overflow is desirable for the flow measurement be selected. If the weir gives an incomplete overflow or submerged overflow, both the upstream and downstream water levels must be measured.

3) Measurement of flood flow

Flow measurement method during flood is as follows.

(a) Flow measurement with float

Floats are used when measurement with current meter is not possible during flood. There are two types, the surface float and bar float, as shown in Figure 9-32.

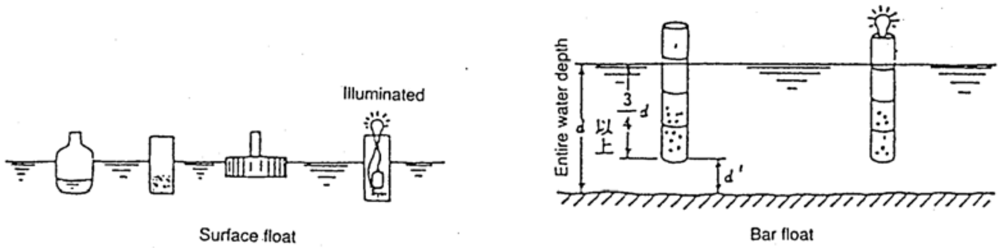


Figure 9-32 Surface float and Bar float

The measurement location should be at a point where the river cross section is even and on a straight line. The floats travel distance should be not less than 30m. Ideally, the distance should exceed the river width. For the float travel distance, The lines of AA' to CC' are determined such that the lines are at right angles to the center of river flow as shown in Figure 9-33.The following procedure is used.

- Observers are deployed at each point.
- A float is dropped approximately 10 to 20 m upstream of the upper line (AA).
- By using a stop watch, travel time of the float over distance L (m) between AA' and CC'

is measured.

- This measurement is repeated several times and acquires the mean travel time (t) in second.

The mean velocity V_m (m/sec) is acquired by the following method;

With a surface float: $V_m = 0.8\dfrac{L}{t}$

With a bar float: $V_m = \left(1.012 - 0.116\sqrt{\dfrac{d'}{d}}\right)\dfrac{L}{t}$

where,

V_m	: Average velocity (m/sec)
L	: Distance between AA' line and CC' line (m)
t	: Travel time between AA' line and CC' line (s)
d	: Total water depth (m)
d'	: Depth from the bottom end of bar float to river bottom (m)

After acquiring the flow velocity of each part (right, center and left) with the float, then the flow of each part is added to acquire the total flow as same as the method explained in item (e).The average value of area for AA', BB' and CC' traverse line is applied as the area of stream cross section. A piece of wood or an empty bottle can be used as a surface float. However, caution is required since these objects are easily affected by wind direction and velocity. When using a bar float, the length submerged should be not less than 3/4 of the entire water depth as shown in Figure 9-32.

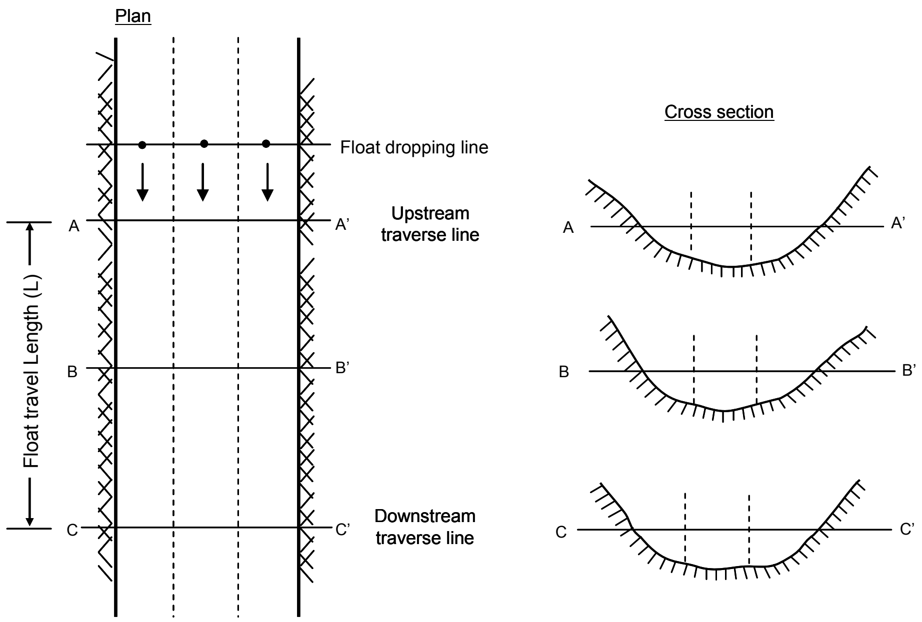


Figure 9-33 Float Measurement

(b) Flow calculation with Manning's formula

When measurement with a current meter or float is impractical during flood season, the mean velocity is calculated by Manning's Formula. It is desirable to select a location where the river cross section is even, river flow center line is straight, and riverbed gradient is uniform.

The mean velocity is acquired by the following Manning's Formula;

$$Q = A \cdot V_m \ (m^3/sec)$$

$$V_m = \frac{1}{n} \cdot R^{2/3} \cdot I^{1/2} \ (m/sec)$$

where,

Q	: flow (m^3/sec)
A	: average stream cross section (m^2)
V_m	: mean velocity (m/sec)
n	: roughness coefficient (See Table 9-4)
R	: hydraulic mean depth (stream cross sectional area/wetted perimeter length) (m)
I	: gradient of water surface

Table 9-5 Roughness Coefficient(n) for Natural Rivers

Wetted perimeter conditions	Coverage of n	Average value of n
Regular alignment and cross section with large water depth	0.025-0.033	0.030
Slightly meandered with curb and shoal	0.033-0.045	0.040
Slightly meandered with shallow water	0.040-0.055	0.050
Slightly meandered with gravel bed and shallow water	0.045-0.060	0.055
Significantly meandered with varied curb/shoal and many water plant	0.050-0.080	0.070
Same as above with slow current due to deep water plant	0.075-0.150	0.080
Shallows with rapid velocity	0.060-0.080	0.070

(4) Evaporation

Evaporation is generally measured with an evaporation pan. The type of evaporation pan depends on countries. Class A pan in USA, Symon's pan in Europe and India are the examples. There are three types of evaporation pan depending on its installation method and location due to its economical and maintenance factors.

Surface type (placed on the ground)
Sunken type (buried in the ground)
Floating type (floats on water)

The specifications of a Class A Pan, which is the surface type and the most popular, are as follows.

Diameter 122 cm (4 feet)
Pan depth 25.4cm (10inches)

Water depth 20 cm (8 inches) - 18cm (7 inches)

The water surface level is measured daily. And the evaporation is computed as the difference between observed levels, by adjusting precipitation measured for the day in a rain gage.

Source: Wikipedia

Figure 9-34 Class A Evaporation Pan

(5) Suspended sediment

Sediment flowing into a reservoir consists of suspended load and bed load. The suspended load is relatively easily measured but the bed load is difficult to measure. A sampler is lowered from river surface to river bottom to continuously collect suspended load in the water. A rod with a built-in bottle is attached to the sampler. The measured suspended load is usually expressed in unit of p.p.m. In addition to the measurement of sediment, gradation and heavy metal content should be analyzed together with chemical analysis as these are useful in later stages when searching the origin of the suspended sediment.

9.3.3 Runoff Analysis for Hydropower Planning

(1) Flow data

1) Required length of period of runoff data

The required period for the economic analysis of a hydropower planning is generally 50 years which is the service life of a hydropower plant. It is recommended, therefore, to acquire runoff data which covers as long a period as possible since this is the basic material for the plant output and electric energy calculation. Taking into account the expression of average runoff and economic service life of hydropower, the period of at least 10 years is necessary and that of about 30 years is recommendable if possible as described in Appendix A-9-1 .

2) Type of runoff data

Daily runoff data is used for pondage type and run-of-river type. Monthly runoff data is used for reservoir type in principle.

(2) Calculation of river flow at the dam site

When the runoff data of the dam site does not cover a long period, the flow at the dam site is calculated by the following methods. Which method

- Conversion by the catchment area ratio
- Conversion by the catchment area ratio considering the weight of the rainfall
- Use of the correlation between the gauging stations
- Simulation with a mathematical model such as tank model, etc
- Correlation between the flow and rainfall
- Other method

Which of the above methods or combination thereof are adopted depend on the kind and period of the existing hydrological data which is used for the flow calculation.

(3) Conversion by the catchment area ratio

When a gauging station is located near the dam site and the meteorological conditions at the gauging station and watershed of the dam site are the same, the flow at the dam site is calculated by conversion of the catchment area ratio. The details are described in 5.3.2 (2). The following equation is also applied when the flow at the dam site is calculated from the flow at two stream flow gaging stations as shown in Figure 9-35 Whether the rainfall conditions are same or not is judged by using monthly or yearly rainfall depth.

$$Qd = \left(\frac{Qg_1 \times AB_1}{Ag_1} + \frac{Qg_2 \times AB_2}{Ag_2} \right) \times \frac{Ad}{AB_1 + AB_2}$$

where,

Qd	: flow at dam site (m^3/sec)
Qg_1	: flow at No. 1 gauging station (m^3/sec)
Qg_2	: flow at No. 2 gauging station m^3/sec)
Ad	: Catchment area of dam site (km^2)
Ag_1	: Catchment area of No. 1 gauging station (km^2)
Ag_2	: Catchment area of No. 2 gauging station (km^2)
AB_1	: Catchment area of No. 1 tributary (km^2)
AB_2	: Catchment area of No. 2 tributary (km^2)

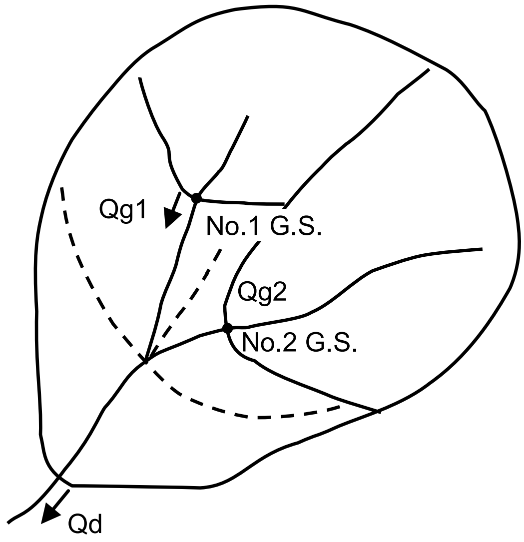


Figure 9-35 Catchment Area of River basin

(4) Conversion by the catchment area ratio considering the weight of the rainfall

1) Average rainfall of the river basin

Rainfall depth at an observatory station means the point rainfall value. However it is necessary for hydropower business to grasp the rainfall data for entire basin of the project because of runoff analysis and flood analysis. Thiessen method, isohyetal method etc. are useful to estimate the rainfall.

Here, Thiessen method is explained. As shown in Figure 9-36 this method is to estimate rainfall depth over entire basin by weighted average of all observatory stations concerned. The method's procedure is as follows.

- Dividing the basin by Thiessen method for all observatory stations
- Perpendicular bisector for connecting line of adjacent two observations is drawn, and territorial area of each station is determined.
- Determining the area of each station and measuring the area
- Average rainfall of the basin is calculated from areas and rainfall depth by weighted average

$$R = \frac{a_1 r_1 + a_2 r_2 + \ldots\ldots a_N r_N}{A}$$

where,

R	: Average rainfall depth (mm)
$a_1, a_2, .. a_N$	: Area of each polygon such as A, B, C station (km^2)
A	: Total area ($A = a_1 + a_2 + \ldots\ldots a_N$) (km^2)
N	: Number of observatory stations
$r_1, r_2, \ldots, r_N$	: Rainfall depth of each observatory station (mm)

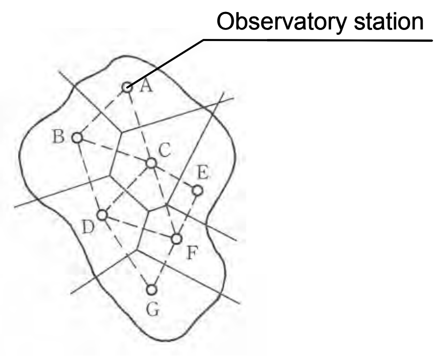


Figure 9-36 Thiessen Method

2) Flow at the dam site

When the dam site and stream gauging station are located separately, and the meteorological conditions (rainfall) at the two sites are different, the flow at the dam site is calculated by conversion of the catchment area ratio considering the weight of rainfall. The average rainfall at the river basin is acquired by either the isohyetal method or Thiessen method. Flow Qd at the dam site is expressed by the following equation;

$$Qd = Qg \times \frac{Rd \times Ad}{Rg \times Ag}$$

where,

Qd	: flow at dam site (m^3/sec)
Qg	: flow at gauging station (m^3/sec)
Rd	: average rainfall at dam site (mm)
Rg	: average rainfall at gauging station (mm)
Ad	: Catchment area of dam site (km^2)
Ag	: Catchment area of gauging station (km^2)

(5) Correlation of gauging stations

In the case the runoff data at the dam site is not long enough, but there are other gauging station nearby having data of long enough, the data of the dam site can be extended for long period.

Relation between flow data (X) at adjacent gauging station and flow data (Y) at dam site can be analyzed for the same period by using regression analysis. Assuming the linear expression of y=a+bx for two kinds of variables (xi, yi), a and b is obtained by the least-square method as shown bellow and Figure 9-37.

Y=a+bX

$$b=\frac{n\Sigma x_i y_i - \Sigma x_i \cdot \Sigma y_i}{n\Sigma x_i^2 - (\Sigma x_i)^2}$$

$$A=\overline{y} - b\,\overline{X}$$

where,

$$\overline{X} = \frac{1}{n}\Sigma X_i,\quad \overline{y} = \frac{1}{n}\Sigma y_i$$

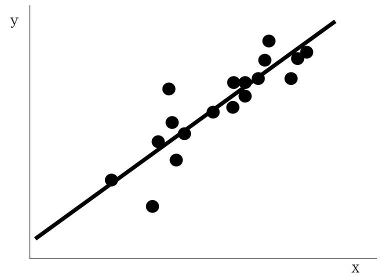


Figure 9-37 Example of Linear Expression

By using this formula, the flow data at dam site can be calculated. In order to check how this regression formula fits in data, correlation coefficient (r) calculated by formula below is examined. If the value of R is more than 0.7, it can be empirically said that two sets of data have good correlation.

$$R=\frac{\sum_{i=1}^{n}(x_i-\overline{x})(y_i-\overline{y})}{\sqrt{\sum_{i=1}^{n}(x_i-\overline{x})^2 \sum_{i=1}^{n}(y_i-\overline{y})^2}}$$

where,

$$\overline{X} = \frac{1}{n}\sum_{i=1}^{n} X_i,\quad \overline{y} = \frac{1}{n}\sum_{i=1}^{n} y_i$$

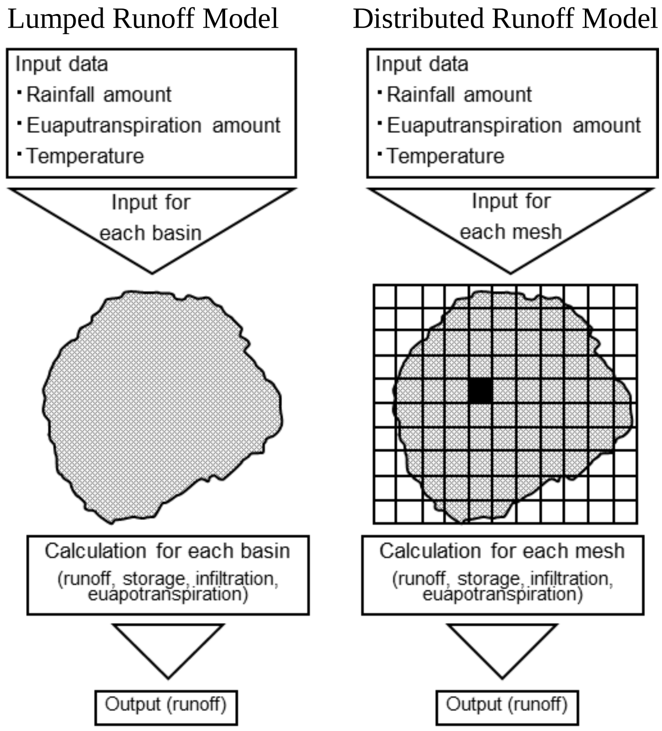
(6) Runoff model based on relation between rainfall and runoff

1) Lumped model and distributed hydrological model

Runoff model is classified to lumped model and distributed hydrological model (hereinafter, distributed model).

The lumped model is the model to input data for a basin unit as shown in Figure 9-38 (left figure). This model is a black box model of not taking into account spatial distribution which expresses basin conditions. Typical models of the lumped model are rational formula, tank model, storage function method.

Recently data base on topography, vegetation, soil, etc. and radar rain gage system has been promoted, distributed model shown in Figure 9-38 (left) is used by taking into account spatial distribution of hydrological data.



Source: Nagaoka University of Technology, Lu Mingiao, 2006

Figure 9-38 Lumped Model and Distributed Model

2) Rainfall-runoff analysis and flood analysis

A feasibility study of hydropower project needs the following runoff analyses.

Runoff analysis: Flow data is analyzed to calculate power and energy of hydropower planning. This method can estimate daily, monthly and/or yearly flow for a relatively long term period. Record of rainfall data is generally much longer than that of river flow as shown in Figure 9-39. Runoff model is made by using the data of rainfall and river flow for the same period, then flow data of long term period is generated.

(a) Flood analysis: Flood discharge is analyzed for designing of dams and other civil structures. The flood discharge is calculated by using flow data of hourly and/or shorter period. A model is made by analyzing past records of rainfall and river flow during flood period. Then flood discharge is calculated for predicted rainfall pattern which is different from the past record.

3) Method of Analysis

The method consists of following two components

- Relation between rainfall and runoff is analyzed by using the both existing data. It is called “Model identification”.
- Flow data is made for arbitrary rainfall data by the model.

Figure 9-39shows the period of the existing data for rainfall and runoff. River flows is calculated for corresponding existing rainfall data.

Figure 9-40 shows the work process to make runoff model and flow estimate by the model. The process consists of analysis of rainfall data and flow data, model identification and calculation of runoff.

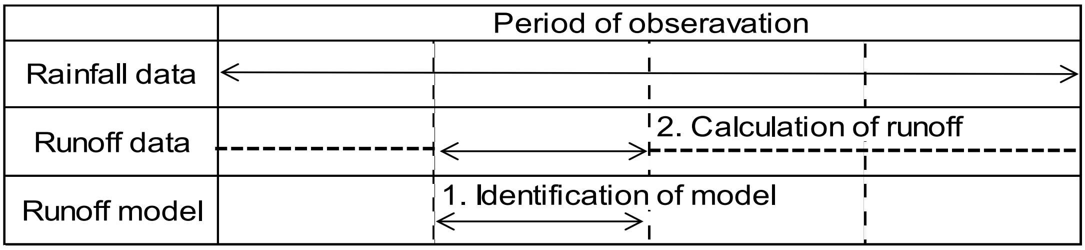


Figure 9-39 Period of Rainfall and Runoff Observation, and Runoff Analysis

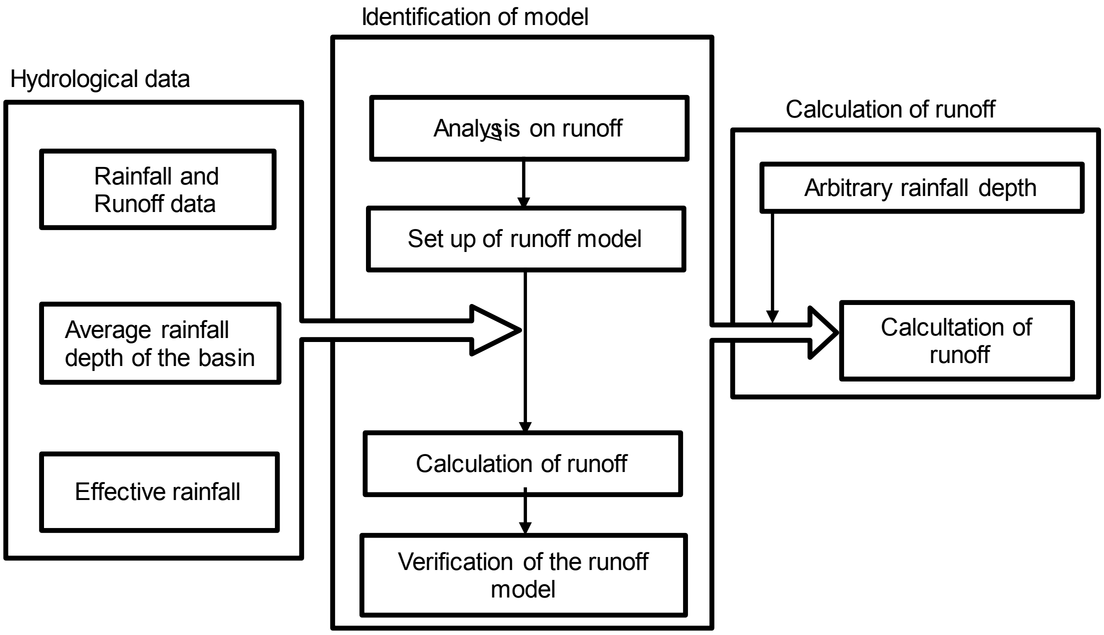


Figure 9-40 Flow Chart of Analysis

4) Lumped model

There are many lumped modes such as Tank model for long term period, storage function for analyzing flood hydrograph. The Tank model is explained below.

The tank model method is a runoff calculation method in which the river basin is replaced by tanks with several outlets in their sides and bottom as shown in Figure 9-41. The rain water is put into the top tank of the model. The second tank receives the water from the outlets in the bottom of the top tank. A part of the water in each tank flows through the side outlets. The remainder of the water drops to the next lower tank through the bottom outlets. The total flow from the side outlets of each container is the river flow. This model is considered to respond to the aquifer structure of the river basin shown in Figure 9-41. The top tank corresponds to the surface runoff, the second tank corresponds to the subsurface runoff, and the third tank corresponds to the ground water runoff.

The hydrological data such as daily runoff data of at least 3 to 5 years, rainfall, evapotranspiration, atmospheric temperature, etc. are required to calculate the flow using a tank model.

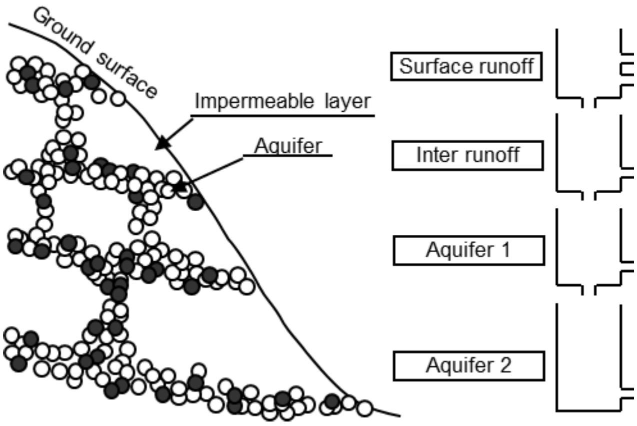


Figure 9-41 Tank Model and Example of Runoff System

The 1st tank as an example for time (i) and time (i+1), as shown in Figure 9-42. Flow from the side outlet and infiltration amount from the bottom outlet are expressed as follows.

$$
\begin{aligned}
q &= k_1(h-h_1) + k_2(h-h_2) && (h > h_2) \\
&= k_1(h-h_1) && (h_1 < h \leq h_2) \\
&= 0 && (h < h_1)
\end{aligned}
$$

$$q_d = k_3 h$$

Water sytorage volume at the bigining of time (i+1) is expressed as follows.

$$h' = h - q - q_d + P_{n+1}$$

where,

Q	: flow from the side outlet (mm)
q_d	: infiltration amount from bottom outlet (mm)
h	: water storage volume at time (i) (mm)
h'	: water storage volume at time (i+1) (mm)
h_1 , h_2	: heights of side outlets (mm)
k_1, k_2	: parameter of side outlet (runoff coefficient)
k_3	: parameter of bottom outlet (infiltration coefficient) (mm)
P_n	: rainfall depth at time (n) (mm)

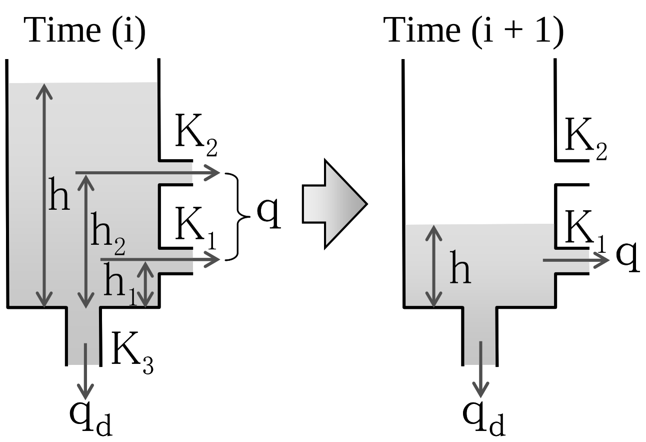


Figure 9-42 Tank Model and Temporal Alteration of the Tank

Flow amount(mm) of one day is converted to flow rate（m3/sec) as follows.

$Q = q \times A / 86.4$ (86.4=24hour×3,600sec/1000

where,

q	: flow from the side outlet (mm)
A	: catchment area (km^2)
Q	: flow (m^3/sec)

5) Distributed model

Distributed model as shown in Figure 9-43 is the analysis method in which surface flow, subsurface flow, groundwater flow and river flow is calculated. Topographical information of each mesh, the direction of water flow, rainfall, vegetation and soil information are used for analysis. The model is recently used because factors to affect runoff such as land elevation of, slope, vegetation, geology, etc. can be utilized as database for 1 km mesh. Figure 9-43 shows a grid model indicating the direction of water flow, and other information mentioned above is added for the analysis.

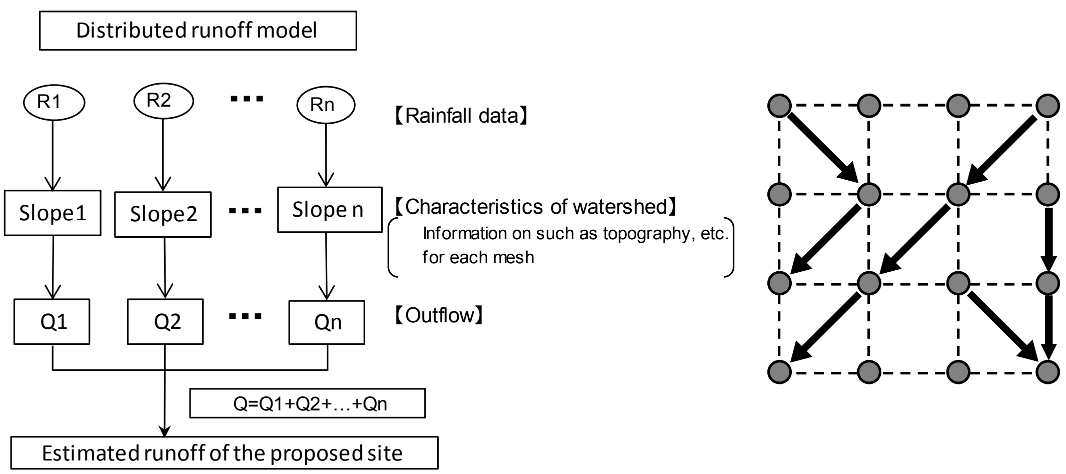


Figure 9-43 Distributed Runoff Model

6) Other method

The stochastic approach enables production of long period flow data from a short period flow data by generating more data while retaining the statistical elements (average, dispersion, skewness, and correlation coefficient) that are particular in the observed time sequence data such as the flow and rainfall. The Markov process is a linear self regression process and is expressed by the following equation;

$$X_i = A \times X_{i-1} + B \times \varepsilon_i$$

where,

X_i	: unknown hydrological volume at time of (i)
X_{i-1}	: known hydrological volume at time of (i-1)
ε_i	: random number at time of (i)
A, B	: coefficients acquired from the statistic of original data

9.3.4 Study on Evaporation

Hydropower planning needs usable flow data by a power plant under the condition a reservoir is constructed. Figure 9-44 and 9-45are schematic figures to show water balance of the river basin. Before dam construction shown in Figure 9-44, evapotranspiration occurs at the river and forest etc., after dam construction shown in Figure 9-45, it occurs at the reservoir having free water surface, river , forest, etc. Therefore the effect of evaporation should be considered for hydropower planning. Especially it should be carefully considered for the projects in the arid region and semiarid regions.

(1) Before dam construction

$$P' = R' + E'_{TA1} + E'_{TA2}$$

where,

P’	: rainfall of watershed at the dam site before dam construction (mm)
R’	: flow at the dam site before dam construction (mm)
E'_{TA1}	: actual evapotranspiration within the reservoir area (mm)
E'_{TA2}	: actual evapotranspiration outside of reservoir area (mm)

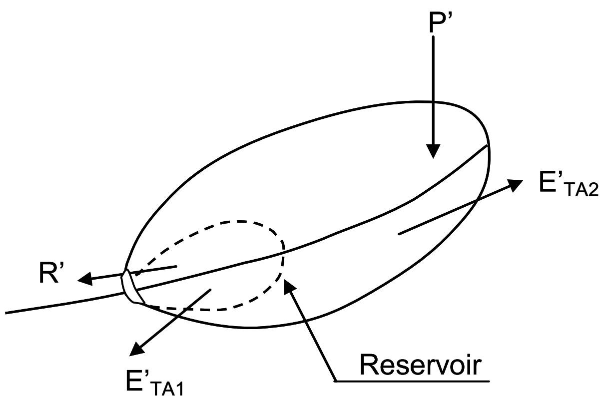


Figure 9-44 Water Balance at Dam Site

(2) After dam construction

$$P=R+E_f+E_{TA2}$$

where,

P	: rainfall of watershed at the dam site after dam construction (mm)
R	: flow at the dam site after dam construction (mm)
E_f	: Evaporation from reservoir surface area (mm)
E_{TA2}	: actual evapotranspiration outside of reservoir area (mm)

Here, since $P=P'$, $E_{TA2}=E'_{TA2}$, we have; $R=R' - (E_f - E'_{TA1})$

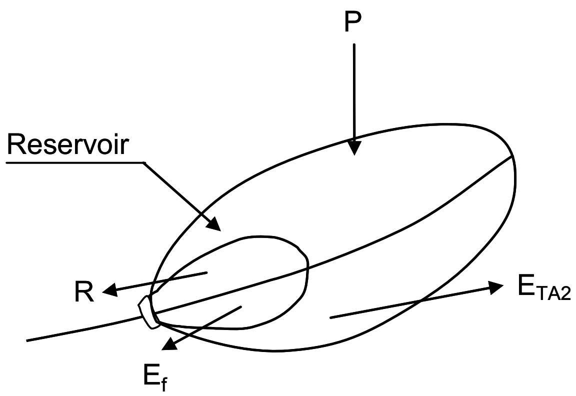


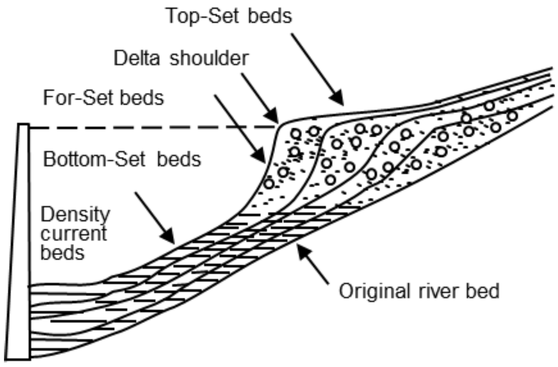
Figure 9-45 Water Balance at Dam Site

Consequently, the evaporation occurring due to dam construction is (E_f - E'_{TA1}). Since R' is already known, R is acquired once E_f and E'_{TA1} are acquired. Evaporation E_f from the reservoir is obtained by multiplying the measurement value of a Class-A pan and evaporation pan coefficient of 0.7. Actual evapotranspiration E'_{TA1} before construction is estimated from the potential evapotranspiration using Blaney-Criddle equation, etc. or from the water balance of river basin.

9.3.5 Study on Reservoir Sedimentation

(1) Shape of sedimentation and influence to power generation

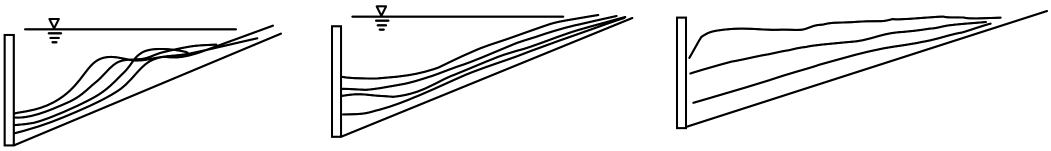
After completion of a dam, soil, sand and gravel flow into reservoir and pond, and become deposited. The shape consists of Top-Set beds, For-Set beds, Bottom-Set beds and Density current beds as shown in Figure 9-46. Top-Set beds and For-Set beds have the terrace shapes which is called delta consisting of relatively large grain size such as bed load and suspended load. The sedimentary layers is formed by settling of soil and sand due to speed reduction of river water in the reservoir. The front slope which equal to the angle of repose in water becomes steep compared with the original river bed. On the other hand, the gradient of Top-Set beds is more gentle than that of original river bed. This delta moves forward with its shoulder keeping to be the water level of the reservoir. And upstream end of the Top-Set beds moves to the upstream. Bottom-Set beds and Density current beds consists of wash load and relatively small particle of suspended load.



Source: Disaster Prevention Research Institute Kyoto University, K.Ashida, T.Okumura

Figure 9-46 Typical Shape of Reservoir Sedimentation

Actual shape differs depending of reservoir water level, volume of supply of sediment, river gradient of upstream of reservoir as shown in Figure 9-46 and Figure 9-47.



Source: Kyoto University, K.Ezaki

Figure 9-47 Shape of Sedimentation

Sediment phenomena progresses and it reduces effective storage capacity between HWL and LWL as shown in Figure 9-48(left figure). In this stage, reservoir storage effect to regulate river water is reduced, and consequently the power plant cannot generate energy as expected at the planning stage. In the case the sediment reaches to the intake and the level reaches the intake sill, the power plant faces a problem to stop the generation because of intrusion of sediment to the waterway. In order to avoid this phenomenon, there are some methods to be taken. Flushing method and dredging method carry the sediment to the space under LWL. Dredging method and flood bypass tunnel method remove the sediment to the downstream of the dam.

(2) Design sediment volume and level

At the planning stage, it is assumed that the sediment volume deposited between HWL and LWL is moved and stored under the LWL as shown in Figure 9-48 (right figure).

Sediment volume for a design period is assumed at the planning stage, and it is used for reservoir planning. The LWL should be set considering sediment volume and the level, and required water depth for the intake structure. In many cases 100 years is adopted as the design period, so this Manual uses the value of 100 year for the planning.

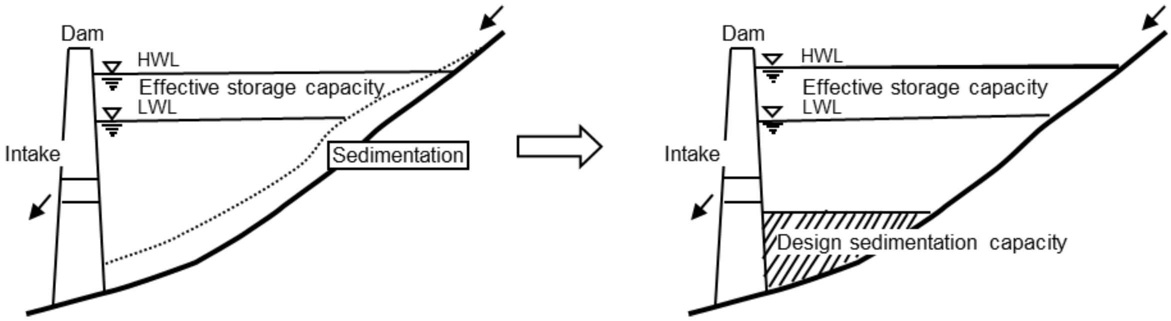


Figure 9-48 Planning of Reservoir for Sedimentation

(3) Estimate of reservoir sedimentation

Concerning the calculation of sediment volume in a reservoir or a pond, there are two methods explained bellow.

1) Estimate by the actual measurement of suspended load

(a) Calculation flow

Although measurement of bed load is not possible but that of suspended load is possible. In the case data of suspended load is available near the proposed dam site, sediment volume of reservoir can be estimated as follows.

- Annual suspended sediment (weight) flowing into a reservoir is calculated from measurement data of suspended load.
- Annual bed load (weight) is calculated from relation between suspended load and bed load.
- Taking into account the trap efficiency of the reservoir, the suspended load (weight) of trapped in the reservoir is calculated.
- Density of the total load is calculated from the bed load and suspended load trapped in the reservoir.
- Sediment volume for 100 years is calculated on the basis of density.

(b) Estimate by the actual measurement of suspended load

Generally, suspended load is expressed by concentration (mg/1 = p.p.m.). The flow is expressed by the mean daily flow (m3/sec). These are plotted on the log-log paper as shown in Figure 9-49. Coefficients (a) and (b) are acquired by the least square method in the following functional equation. Annual suspended load is calculated by using this formula, however it is recommended that the load should be calculated by using hourly flow data instead of the mean daily flow.

$$\ell og\ qs = a \times \ell og q + b$$

$$\sum qs = \sum_{i=1}^{365}(qsi \times qi \times 86.4)$$

where,

qi	: mean daily flow (m^3/sec)
qsi	: suspended load (mg/ℓ)
a, b	: coefficients
$\sum$qs	: annual suspended load (kg)

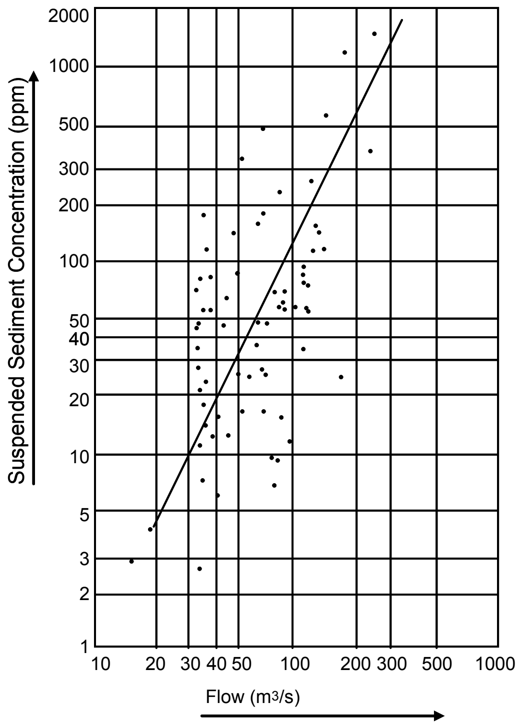


Figure 9-49 Relation Between Suspended Sediment Concentration and Flow

(c) Estimate of bed load

Since bed load cannot be measured, bed load is normally expressed by the ratio against the suspended load. According to the Bureau of Reclamation shown in Table 9-5, the bed load ratio against the suspended load can be estimated by considering river condition such as suspended sediment concentration, streambed material, texture of suspended load, percent bed load in terms of suspended load.

Table 9-6 Bed Load Correction

Condition	Suspended sediment concentration (mg/ℓ)	Streambed material	Texture of suspended material	Percent bed load in terms of suspended load
1/1	<1,000	Sand	20 to 50 percent sand	25 to 150
1/2	1.000 to 7,500	Sand	20 to 50 percent sand	10 to 35
3	>7,500	Sand	20 to 50 percent sand	5
2/4	Any concentration	Compacted clay gravel, cobbles, or boulders	Small amount up to 25 percent sand	5 to 15
5	Any concentration	Clay and silt	No sand	<2

1/ Special sampling program for Modified Einstein computations required under these conditions.

2/ A bedload sampler such as the Helley-Smith bed load sampler may be used or computations made by use of two or more of the bed load equations when bed material is gravel or cobble size.

Source：Technical Guideline for Bureau of Reclamation, US Bureau of Reclamation, October 1982

(d) Trap efficiency

The capability of a reservoir to trap and retain sediment is known as the trap efficiency. The relation between the "trap efficiency" and "total storage capacity/total annual inflow" is indicated by G.M. Brune in Figure 9-50. When the ratio of total storage capacity against total annual inflow is small, the water is frequently discharged from the spillway, and much sediment is carried out from the reservoir in the overflow water, resulting in small trap efficiency. Contrarily, when its ratio is large, there is almost no spill from the spillway and most of the sediment flowing into the reservoir is trapped.

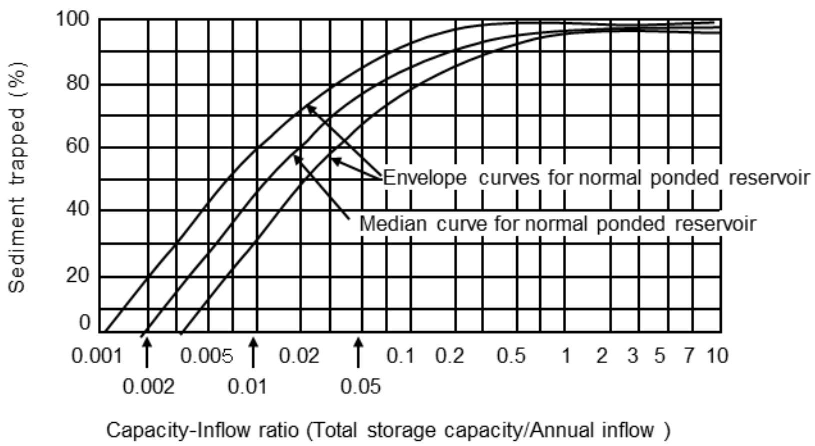


Figure 9-50 Trap Efficiency

(e) Density of deposited sediment

Volumes of bed load and suspended load trapped in a reservoir is obtained from b), c) and d). As the sediment is consolidated as time passes, its density increases. Method of the Bureau of Reclamation to calculate the sediment density is explained bellow.

Types of sediment consist of clay, silt and sand. The size ranges of clay, silt and sand are less than 0.004mm, 0.004 - 0.062mm and 0.062 - 2.0mm for clay, silt and sand respectively. By using these values, the sediment density is calculated as follows.

$$W=WcPc + WmPm + WsPs$$

where,

W	: unit weight（kg/m)
Pc, Pm, Ps	: percentage of clay, silt and sand, respectively
Wc, Wm, Ws	: coefficients for initial weight for clay, silt, sand, respectively

In determining the density of sediment deposits, it is recognized that part of the sediment will deposit in the reservoir in each of the T years of operation, and each year's deposits will have a different compaction time. Average density of all sediment deposited in T years of operation is expressed as follows.

$$W_T=W_1+0.434K[T/(T-1)\times(\log_e T)-1]$$

where,

W_T	: average density after T years of reservoir operation (kg/m^3)
W_1	: initial unit weight (density) as derived from formula above (kg/m^3)
K	: Constant based on type of reservoir operation and sediment size analysis

(f) Design sedimentation volume

Since design sedimentation volume of a dam is determined for 100 years, the value for 100 years is expressed in the following formula.

$$Vs_{100}=\alpha(\sum qs\times 100)(1+\beta) / W_{100}$$

where,

Vs_{100}	: design sedimentation volume (m^3)
$\sum qs$	: annual suspended load (kg/year)
α	: trap efficiency against total sediment volume flowing into reservoir (%)
β	: ratio of bed load to suspended load (%)
W_{100}	: average density in 100 years (kg/ m^3)

2) Estimate by sedimentation record of existing dam

When dams already exists near the proposed dam site and its sedimentation records are available, the sediment volume of the proposed dam site is estimated from the specific sediment yield which is the sediment volume delivered to the reservoir from catchment area of 1 km^2 in a year (unit: m^3/km^2/year).

$Vs_{100} = v \times A \times 100$

where,

Vs_{100}	: design sedimentation volume (m^3)
V	: specific sediment yield ($m^3/km^2/year$)
A	: catchment area (km^2)

If the condition such as climate, geology, topography and scale of reservoir are resemble in the proposed project site, the nearby sedimentation record can contribute to estimate the sediment volume with high reliability.

(4) Prediction of shape of sedimentation

Horizontal shape of sedimentation is shown in Figure 9-48 which is assumed at the planning stage. However, if the effective storage capacity between HWL and LWL might be predicted to face the following situation,

- Effective storage capacity will be decreased due to sedimentation
- Serious effect will be caused even though a countermeasure to discharge the sediment from reservoir is taken

In this situation, prediction of sediment shape might be necessary from a view point of long term maintenance of the reservoir. One-dimension river bed variation analysis is used for prediction of sedimentation shape, which consists of calculation of water profile (on steady flow), river bed variation analysis.

9.3.6 Flood Analysis for Design of Dam

(1) Design flood

Calculation methods used for flood discharge are classified to a method based on runoff data and to a method based on rainfall data. The empirical and regional calculation formulas, envelope curve, frequency analysis, etc. belong to the former method. Probable Maximum Flood (PMF) derived from analysis of Probable Maximum Precipitation belongs to the latter method.

Dam failure might cause tremendous damage against downstream area. The damage consists of factors of loss of life and economic loss. Table 9-7 shows guidelines of design flood of US Army Corp of Engineers (USCE) concerning the safety of dams, which is relating to reservoir scale and causes of disaster. Table 9-7 shows guidelines of Australian Committee on calculation of design flood. PMF and probable flood are adopted in both guidelines. The detail is written in “Selection of Design Flood, Icold 1992, Bulletin 82”.

There are following basic concepts regarding the design flood discharge for the spillway;

- When the reservoir capacity is large and its failure could result in tremendous loss and damage to human life and property, the Probable Maximum Flood (PMF) or equivalent probable flood discharge is applied as the design flood discharge.

- When the reservoir capacity is small and its failure would not result in tremendous loss or damage to human life and property on the downstream, the probable flood discharge is used.

Table 9-7 Guidelines of the US Army Corps of Engineers

(Size classification)

Category	Reservoir capacity ($10^6 m^3$)	Height of the dam (m)
Small	from 0.62 to 1.23	from 7.6 to 12.2
Intermediate	from 1.23 to 61.5	from 12.2 to 30.5
Large	≥61.5	≥30.5

(Hazard potential classification)

Category	Loss of life (Extent of development)	Economic loss (Extent of development)
Low	None expected (No permanent structures for human habitation)	Minimal (Undeveloped to occasional structures or agriculture)
Significant	Few (No urban developments and no more than a small number of inhabitable structures)	Appreciable (Notable agriculture, industry or structures)
High	More than few	Excessive (Extensive community, industry or agriculture)

(Recommended safety standard)

Hazard	Size	Safety standard
Low	Small Intermediate Large	50 to 100 year flood 100-year flood to 50% of the PMF 50% to 100 % of the PMF
Significant	Small Intermediate Large	100-year flood to 50% of the PMF 50% to 100% of the PMF PMF
High	Small Intermediate Large	50% to 100% of the PMF PMF PMF

Table 9-8 Guidelines of the Australian Committee on Large Dams for the Calculation of Design Flood

(Incremental Flood Hazard Categories)

High	Significant	Low
Loss of identifiable life expected because of community or other significant developments downstream.	No loss of life expected, but the possibility recognized. No urban development and no more than a small number of habitable structures downstream.	No loss of life expected.
Excessive economic loss, such as serious damage to communities, industrial, commercial or agricultural facilities, important utilities, the dam itself or other storages downstream.	Appreciable economic loss, such as damage to secondary roads, minor railways, relatively important public utilities, the dam itself or other storages downstream.	Minimal economic loss, such as farm buildings; limited damage to agricultural land, minor roads, etc.
Dam essential for services and repair not practicable.	Repairs to dam practicable or alternative sources of water/power supply available.	Repairs to dam practicable. Indirect losses not significant.

(Recommended Design Floods (AEP) Annual Exceedance Probability)

Incremental flood hazard category (IFHC)		Annual exceedance probability (AEP)
High	Loss of life, extreme damage	10,000 years flood to PMF
Significant	Unlikely of life, significant damage	1,000 to 10,000 years flood
Low	No loss of life, minor damage	100 to 1,000 years flood

In Japan, the maximum historical flow or 200-year probable flood discharge which is estimated to occur at a rate of once in 200 years is applied to concrete dams and a 1.2 times larger flood discharge than that for concrete dams is applied to fill dams.

In this Manual, the probable flood discharge and the PMF which is specific to a large storage capacity reservoir are outlined.

(2) Calculation of design flood discharge

1) Probable flood discharge

Hydrological value of X_T of T- year return period is calculated by using a certain flood flow of Xu and probability density functions. Generally, Gumbel’s distribution, Pearson distribution, log normal distribution, log Pearson type III distribution are frequently applied as the probability density function to analyze the annual peak flood flow. The data (Xu) of annual flood peak flow of at least 20 to 30 years is required for the calculation. Figure 9-51 shows examples of analyses for Gumbel’s distribution and log Pearson type III distribution.

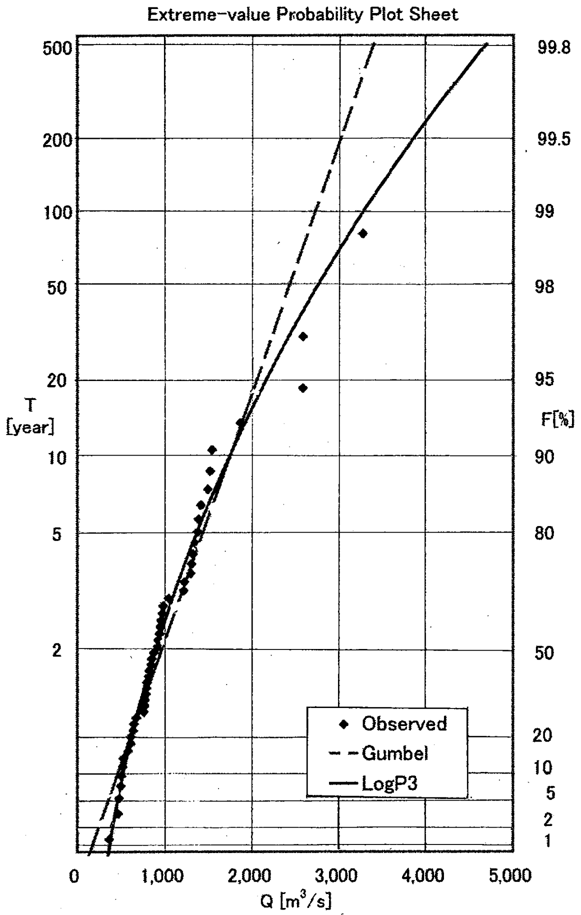


Figure 9-51 Examples of Gumbel's and Log Pearson type III Distributions

2) Probable maximum flood (PMF)

PMF is generally defined as flood expected when the theoretically worst hydrologic and meteorological conditions are combined in an area. It is necessary to consider the three factors of "runoff due to rainfall", "runoff due to melting snow" and "base runoff" that form PMF. Flow chart for calculation of PMF is shown in Figure 9-52. The example of PMF for the possibility of rainfall occurring in a few days' intervals is shown in Figure 9-53. As further information, please refer to the special technical book on PMP published by WMO.

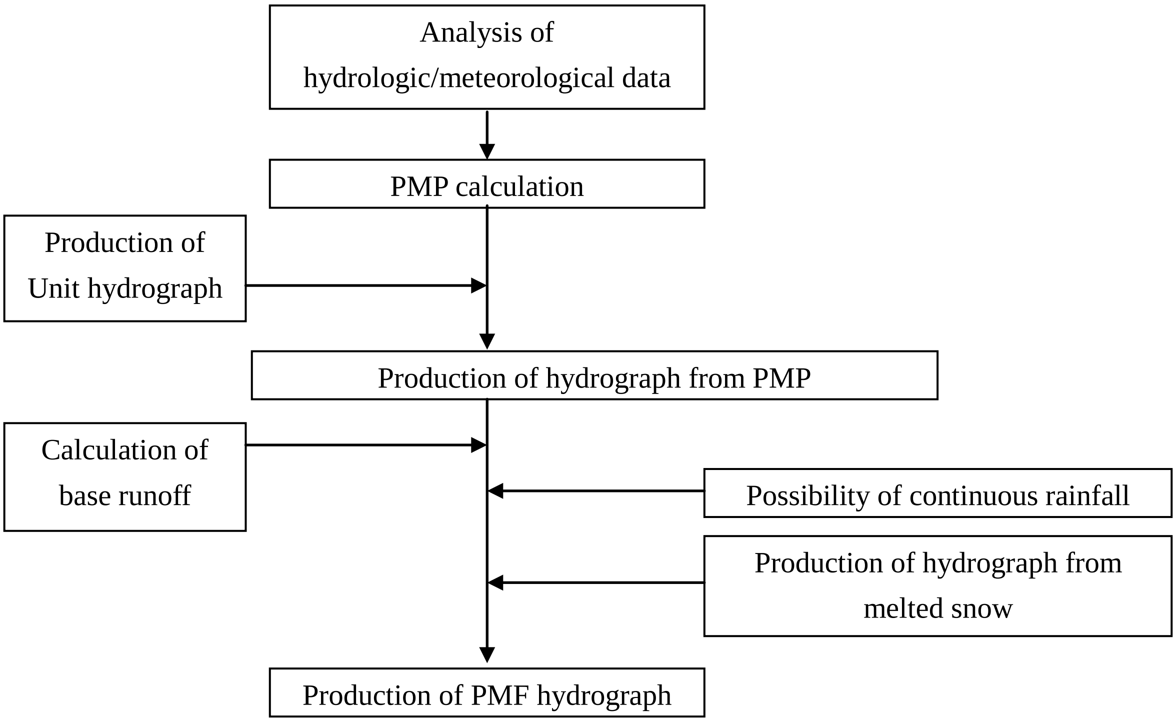


Figure 9-52 Calculation Flow of PMF

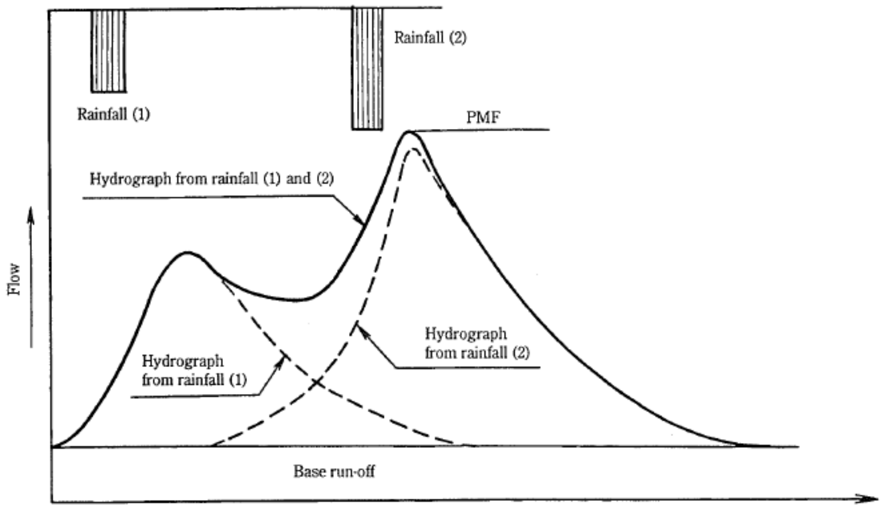


Figure 9-53 PMF Hydrograph During Continuous Rainfall

Reference

[1] Technical guide to aseimic design of Nuclear Power Plants, Japan Electric Association, 1987

[2] Technical Guideline for Bureau of Reclamation, Sedimentation and Hydraulics Section Hydrology Branch, Division of Planning Technical Services, Engineering and Research Center, 1982

[3] Effort against Sedimentation and Turbid Water in Reservoir and Problems (in Japanese), Japan Society of Civil Engineers, 2002

[4] Considerations about setting the design sediment capacity and a rough estimate method for reservoir sedimentation based on sediment inflow (in Japanese), Water Resources Environment Technology Center

[5] Handbook on hydrology and water resources (in Japanese), Japan Society of Hydrology and Water Resources

[6] Guideline on River flow Investigation for Hydropower (in Japanese), Electric power Engineering Association

[7] Selection of Design Flood, Bulletin 82, ICOLD, 1992

Chapter 10

Planning of Conventional Hydropower Projects

Chapter 10 Planning of Conventional Hydropower Projects

10.1 General

10.1.1 Flow of Project Planning

A feasibility study is conducted for promising projects which are selected from a reconnaissance study, hydropower potential study and master plan study. The purpose of feasibility study is to analyze the project from a technical, economic and environmental view point to judge the possibility of development.

Figure 5-2 in Chapter 5 shows the planning flow chart of hydropower projects. Latest data are used in the feasibility study to determine the development scale and to examine the commissioning time from the power demand forecast and power development plan.

10.1.2 Review of Existing Reports

(1) Review of Hydropower potential survey and master plan study

1) Reconnaissance study of individual project and hydropower potential survey

In case a hydropower potential study report is available, whether the project is worth to conduct a feasibility study or not is examined and re-studied on the desk and on site. Promising projects are compared from a view point of future demand, access road, transmission lines, effects to other projects, land usage in the pond/reservoir area, economy and environmental impact, etc.

Although a potential study report was not prepared, reports on individual projects should be reviewed if the reports are available.

2) Master plan study

Promising projects in a report of master plan study should be examined in the same way as the potential survey. The order of priority for each project should be made before starting the feasibility study referring to the study result of master plan. When the master plan contains several core projects which play an important role for basin development, the development scale and order may mutually be affected due to regulating effect of upstream reservoirs, the cost allocation of transmission lines and other factors. In this case, an integrated feasibility study should be conducted for the several core projects.

(2) Pre-feasibility study and feasibility study

When sufficient data were not available to conduct feasibility study, a pre-feasibility study should be made as a step before feasibility study. Each project might face many obstacles, and the projects might not be implemented even though its feasibility study shows good result from an economical point of view.

Feasibility study including the following study items should be revised.

In case the development priority is low compared with other projects due to the low economic viability, the following might be one of countermeasure. The feasibility study should be carried out to increase its economic viability from several viewpoints such as development method of the basin, synergy effect between upstream and downstream projects, burden share of transmission line and/or access road costs. In case there are core projects in the same basin, the development order should be studied.

In case an environmental issue is an obstacle to construct the project due to large scale of a dam, the project should be re-studied by lowering the dam height, reducing the plant output or avoiding the restricted area.

10.1.3 Basic Information for Project Planning

Following information is necessary for conducting a feasibility study. When these data are not available, a pre-feasibility study should be conducted instead of feasibility study.

(1) Topographic map

Topographic maps of the following accuracy are usually required for a feasibility study.

- Project site (reservoir and civil structures);
  About 1:5,000 - 1:10,000 for small reservoir
  About 1:25,000 for large reservoirs
- Dam site and powerhouse site;
  About 1:1,000 – 1:2,000 scale

(2) Runoff, evaporation, sedimentation data at dam site

It is preferable to use river flow data covering a period of 20 to 30 years. If the data are unavailable, data are prepared for longer period by using the methods described in Chapter 9.

Reservoir type hydropower requires monthly flow data for the planning study. Pondage and run-of-river types require daily flow data. If flow data is not measured at or near the dam site, it is difficult to conduct feasibility study. Therefore, it is necessary to install a gaging station to measure reliable flow at the dam site.

For a study of reservoir type, the reservoir operation should be studied taking the evaporation from the reservoir into consideration. Therefore, evaporation data should be prepared. Sedimentation data should also be prepared for reservoir and pondage types, as described in Chapter 9.

(3) Rating curve at powerhouse or tailrace site

In order to calculate the power output and energy, and to design the powerhouse and/or tailrace, rating curve which shows relationship between river flow and its water level at the site is necessary. Generally, since river flow at the tailrace site is not observed, the rating curve is made with a river cross section and river gradient of the site by assuming Manning' coefficient of roughness and uniform flow condition. If several river cross sections are available downstream of the site, more accurate rating curve can be made by varied flow calculation.

(4) Geologic data for dam, powerhouse and main structure sites

Geologic data by surface reconnaissance, drilling, and other methods are required.

(5) Load curve on the day of maximum load day and the supply power source

These data are required to study the necessity of the project from a standpoint of supply and demand.

(6) Information and data related to environmental regulations

10.1.4 Position of Supply Power

In the feasibility stage, future demand and supply capability should be kept in mind so that the project study is conducted clearly identifying its role.

(1) Electric power system where a new hydropower is proposed

When studying a project, it is important to understand the characteristics of the electric power system from the viewpoint of both demand and composition of power source. Figure 10-1 gives examples of load duration curves in which demand (load) are arranged in the order of magnitude.

Figure (a) shows a system supplied mainly by thermal power plants, and hydropower is used mainly for peak loads. The supply capability by thermal power plants is almost constant throughout the year, and the critical power supply occurs on the day when the maximum demand occurs and/or dry season when capability of hydropower drops.

Figure (b) shows a system supplied mainly by hydropower. The characteristic is that the power supply is affected by rainy and dry seasons. The critical power supply occurs on the day of maximum demand during the dry season.

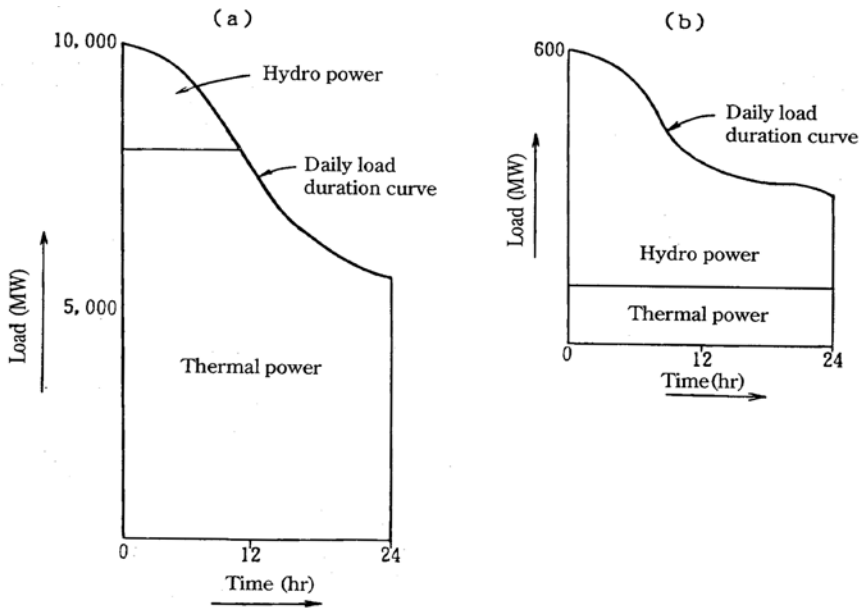


Figure 10-1 Examples of Daily Load Duration Curve and Supply Capability

(2) Power source for future demand

A schematic of future power demand and power supply capability is shown in Figure 10-2. Future Figure (b) is obtained by extrapolating the current demand (a) by the estimated growth rate.

If the current load curve is deformed due to deficiency of power supply capability, it is necessary to correct the load curve by taking latent demand into consideration. Shaded part in Figure 10-2 (b) indicates the supply capability of existing power plants. The white part at the top of the figure indicates the shortage of supply capability, and most appropriate combination of hydropower, thermal power and other power sources will supply for this part. Concerning hydropower projects, the most appropriate projects for this part is chosen.

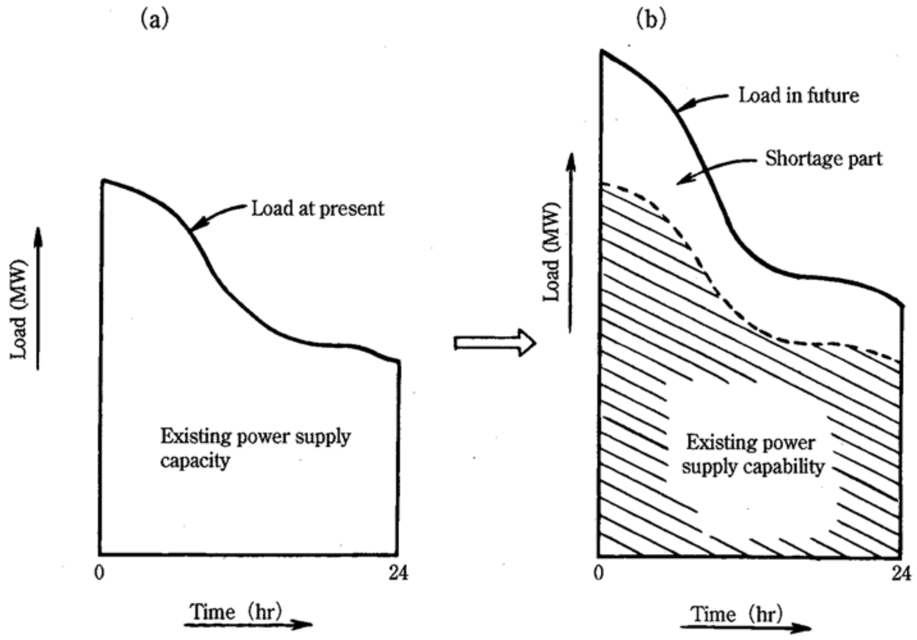


Figure 10-2 Relation between Future Demand and Planned Project

10.1.5 Methodology of the Planning

The terminology of hydropower planning is explained in Chapter 5.

(1) Sequence of study

The contents of the study are basically the same as the flow chart shown in Figure 5-2, Chapter 5. However, the latest data are used for the feasibility study. The main points are as follows;

1) Determination of type of power generation

The types of power source required from the standpoint of demand are described in previous section (3). Type of power generation such as run-of-river, reservoir and poundage types is determined from demand supply characteristics of the power system and by confirming the topography and geology at the project site.

2) Optimization of scale of development

Alternatives changing the waterway route, dam and powerhouse sites, storage capacity of reservoir and maximum plant discharge are compared. The optimum development plan is determined by the economic analysis described in Item (4).

The typical parameters in this study are;

- Reservoir and pondage types

  High water level (dam height), effective storage capacity, maximum plant discharge, waterway route (locations of dam and powerhouse), water use of tributaries

- Run-of-river type

  Maximum plant discharge, waterway route (locations of intake weir and powerhouse), water use of tributaries

3) Number of units of turbine & generator and unit capacity

   Generally, the larger the unit capacity is, the better the project economics improves because of scale merit. Therefore, the unit capacity should be as large as possible bearing in mind reliability, manufacturing technology and transportation. Although a large unit does give certain merits, outage of the unit causes large frequency fluctuations in the power system, and a larger reserve capacity is required in the event of outage of the equipment. These factors should be considered when determining the unit capacity.

   Since plant discharge of the run-of-river type fluctuates greatly following the river flow, turbine efficiency drops and energy generation decreases significantly when the river flow is small compared with the plant discharge. For this consideration, this problem can be solved by increasing the number of units.

4) Study of timing of implementation

   The implementation time including stage development is studied by economic analysis described in Item (4), using the discounted cash flow method.

(2) Peak duration hours

The concept of peak duration hours as shown in Fig 10-3 is used to study reservoir, pondage and pumped storage type projects. When a hydropower plant is continuously operated to supply the energy (E kWh) at the maximum output (P kW), the concept is expressed as T = E/P. Peak duration hours is generally about 4 to 8 hours, which can be obtained by analyzing the load duration curve of power system.

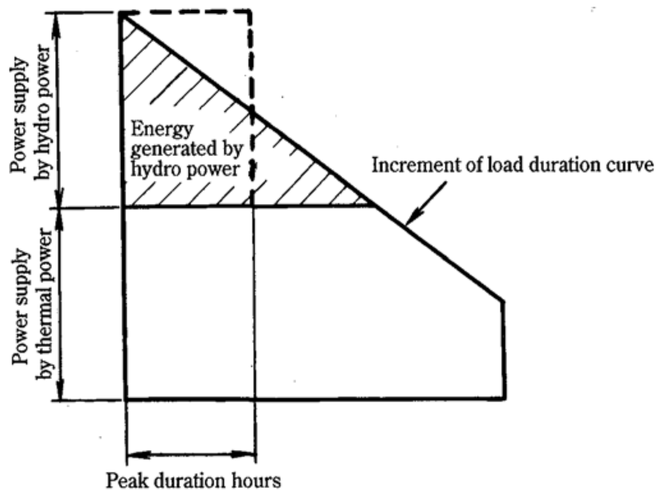


Figure 10-3 Peak duration hours

(3) Maximum output and energy generation

1) Maximum output

Maximum output is calculated using the following formula.

$$P = 9.8 \times Q_{max} \times H_{es} \times \eta_t \times \eta_g$$

where,

P	: Maximum output (kW)
Q_{max}	: Maximum plant discharge (m^3/sec)
H_{es}	: Effective head (m)
η_t	: Turbine efficiency
η_g	: Generator efficiency

2) Head loss and effective head

Figure 10-4 and Figure 10-5 are schematic diagrams of the effective heads of the Francis turbine including propeller turbine and Pelton turbine. After the head loss is calculated for each facility indicated in the diagram, the effective head is calculated by subtracting the head loss from gross head.

Effective head is expressed by the following equation.

Francis turbine and propeller turbine:

$$He = Hg - \left(H_{L1} + H_{L2} + \frac{V_2^2}{2g} + H_{L3} \right)$$

Pelton turbine:

$$He = Hg - \left(H_{L1} + H_{L2} + Hs + H_{L3} \right)$$

where,

Hg	: Gross head (Difference in elevation between normal water level and tailwater level 1
He	: Effective head
H_{L1}	: Sum of head loss of intake, headrace and other waterway structures between intake and tank (surge tank or head tank)
H_{L2}	: Head loss of penstock between head tank and turbine inlet
H_{L3}	: Head loss of tailrace channel between tailrace bay and tailrace
$V_2^2/2g$	: Velocity head of draft tube outlet flow velocity V_2 (Discharge head loss for reaction turbines)
Hs	:Pelton turbine installation height

3) Calculation of output and energy generation

Output and energy generation are calculated from the turbine efficiency, plant discharge and effective head considering fluctuation of flow and head due to fluctuation of reservoir water level.

4) Primary energy and secondary energy

Primary energy and secondary energy should be accounted separately if their values differ in the power system. The concept of these energies is defined in this Manual as follow.

Run-of-river type: Energy generation corresponding to firm discharge is termed primary energy, and the other energy is termed secondary energy.

Reservoir and pondage types: Energy generation corresponding to the firm plant discharge during peak duration hours is termed primary energy, and other energy is termed secondary energy.

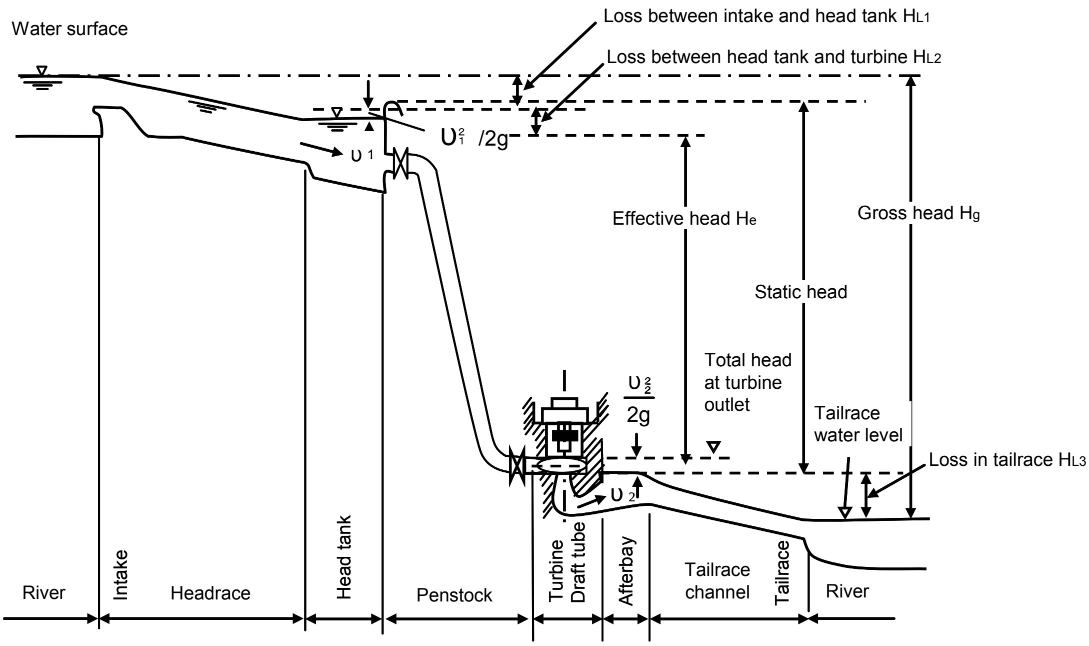


Figure 10-4 Schematic Diagram of Effective Head (Francis Turbine)

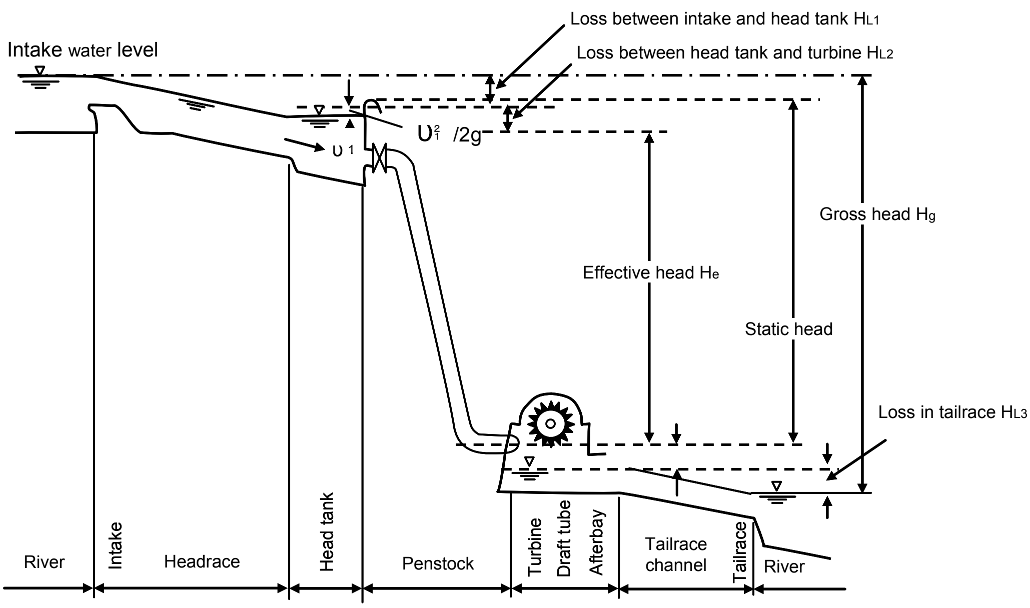


Figure 10-5 Schematic Diagram of Effective Head (Pelton Turbine)

(4) Economic analysis

The cost (C) of a hydro power plant and the cost (B) of an alternative thermal power plant having a supply capability comparable to the hydro power plant are analyzed using the "Discounted Cash Flow method" and compared. Then, the net present value of (B – C), benefit-cost ratio (B/C), and internal rate of return (IRR) are calculated. The analysis giving the highest value is adopted as the optimum plan.

10.2 Planning of Development Scheme

10.2.1 Type of Power Generation

The required supply capability for the supply deficiency indicated in Figure 10-2 is analyzed, and then the type of hydro power is confirmed.

Generally, a reservoir or pondage type capable of supplying peaking power is necessary for the system described in Figure 10-1 (a). A run-of-river, reservoir or pondage type is usually necessary for the system indicated in Figure (b). The type of power generation for the proposed project is decided with consideration given to the supply capability of the existing power source and the planned new thermal power and hydro power plants.

10.2.2 Run-of-River Type

(1) Required supply capability

The run-of-river type is mainly used to supply power to areas remote from large scale power sources where the transmission line system is weak, or to supply power to isolated power systems. A run-of-river type power plant connected to the large scale power system can contribute to reduce the fuel consumption of thermal power plants.

Run-of-river type as source of supply for a small scale system is explained below.

Figure 10-6 shows the relation between the power supply output (P) newly required by the system and the firm output (P_f) often proposed hydropower project. Figure 10-6(a) is the case when the P_f is larger than P, which means that a run-of-river type power plant is appropriate because the power plant has sufficient supply capability to satisfy the demand. When the P_f is significantly larger than P, turbines & generators may be installed in stages corresponding to the growth of future demand.

Figure (b) is the case when P_f is smaller than P. When developing this project site, a pondage type is adopted to regulate the river flow by a pond so that P_f is equal to P, or to develop another run-of-river type project capable of making up the difference.

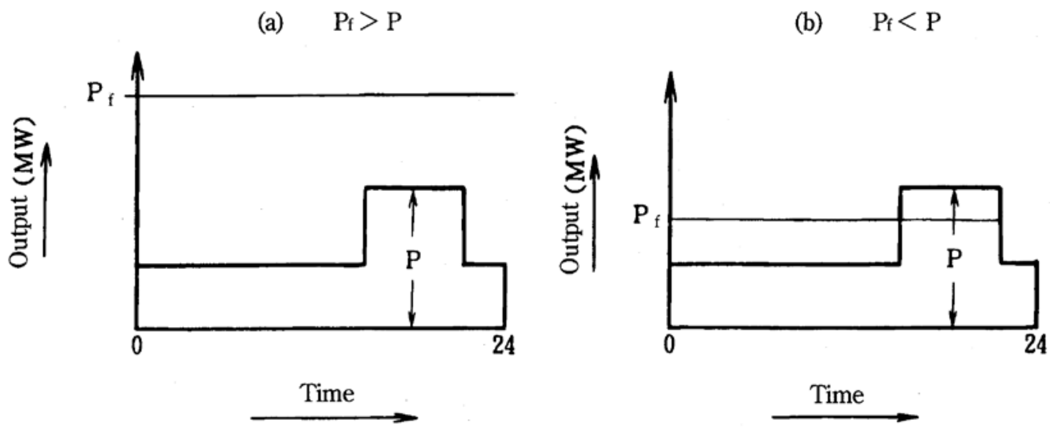


Figure 10-6 Relation between Required Supply Capability and Firm Output

(2) Optimization study of development scale

The main points of the study are as follows.

1) Waterway route

The topography and geology of possible sites are confirmed using topographic maps of scale 1/5,000 - 1/10,000, before proceeding to the feasibility study. Alternative plans changing the sites of intake weir and powerhouse are prepared, and the waterway route selected in the reconnaissance study is re-studied using the latest data such as topographic map, geology, and runoff data. If both an open channel and tunnel are possible for the headrace, the final decision is made by economic comparison.

2) Maximum plant discharge

Alternative plans are prepared varying the maximum plant discharge for the selected waterway route and preliminary designs are made for the major plans. The turbine type is selected with consideration given to the conditions in the proposed plan, and then the efficiency is set. The output and energy generation are calculated for the alternative plan, and economic comparisons are made to determine the optimum maximum plant discharge.

3) Intake facilities at tributary, etc.

It might be technically possible to draw water from a adjacent gully, tributary or river. An economic study is made by comparing the additional cost incurred to draw water and the additional benefit of increased power and energy thereby to judge whether to draw water from adjacent tributaries and river. Run-of-river types have long waterways which often cross a tributary. In many cases, therefore, the facility to draw water from tributaries requires only a small cost increase.

4) Number of turbine unit

During the low flow season, the available discharge of run-of-river type power plants decreases and might be less than the minimum discharge of the turbine. On some days it is not possible to generate power as shown in Figure 10-7. To avoid this situation, a plan to install two or more turbines is necessary to reduce the discharge for one turbine.

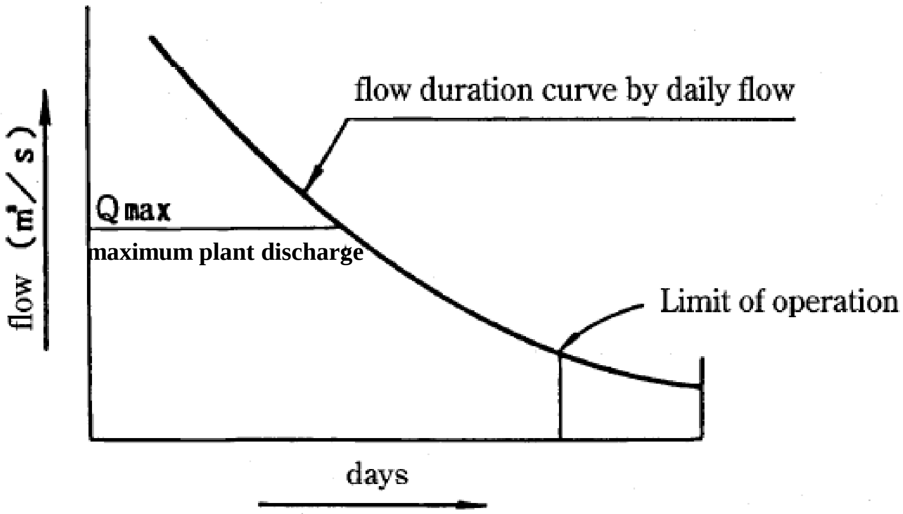


Figure 10-7 Flow Duration Curve and Operating Limit

5) Development scale and time of implementation

The effective head is obtained by fixing the dam and powerhouse sites, and the maximum plant discharge is determined, and then the maximum output is determined. The implementation timing of the project is studied, and the sequence of development is studied in case staged development is needed.

10.2.3 Reservoir Type

(1) Required supply capability

There are the following functional purposes of reservoir type hydropower projects.

1) Supply source for peak demand

Reservoir type power plants are generally used as a power source for peak demand in electric power systems composed mainly of thermal power. This pattern depicted on a mass curve and daily load duration curve is given in Figure 10-8(a) and (b). The mass curve shows an example of movement of reservoir water surface from high water level to low water level in one year, and q_1 represents firm discharge corresponding to firm peak output. This discharge corresponds to primary energy, and discharge of q_2, is larger than q_1, the difference means secondary energy indicated in Figure 10-8(b).

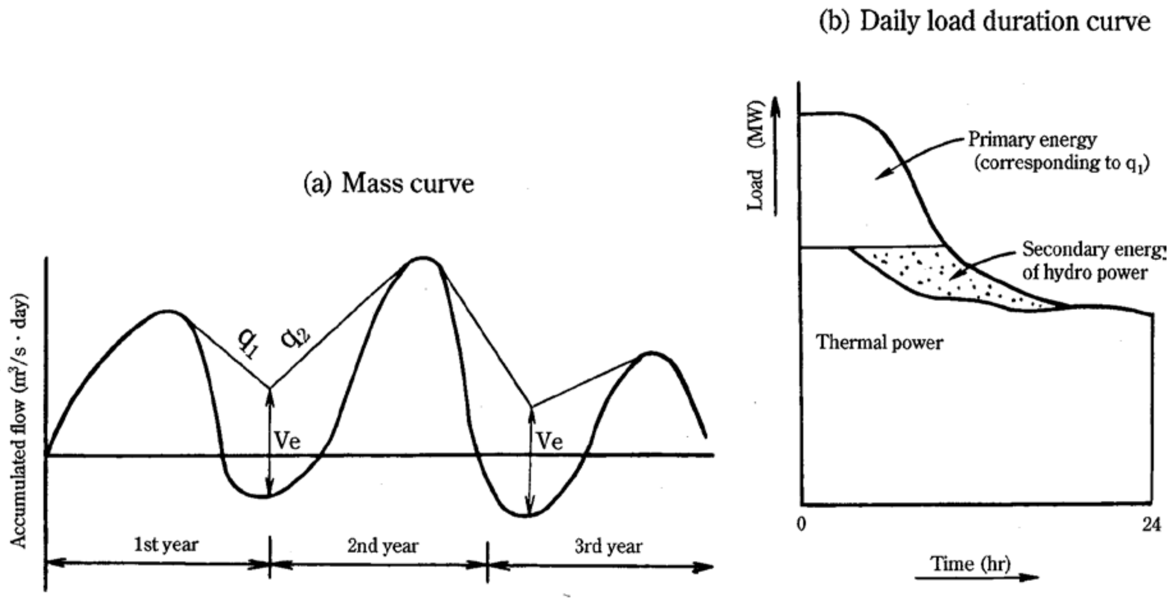


Figure 10-8 Mass Curve and Daily Load Duration Curve

Figure 10-9 shows the mass curve of a carry-over reservoir which is operated from full to empty condition once in several years. In this case, the discharge q_1 indicates the firm discharge and very little secondary energy is generated.

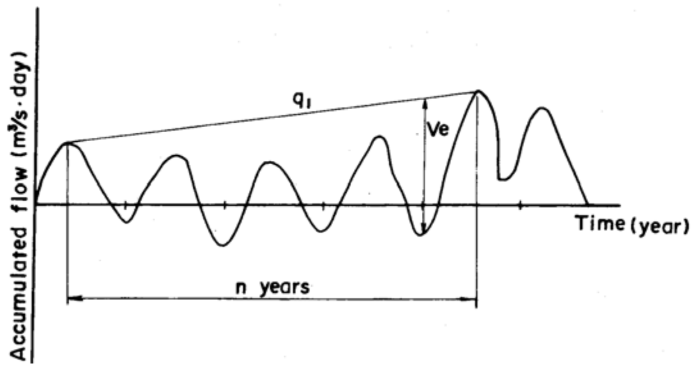


Figure 10-9 Mass Curve (Carry-over reservoir)

2) Peak and base demand

Reservoir type power plants are mostly planned as power sources supplying for peak demand mentioned above, however the type may be rarely planned for the load as explained below.

The example shown in Figure 10-10 is a reservoir type power plant to supply for base demand in a system mainly composed of hydro power plants. This discharge "q_1" is more than the mean flow during the dry season. In the wet season, it is operated to supply for peak demand controlling discharge "q_2". This operation pattern is taken because in the wet season run-of-river and pondage type power plants have a small regulating capacity and therefore are operated 24 hours a day to supply for base demand, and the reservoir type plants supply the peak demand. Conversely, in the dry season, pondage type plants are operated to supply for peak load, and reservoir type plants supply for base demand using a certain volume of flow stored during the wet season.

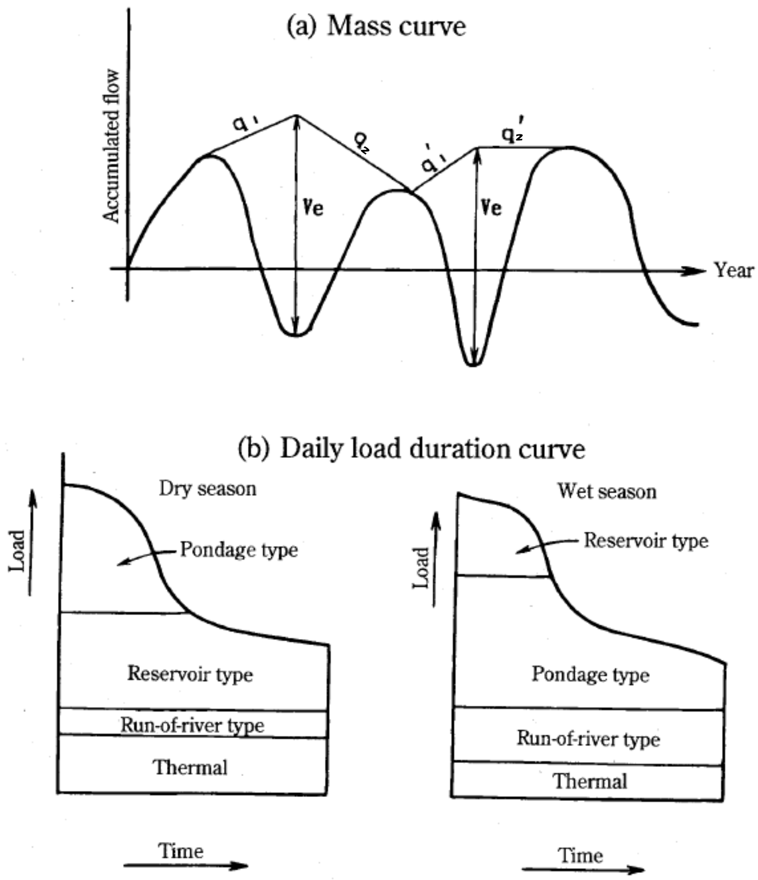


Figure 10-10 Mass Curve and Daily Load Duration Curve

(2) Optimization study of scale of development

The study method is the same as that described in 10.1.5. The following are the main points.

1) Study of dam site, powerhouse site and waterway route

The dam site selected in the reconnaissance study is an approximate location based on a topographic map of scale of about 1:50,000. The topography of the dam site is, therefore, confirmed using an approximately 1:5,000 - 1:10,000 scale topographic map prepared before conducting feasibility study. The suitability of the site is confirmed from geological surveys such as geological reconnaissance and drilling, etc. Where there are multiple candidate sites for the dam, these sites are compared from technical and economic viewpoints and the site is then decided.

2) Preparation of storage-capacity curve, estimation of sedimentation, and setting sedimentation level

The storage-capacity curve is prepared using an approximately 1:5,000 - 1:10,000 scale topographic map.

Sedimentation volume is estimated using the study results described in Chapter 9, and a sedimentation level is set corresponding to the sediment volume. Generally, sedimentation for 100 years is used to calculate the sediment volume. Of the sediment brought into a reservoir by river flow, only a portion may be trapped and retained in a reservoir. The trap efficiency of a reservoir is considered in estimating the sedimentation volume. When reservoirs exist upstream, the effect of trap efficiency of the upper reservoirs is also taken into consideration. The estimated sedimentation volume may be decreased when there are facilities to discharge the sediment from a reservoir such as bottom outlets.

3) Calculation of output and energy generation

Output and energy are calculated from the head and plant discharge determined by the reservoir operation, turbine efficiency determined from head and discharge fluctuation. A rule curve prepared using the mass curve and/or Dynamic Programming (DP) is used in reservoir operation.

4) Study of high water level (height of dam) and active storage capacity

(a) Low water level is tentatively set based on the sedimentation level, intake shape and velocity of flow.

(b) Figure 10-11 is a schematic of studies conducted with different high water levels and active storage capacities. The lower limit of the low water level is determined by the sedimentation level. The limit of the high water level is determined by topography, geology, compensation and resettlement, etc. Alternative plans with low and high water levels within this range, with different effective storage capacities, are prepared. Economic comparison is made based on those plans, and the optimum high water level and effective storage capacity are determined. In the case of a project that involves a huge area of inundated land and resettlement, but the unit compensation and resettlement cost are relatively low, the difference may not be overly remarkable in an economic comparison. In this case, it might be important to determine the high water level politically, such as for the acquisition of alternative land etc. This issue must, therefore, be fully discussed with the authorities concerned.

(c) Maximum plant discharge which is used tentatively in this study is calculated from the following equation.

$$Q_{max} = \frac{Q_f \times 24}{T}$$

where,

Q_{max}	: Maximum plant discharge (m^3/sec)
Q_f	: Firm discharge (m^3/sec)
T	: Peak duration hours (hr)

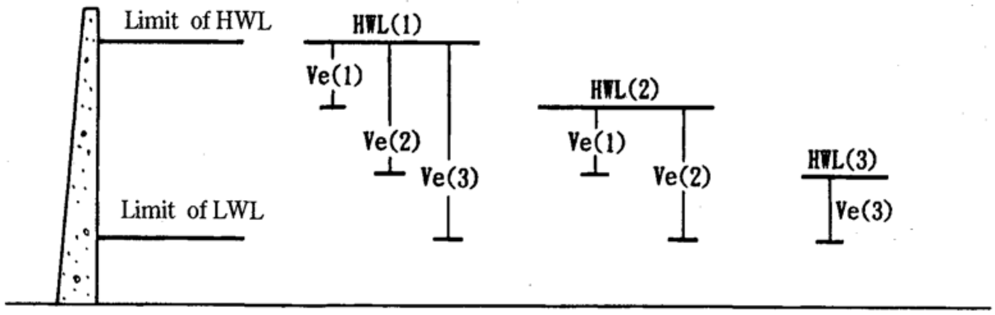


Figure 10-11 Study of High Water Level and Effective Storage Capacity

5) Study of maximum plant discharge

Economic comparison is made by altering the maximum plant discharge against the selected high water level and active storage capacity, and the optimum maximum plant discharge is then determined.

High water level, active storage capacity and maximum plant discharge may be studied together using matrix.

6) Water use of tributaries

In some cases, it is technically possible to draw water from an adjacent gully, tributary or river. Economic comparison is made of the cost required for such intake facilities and the benefit gained from the increased output and energy. Judgment is made whether such an intake facility should be constructed or not from the study.

7) Economic comparison considering the benefit of other sectors

When the dam is a multi-purpose project which includes irrigation, flood control, etc., the benefits combined with power generation and other sectors are considered in the economic analysis.

8) Determination of scale of development

As the development scale of dam and maximum plant discharge are determined, the maximum output is determined.

9) Example of determining the scale of development

Figure 10-12 shows examples of comparisons made with high water levels between 155 – 170m and active storage capacities between 150 x 106m3- 330 x 106m3. The active storage capacity is altered for each high water level, and the optimum active storage capacity for that high water level is obtained. This shows that the values of (B – C) and B/C rise as the high water level rises.

In this project, however, when the high water level exceeds 165m, the problem of resettlement of local residents arises. As this problem affects the project feasibility, discussions were held with the national government and 165m was subsequently selected as the high water level. In this case, the optimum active storage capacity is 240 x 10^6m^3. Figure 10-13 provides examples where the maximum plant discharge is altered. The high water level and the active storage capacity are determined by the study described above, and then the maximum plant discharge is changed. From this, the plant discharge of 214m^3/sec was determined as optimum with the maximum output set at 165MW.

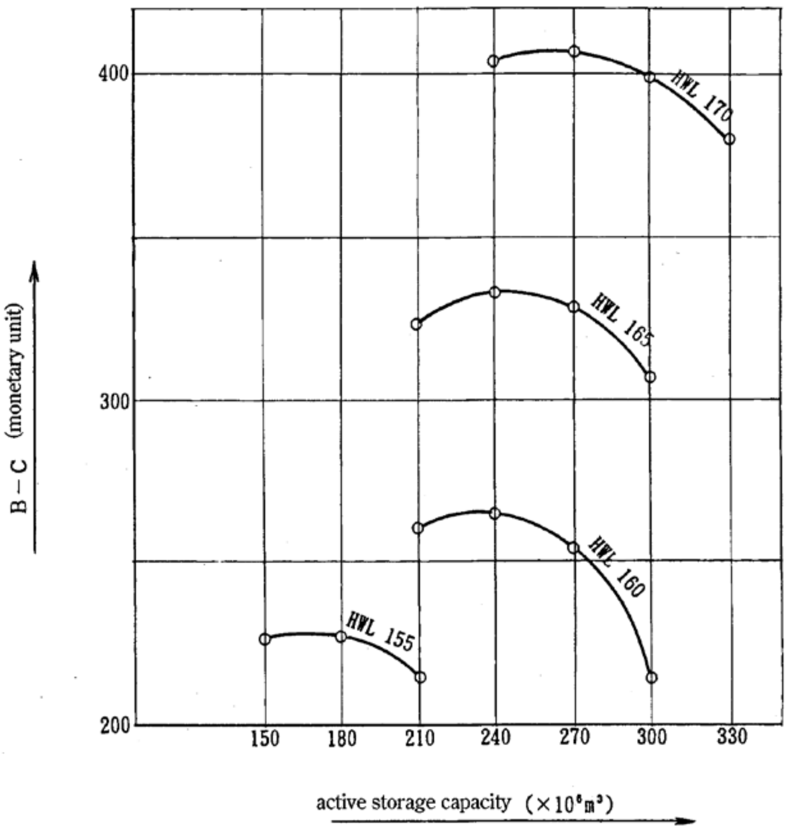


Figure 10-12 Study of High Water Level and Active Storage Capacity

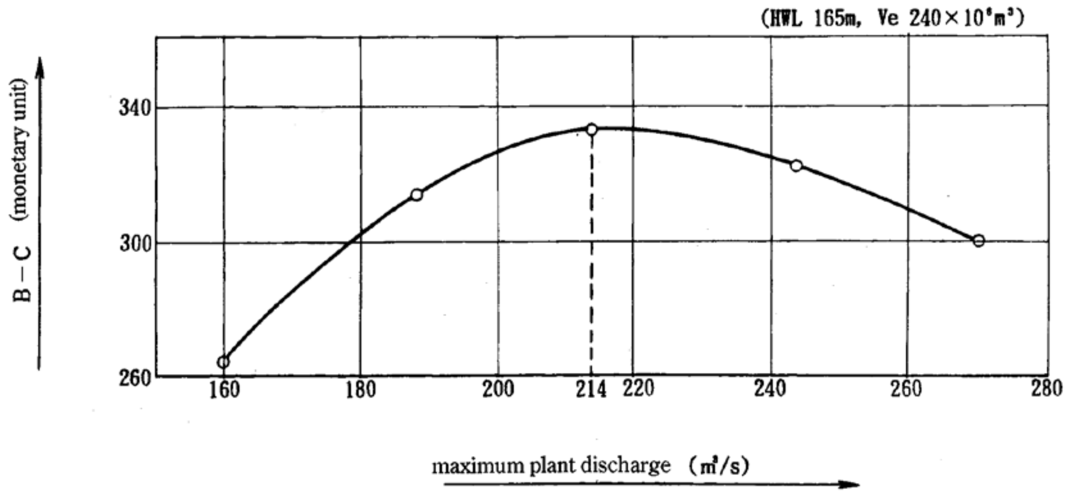


Figure 10-13 Study of Maximum Plant Discharge

(3) Reservoir operation

1) Preparation of rule curve based on mass curve

Reservoir operation based on a mass curve is done on the assumption that the runoff pattern is assumed to be known. However, in actual practice, as it is very difficult to predict the future runoff pattern, an operation rule curve is prepared and used to determine the plant discharge.

The following is an explanation of a rule curve prepared based on a mass curve. The coordinate of the rule curve shown in Figure 10-14 represents the storage volume on its ordinate and the time on its abscissa. There are three operation lines of V_{max} line, V_{min} line and V_u line. The discharges of "Q_u" and "Q_ℓ" are set between the V_{max} line and the V_u line, and between the V_u line and V_{min} line respectively. The reservoir is operated so that the storage volume runs along the V_u line. Q_ℓ indicates the minimum plant discharge, and generally firm discharge is used. However, when the storage volume becomes lower than the V_u line, this Q_ℓ is used to ensure the storage volume does not drop below the V_{min} line (LWL). Q_u is discharge larger than Q_ℓ, for example, the annual mean flow. When the storage volume is higher than the V_u line, Q_u is used to prevent spill of water from a reservoir. The reservoir is operated by using Q_ℓ and Q_u to ensure the storage volume is close to the V_u line. Q_u and Q_ℓ are not for peak discharge, however daily mean discharge; but the water level instead of storage volume on its ordinate also can be used.

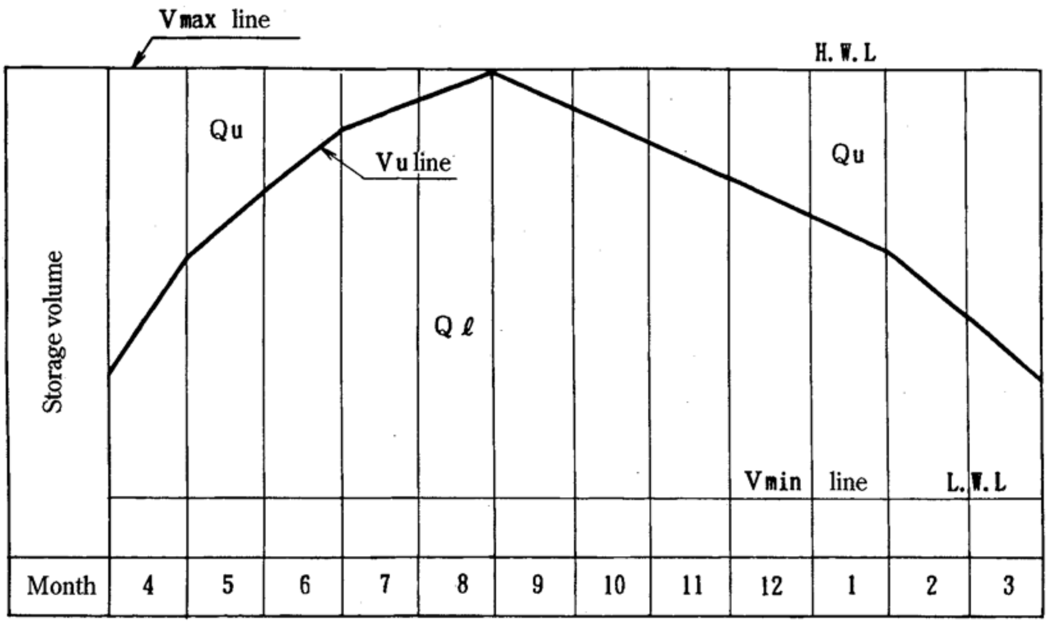


Figure 10-14 Reservoir Rule Curve

2) Preparation of rule curve based dynamic programming

Dynamic programming (DP) is widely used as an increase of operation speed of computers for solving extreme value problems such as maximization or minimization of objective function.

Since many different varieties of methods and models using DP for optimizing reservoir operation rule have been introduced in many literary documents, it is impossible to introduce all of them.

DP applied for deriving a optimal reservoir operation is classify broadly into two types, deterministic DP and stochastic DP.

It will initiate discussion in concluding how much energy we should expect from reservoir type hydropower during planning stage. To derive an optimal reservoir operation rule based on historical inflow data is equivalent to presuppose that historical inflow pattern will be repeated in the future. Therefore, an optimal reservoir operation rule bade on a mass curve belongs to deterministic method.

On the contrary, inflows in the future cannot be same as those in the past, and a reservoir operation rule based on deterministic also cannot be an optimal one.

(a) Approach to reservoir operation rule

As shown in Figure 10-15, it can be solved by using DP which route, from the initial storage capacity A to the final storage capacity B, gives the maximum energy.

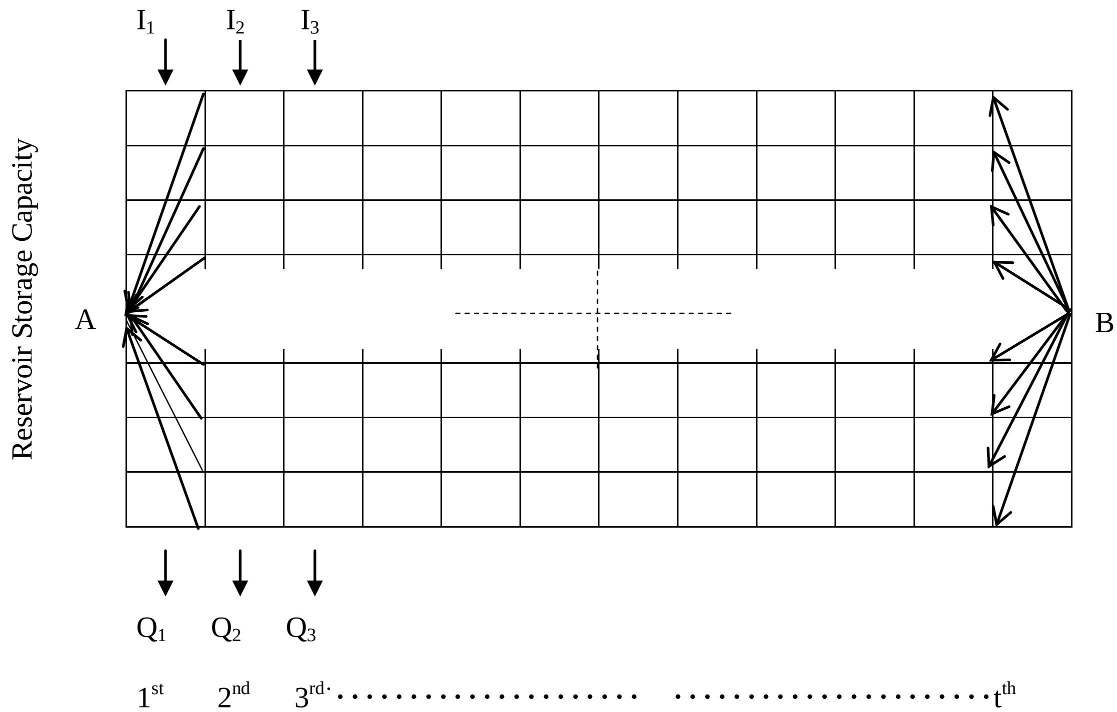


Figure 10-15 Optimal Rule of Reservoir Operation (Backward)

(b) Formulation

The above problem can be formulate as following

i) Continuity equation (water balance equation)

$$S_{t+1} = S_t + I_t - R_t - L_t$$
$$= \max [S_{min}, \min (S_t + I_t - Q_t - L_t, S_{max})]$$

where,

S_t : Reservoir storage capacity at the beginning of period t

I_t : Inflow during period t

Q_t : Discharge through turbine(s) during period t

L_t : Water losses (overflow, evaporation, seepage) during period t

ii) Energy

$$E = P \times T \times D \quad \text{(kWh)}$$

P : Output (kW)

T : Generating time (hours)

D : Number of day per each month (day)

iii) Performance (objective) function (total energy for t months)

$$F = \sum_{t=1}^{t\max} Et \rightarrow \text{maximize} \quad \text{(maximize total E by controlling Q during t months)}$$

t : Month

tmax : Last month

iv) Constrain conditions

$Qmin \leq Q \leq Qmax$

$Smin \leq St \leq Smax$

Qmin : Minimum discharge or minimum release

Qmax : Maximum Discharge

Smin : Minimum storage capacity

Smax : Maximum storage capacity

(c) Optimization

The reservoir storage capacities for each month, which give the maximum value of the performance function, derived based on DP under the above-mentioned conditions is an optimal reservoir operation route. The optimal reservoir operation route is not a rule, and is a just route which gives maximum total energy for historical inflows.

(d) Optimal reservoir operation rule

A reservoir operation rule have to indicate common monthly operation policy to every year for each month in spite that monthly inflows and reservoir storage capacities in a certain month vary every year, and a optimal rule is a operation policy which maximize a project value such as total energy. There are many kinds of optimal rule ways, and a simplest example is introduced here. The optimal reservoir operation, which gives the maximum energy, can be obtained by changing the above-mentioned performance function as following. The concept is shown in Figure 10-16.

$$F = \sum_{m=Jan,n=1}^{m=Dec,n\,max} E_t \rightarrow \text{maximize (maximize total E by controlling Q during n years)}$$

m : Month (from January to December)

n : Year

nmax: Total year

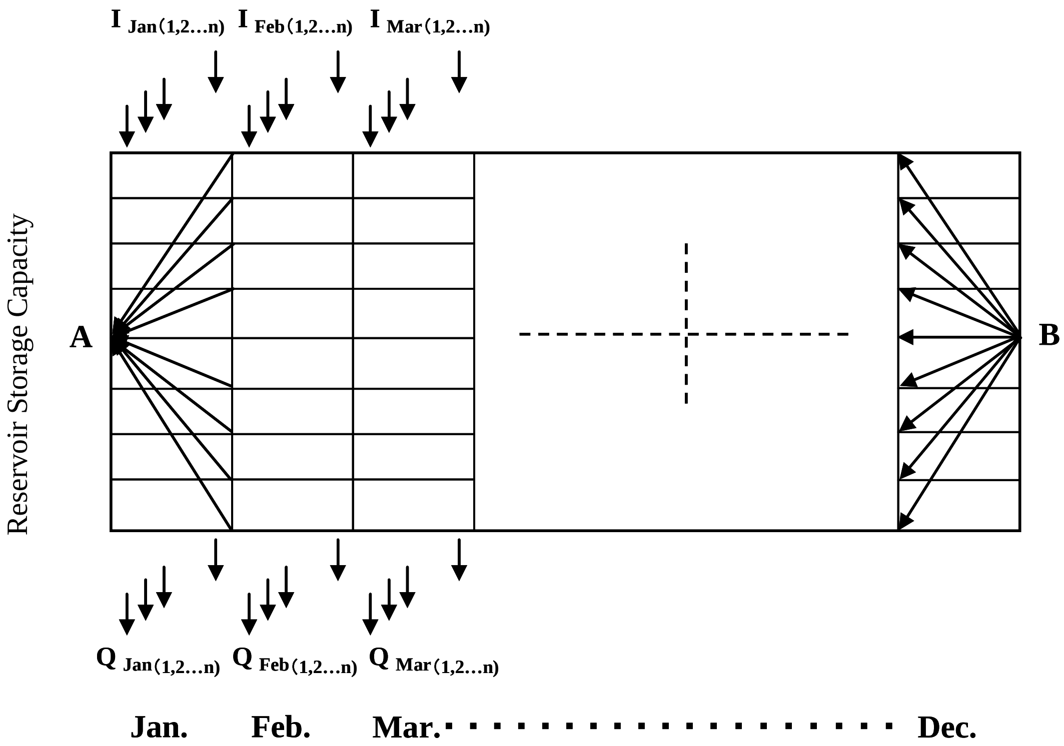


Figure 10-16 Optimal Reservoir Operation Rule

(4) Study of timing of implementation

It is important to install turbine and generator (hereinafter referred to as the equipment) corresponding to the increase of power demand from an economic view point of the project.

When foreseen that the project scale is too large and that not all the facilities will be used if developed all at once, development in stages with two or more equipment should be studied.

The following are methods for development in stages.

Method - 1: In the first stage, the dam, waterway, powerhouse and other civil structures are constructed and the required number of equipment are installed, and the remaining equipment corresponding to increase of demand are installed in the second and subsequent stages.

Method - 2: In the first stage, only the necessary civil structure such as dam and waterway corresponding to the number of equipment are installed, and in Stage 2 and subsequent stages additional waterway required for the additional equipment is constructed and the equipment is installed. The intake structure of additional facilities is generally constructed in the first stage.

10.2.4 Pondage Type

(1) Required supply capability

The pondage type power plants are used as a power source for peak demand in the electric power system, The supply capability is expressed as power output corresponding to fully regulated flow of firm discharge as shown in Figure 10-17 (a) . In small scale and isolated systems, the supply capability corresponding to the load may be used as shown in Figure 10-17 (b). If there is only a minimal difference in the economic viability between the run-of-river type and pondage type, it is advisable to select the pondage type as it provides more flexible operation.

(2) Optimization study of scale of development

1) Study on dam site, powerhouse site and waterway route

The followings are as described in 10.2.3, and therefore refer to that Section.

- To study of dam site, powerhouse site and waterway route.
- To prepare storage-capacity curve, estimate sedimentation volume and set sedimentation level.

2) Calculation of output and energy generation

The power and energy are calculated from the turbine efficiency and available discharge taking into account the effective head determined by the operation of pond.

The water level fluctuates between the high water and low water in daily or weekly cycle. In many cases, as the available drawdown is small, a fixed value of water level is used to calculate the power and energy outputs in this Manual.

3) Study of high water level, active storage capacity and maximum plant discharge

(a) Study of high water level (height of dam)

Studies conducted by changing the high water level and active storage capacity are the same as that described in 10.2.3(2). Generally, however, the available drawdown of a regulating pond is small and a conspicuous optimum high water level may not be obtained as in the case of reservoir type. In this manual, therefore, the active storage capacity and high water level are set as follows.

- The storage capacity required to regulate the firm discharge into maximum plant discharge during peak duration hours (Refer to Figure 10-17 (a))

$$V_e = (Q_{max} - Q_f) \times T$$

where,

Q_{max}	: Maximum plant discharge (m^3/sec)
V_e	: Active storage capacity (m^3)
Q_f	: Firm discharge (m^3/sec)
T	: Peak duration hours (hr)

Maximum plant discharge is calculated by the following formula.

$$Q_{max} = \frac{Q_f \times 24}{T}$$

- The storage capacity corresponding to pattern of demand (Refer to Figure 10-17 (b))
  When used as supply capability to small scale hydro power and isolated system, Q_{max} is calculated based on the demand of the system during peak duration hours.

$$V_e = (Q_{max} - Q_f) \times T$$

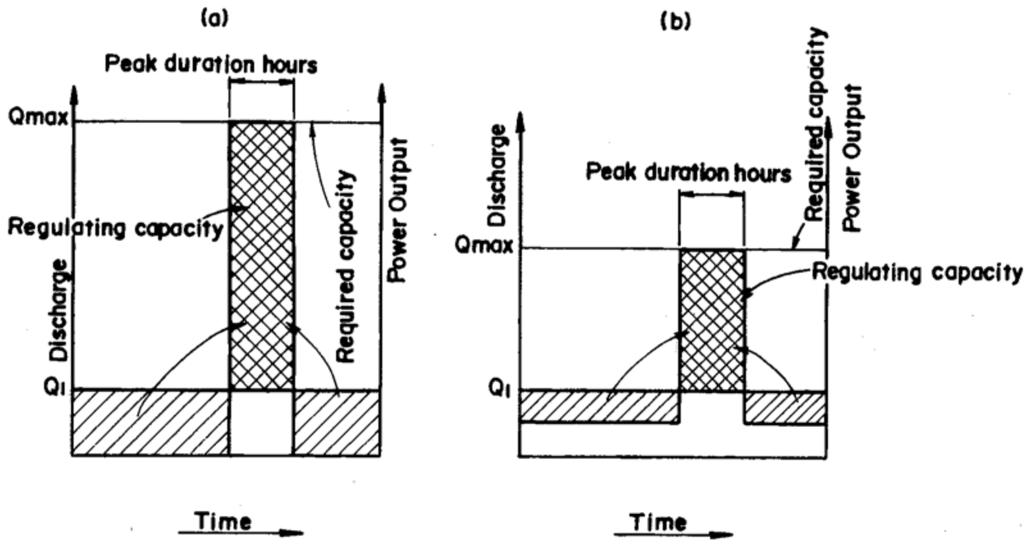


Figure 10-17 Storage Capacity of Pond

- Other than this method to determine the storage capacity, another method is to set the storage capacity to completely regulate all inflow into peaking discharge. The details of this method are omitted here.

(b) Study of maximum plant discharge

The high water level and effective storage capacity determined above are fixed, and economic comparison is conducted by changing the maximum plant discharge to arrive at the optimum maximum plant discharge. There is a method to study high water level, effective storage capacity and maximum plant discharge at the same time.

4) Refer to 10.2.3 (2) to draw flow from adjacent gullies, tributaries or rivers
5) As the scale of dam and maximum plant discharge are now determined, the maximum output is established.
6) Refer to 10.2.3 (4) for timing of implementation.

Reference of Chapter 10

[1] Guide Manual for Development Aid Programs and Studies of Hydro Electric Power Projects, New Energy Foundation, 1996

[2] Feasibility Studies for Small Scale Hydropower Additions, US Army Corps of Engineers, 1979

Chapter 11

Design of Civil Structures

Chapter 11 Design of Civil Structures

The design is executed with a step-by-step approach corresponding to the step of the project. It means that the accuracy of the design is gradually improved with applying more precise design method in each process.

In this chapter, the design for the feasibility study (F/S) or the pre-feasibility study (Pre-F/S) is described. Therefore it is noted that these methods of the design are not always valid for the optimization or definite design (D/D).

The structural design method is classified into an elastic method and a limit state design method which considers the limit states of the structure.

In the elastic method, stipulated is that the stress of the structure is below the allowable stress of the material used for the structure, which involves an inclusive safety margin. It has been applied in many projects due to its simplified manner.

The safety of the structure is also evaluated with the limit state design method by comparing the structural condition under design loads to the limit state of the structure. The safety margin is ensured by providing individual margins in the definition of the loads, estimation of bearing strength and loads, and safety margins for those other items.

Although the latter design method has recently been predominant in the structural design, the adoption of the design method for F/S, Pre-F/S and D/D is justified taking the stage of the project into account. For example, the elastic method is suitable to F/S, Pre-F/S and a determination of basic dimensions of the major structures. The limit state design method may be suitable to D/D.

11.1 Dam and Appurtenant Facilities

11.1.1 Dam

Dams are classified into those creating reservoirs or ponds and weirs for intake of water. Dams are constructed to store and regulate the river flow for a reservoir type, a pondage type and a pumped storage type power plant. Weirs are low dams which convey the river water into the headrace of run-of-river type power plants. This section describes dams for storage of water.

Intake weirs are described in 11.2.5.

Dam
- Storage dam – reservoir type, pondage type, pumped storage type
- Intake weir – run-of river type

(1) Selection of dam axis

Normally, the dam axis is comprehensively determined from the topography, geology, excavation volume, dam volume, and difficulty of construction, spillway layout, and a river diversion scheme.

The dam axis of a gravity dam is selected so as to situate the toe of the dam on the sound foundation, considering the higher stress in the toe of the dam. Linear alignments of the axis are predominant. However a bend of the axis is sometimes applied so as to fit the dam to an adequate abutment of the foundation.

The axis of an arch dam, which has a convex shape in upstream direction, is laid entirely on the sound foundation.

A major factor in determining a dam axis of a fill dam is the control of seepage through the foundation. The topography for the spillway site is another major factor when the spillway capacity is large. The dam axis of a fill dam is linear in many cases. An arch shape may be adopted at the protruding terrain on the upstream side. The radius of the arch curvature varies from some hundred meters to some thousand meters, and is almost linear.

(2) Determination of dam height

The dam height is defined by the height between the dam foundation and the dam crest of the non-overflow section. It is shown in Figure 11-1 for each dam type.

The elevation of the dam crest of non-overflow section is determined so as to ensure the free board involving the wave height due to wind and earthquake and a certain margins above the reservoir water surface which corresponds to the reservoir volume of flood control, water usage and sediment. The cover depth on the crest of rockfill dams without impervious core can be counted for the dam height.

(3) Selection of type of dam

Dams are broadly classified into concrete dams and fill dams as shown in Figure 11-1. Concrete dams are classified into gravity dams and arch dams by their structural features. The structure of the former type is further classified into a gravity dam, a hollow gravity dam and a buttress dam. The latter two types are rarely adopted due to the complexity of the construction works which incur the problem from an economical point of view. Arch dams are classified into a thin arch dam and an arch-gravity dam.

Fill dams are broadly classified into a homogeneous fill dam, a zoned fill dam, and a fill dam with an upstream impervious membrane depending on the dam structure.

Homogeneous fill dams are called earth dams. A zoned fill dam is called a rock fill dam when its pervious zone is formed with rock materials. Earth dams are rarely applied to dams higher than 15 m.

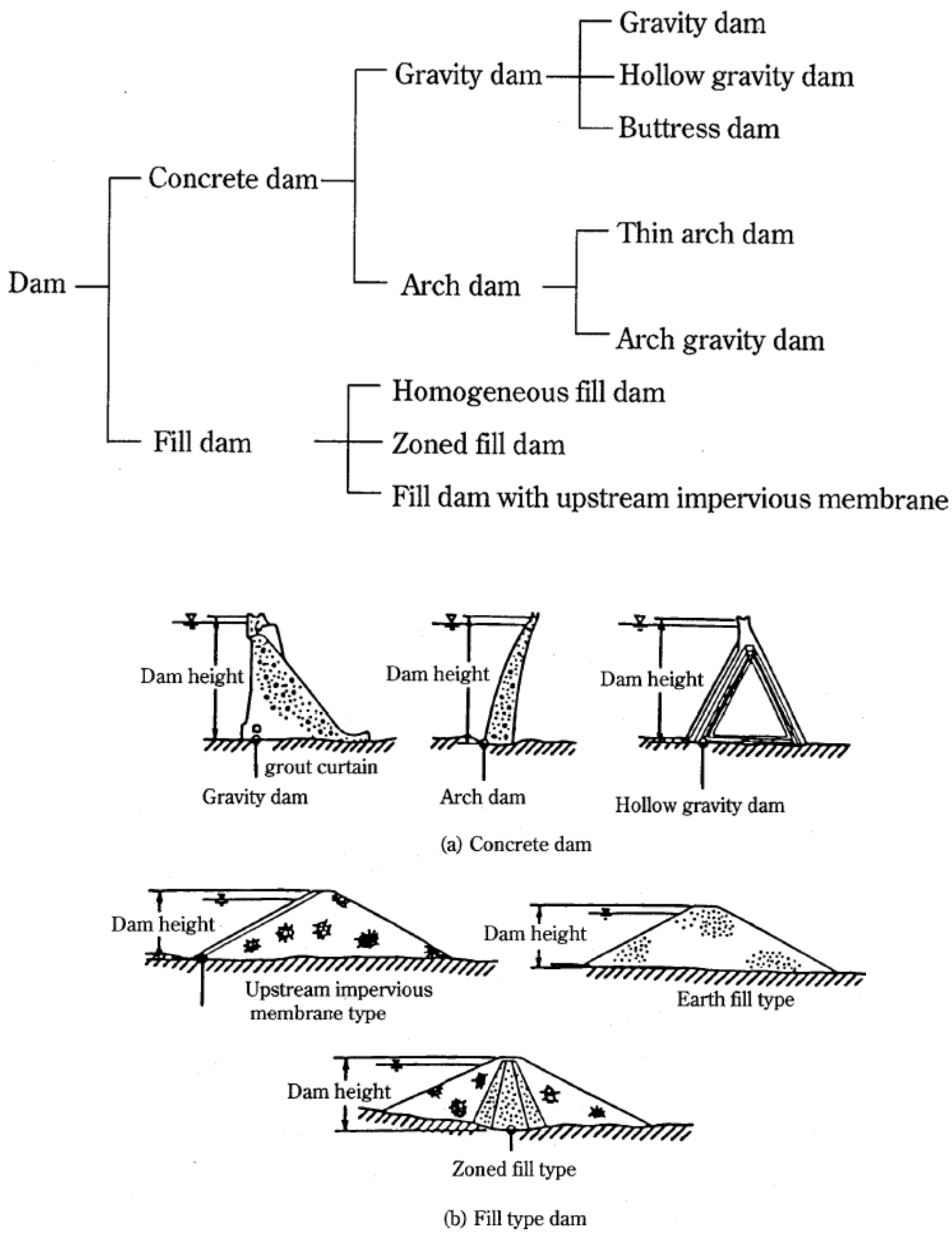


Figure 11-1 Concrete Dam and Fill Dam

Each type of dams is generally applicable to a planned dam site. In only some cases the dam type is restricted. Depending on the foundation condition, however, an arch dam, a gravity dam and a fill dam are predominant for sound foundation, moderate foundation and relatively weak foundation, respectively. The combined dam of a concrete gravity dam and a fill dam may be suitable to the specific topographic and/or foundation condition.

In the feasibility study stage, potential types of dams are designed for a comparison. The type and scale of the dam is then determined based on the technical adaptability and economy.

1) Concrete dam

The external force acting on a concrete gravity dam such as hydraulic load of the reservoir is transmitted to the foundation rock through the dam. This is resisted by the dead load of the dam concrete and the shear strength of the foundation rock. The gravity type is applicable to many sites, as it features simple configurations and less topographic restrictions, and the flood flows may spill over the dam. However, stability against sliding may be a problem where a weak stratum exists in the foundation rock, such as fault with almost horizontal dip. Also, as it requires a large volume of concrete, a large dam may provide problems of getting aggregates for concrete and cement transportation.

Concrete gravity dams are frequently constructed by a layered concreting method such as a method of RCC (Roller Compacted Concrete) or RCD (Roller Compacted Dam), which shows higher cost performance of the construction compared with a conventional Column Concrete Method.

The structures of a hollow gravity dam and a buttress dam are similar to a gravity dam. Their economy is, however, no longer beneficial due to the complicated construction works involved.

The external force acting on a concrete arch dam is transmitted to the abutments on both banks using the arch action of the dam, which is resisted by the shear strength of foundation rock.

An arch type is generally selected when the foundation rock is sound and the ratio of the valley width to the dam height is approximately 3 or less. Stability against sliding may be a problem where a weak stratum exists in the abutments, for instance, a fault with a vertical dip and a strike in the upstream and downstream direction. Regarding an arch gravity dam, its cross section is made thick so that the load per unit area on the foundation rock becomes small to mitigate the strength required of the foundation rock.

The construction of arch dams is usually executed by a Column Concrete method due to its narrow dam width, providing the restriction on construction, and requirements for high concrete strength.

2) Fill dam

Most of the earth dams are built with fine materials of small permeability. Of three types of fill dam with the same height (Refer to Figure 11-1), an earth dam has the longest foundation seepage length. Since the dam is flexible against deformation, it is suitable where the foundation is soft with fewer cracks. However, pore water pressure which generates during construction does not dissipate easily due to its thick impervious zone. Therefore, construction is significantly affected by rainfall. Also, pore water pressure remains at the upstream side when the reservoir water level recedes quickly, thus providing less stability of the dam. Consequently, this type is suitable for a small dam subject to less fluctuation of reservoir water level.

Regarding a zoned fill dam, an impervious zone of earth material is constructed in the center of the structure with semi-pervious zones and pervious zones toward the outsides. The pervious zones, which are the major part of the dam, are formed with highly permeable rock materials and gravel or sand which are mainly quarried near the dam site. A zoned fill type is applied to many large fill dams because of its flexibility to use several types of materials or by a combination thereof, and the stress can be dispersed broadly into the dam and foundation. The zoned fill dam is also classified into a fill dam with a center impervious core and a fill dam with an inclined impervious core depending on the location of an impervious zone; that is, in the center or towards the upstream. A fill dam with a center impervious core is generally selected because it provides better dam stability. A fill dam with an inclined impervious core may be selected where heavy rainfall is expected since the downstream rock zone can be constructed independently prior to core embankment. Also, the center impervious core can be asphalt concrete instead of earth material.

Regarding a fill dam with a facing membrane, the upstream surface of the dam embankment of coarse-grained material is covered with highly impermeable concrete or asphalt concrete. They are referred to as CFRD (Concrete Facing Rockfill Dam) and as AFRD (Asphalt Facing Rockfill Dam) respectively. Since most dam materials provide stability, the volume of the dam is small with less construction restrictions. Careful design and construction of the joints are required when concrete is employed for an impervious material. Asphalt concrete provides good flexibility and adaptability in response to long-term deformation, which requires in particular permeability quality control.

In a combined dam, two or more types of dams are constructed as a continuous structure across a wide river of which foundation shows a variety in mechanical properties. Generally, a combination of a gravity dam and a fill dam is adopted as a combined dam as shown in Figure 11-2. Where the river is wide, spillway and waterway structures may be incorporated in the concrete and a remaining section is a fill dam. The same concept is applied in the design and construction of these combined structures as an individual one.

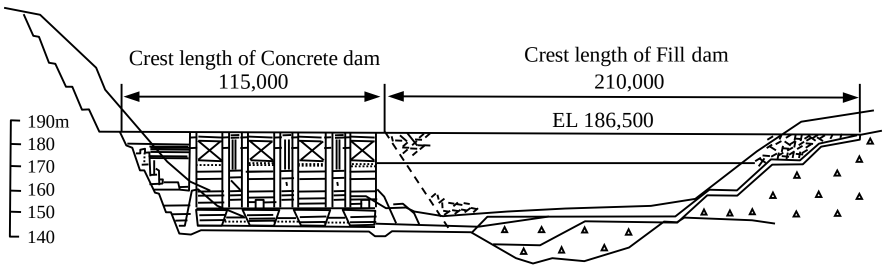


Figure 11-2 Combined Dam

(4) Design load

Design loads to be considered according to each type of dam are described in Table 11-1. Taking a pseudo static method, an inertial force during earthquake is a horizontal force and acquired by multiplying the dead load of the dam and the design seismic coefficient which is evaluated based on the regional seismic risk, a dam type and foundation condition. The coefficient of 0.10 to 0.15, 0.12 to 0.18 and 0.20 to 0.30 are usually adopted to a concrete gravity dam, a rockfill dam, and an arch dam respectively according to a Japanese standard.

A pseudo static method is suitable to evaluate the dam stability in F/S stage. It is recommended that where a large seismic load is expected on a large dam, the dam stability be evaluated using a dynamic analysis method.

Table 11-1 Design Load Considered for Dam Design

Loads	Dam type		
	Gravity dam	Arch dam	Fill dam
Dead weight	√	√	√
Hydro static pressure	√	√	√
Inertial force due to earthquake	√	√	√
Hydro dynamic pressure due to earthquake	√	√	-
Pore pressure or Uplift pressure	√	√	√
Thermal load	-	√	-
Silt pressure	√	√	-

(5) Design of concrete dam

1) Gravity dam

A gravity dam is designed basically as a two-dimensional structure. The criteria concerning its structural stability are described below.

- Tensile stress is not generated in the vertical direction on the upstream surface of the dam.
- The contact face between concrete and foundation, and its surrounding area are stable against sliding by shear force.
- The stress in the dam does not exceed the allowable value.

The first item stipulates that the resultant force of all loads on the dam is confined within the middle third of the bottom of the dam. It can be confirmed by taking all horizontal loads and vertical loads into account.

The second item is confirmed that the factor of safety of sliding, *n* is larger than a certain value by using Henny's formula shown below.

$$n = \frac{\tau_0 \cdot L + f \cdot V}{H}$$

where,

n : Factor of safety of sliding (≥ 4)
H : Horizontal loads
V : Vertical loads

τ_0 : Shear strength
f : Internal friction angle
L : Length of dam base

The last item stipulates that the maximum stress of the dam at the downstream toe σ is less than the allowable strength of concrete. σ is estimated by the following formula.

$$\sigma = \sigma_D (1 + m^2)$$

where,

σ : Vertical stress using beam theory, m: downstream slope, 1 to m

The above-mentioned are the standard in Japan. As for reference, the USBR standard of the United States is as follows. It is noted that because the loading conditions of "Usual/ Unusual/ Extreme" are different from those of the Japanese standard, a simple comparison shall not be done.

- Compressive stress in dam should be less than the allowable compressive stress.
- Compressive stress at the upstream end of a dam, calculated by dam stability without uplift pressure, should be more than the minimum compressive stress, which is determined by uplift/seepage pressure there minus the allowable tensile stress.
- A sliding safety factor should be more than the specific safety factor.

Note: The specific safety factor is defined as 3 for Usual, 2 for Unusual, 1 for Extreme conditions respectively. Allowable stress is determined by the strength divided by the specific safety factor.

2) Arch dam

An arch dam is designed basically as a three-dimensional structure. The conditions concerning its structural stability are described below.

- The stress in the dam shall not exceed the allowable value,
- The contact surface of the dam and foundation, and its surrounding area shall be stable against sliding caused by shear force.

Although the design load is the same as the gravity dam, a doubled value is applied for the design seismic intensity. The trial load method, which structurally divide the arch dam into beam elements horizontally and cantilever elements vertically, was used before for the design. A three-dimensional finite element method is widely used at present with the development of computer technology for the design of arch dams. It can incorporate directly the complicated configuration of the dam and its foundation.

3) Foundation

The dam foundation is stable against the loads transmitted through the dam and ensures adequate water-tightness against the seepage from the reservoir.

Where the above requirements are not ensured, the foundation treatment is necessary. The replacement with concrete for the improvement of the strength and the deformation resistance is popular as well as the grouting for the improvement of the deformation resistance and the water-tightness at the contact area of the dam and the foundation.

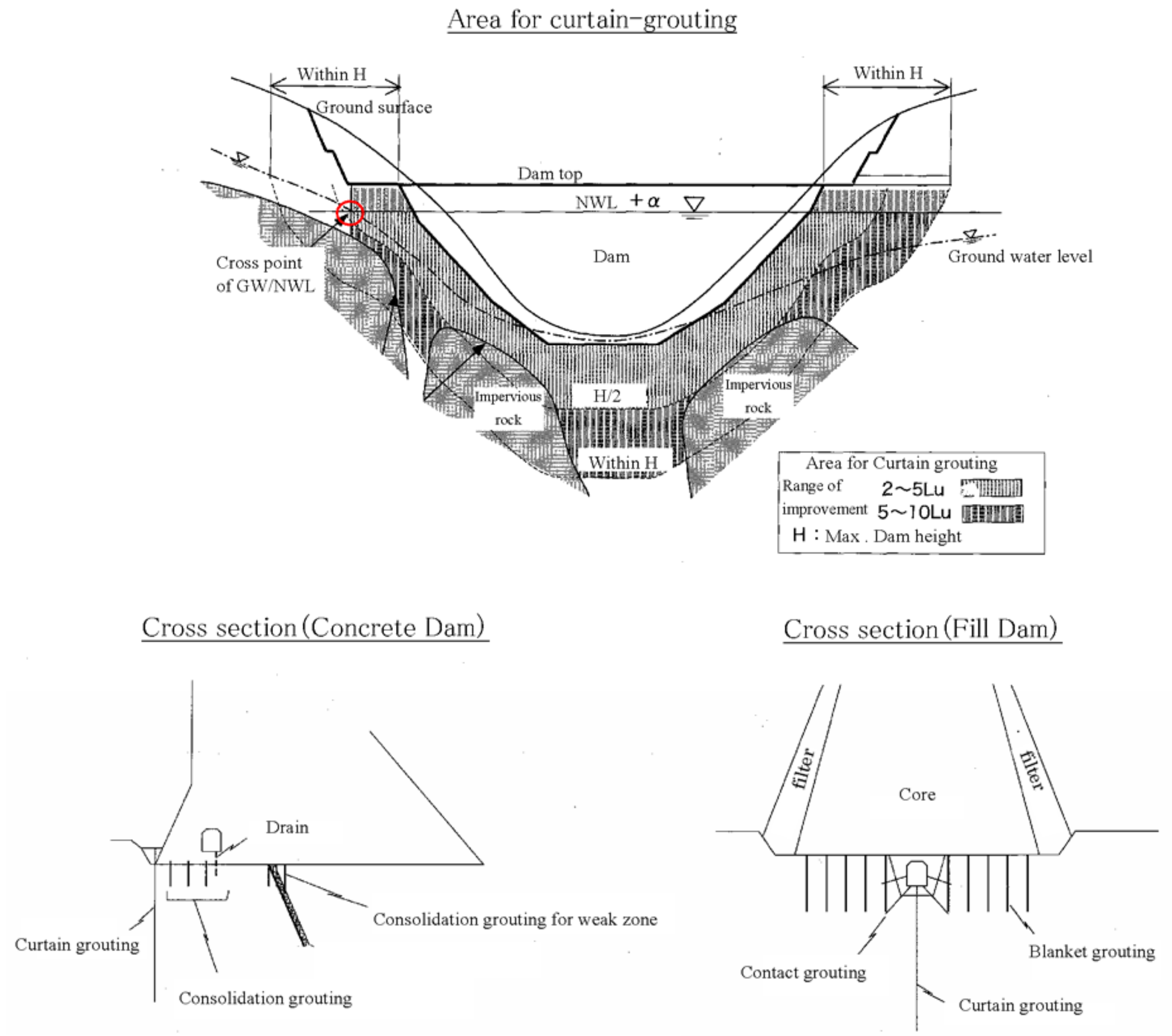
The grouting is categorized as consolidation grouting, blanket grouting and curtain grouting, aiming to improve the deformation resistance and water-tightness.

Consolidation grouting of the foundation of a concrete dam is to prevent deformation, consolidate loosened foundation due to excavation, and geologically defective area, and to prevent seepage of the foundation at the contact surface of concrete as well as to a certain depth. Grouting holes are generally planned in a 4 to 5m grid in the feasibility study.

Curtain grouting as shown in Figure 11.3 is applied to the foundation to improve its water tightness as well as to reduce the uplift force. Curtain grouting is applied in such an area, shown below, to be referred to Figure 11-3. Enhancing the effect of curtain grouting, an auxiliary curtain grouting may be applied. A grouting gallery is usually provided to carry out the curtain grouting independently of a dam work schedule as well as more effectively under the dam load.

- Permeability of the foundation exceeds the target design value.
- Ground water level of the foundation is lower than the reservoir water level. (normally surcharge level)

Blanket grouting is applied to lengthen the seepage length near the foundation surface, in particular on the contact area of core material of a fill dam.



a) Area of Curtain grouting

Figure 11-3 Foundation Treatment

The target design value after curtain grouting is generally 2 to 5 Lugeon. The grouting hole spacing to acquire the specified design value depends on the foundation rock conditions. It is recommended in the feasibility study to predetermine the spacing of holes and injection volume.

It is then recommended that the spacing of grout holes and design injection volume be determined in the detailed design stage based on the field grouting test results.

The following table shows an example of the criteria of the foundation treatment in Japan.

1) Consolidation grouting	
Spacing	3m to 6m grid
Treated area	Upstream toe to drain holes for gravity dams Contact area of dams and foundation for arch dams
Treated depth	5 m deep from the foundation surface
Quality	Permeability of 5 Lugeon of test holes
Timing	After placing of several layers in order to obtain higher performance of treatment
2) Curtain grouting	
Spacing	3m to 6m apart on the dam axis Another row may be provided as an auxiliary curtain
Treated area and quality	2 to 5 Lugeon up to H/2 in depth from the foundation surface 5 to 10 Lugeon from H/2 to H in depth from the foundation Where, H is the dam height. The area of the foundation with enough impermeability is excluded. The area is extended to the intersection of the high water level and the underground water level in both abutments, referred to Figure 11-3.
Timing	After reaching enough height of the dam
3) Blanket grouting	
Spacing	3 to 6m grid
Treated area	Contact area of core material and foundation
Quality	5 to 10 Lugeon
Treated depth	5 to 10m deep from the foundation surface
Timing	After excavation, Cover rock of 0.5m thick is necessary to prevent leakage

(6) Design of fill dam

1) Structural characteristics of fill dam

The slope gradient, crest width, and thickness of each zone, etc., of a zoned fill dam must satisfy the stability against sliding and seepage flow. When determining these, the conditions such as available types of material, characteristics, available quantity, and construction and control methods are considered. Figure 11-4 gives an example of structural characteristics and a typical section of a zoned fill dam. Also Appendix A 11-1-1 shows the examples in the world and Japan. Since the materials used for earth dams are finer than those for zoned fill dams, the shear strength is small and generates higher pore pressure. The slope of an earth dam is accordingly gentler than that of a zoned fill dam.

The gradients of the slope of a fill dam are evaluated by a finite plane method firstly, and then confirmed by a slip circle method in a various combination of loads and water levels. The foundation of a dam is incorporated in the stability study if necessary.

The section of a rock fill dam consists of a central impervious core, and filter zones and rock shell zones on both sides. This is referred to as zoning of the section. A rip lap is provided on the shell surface to prevent scouring by wave and rainfall as well as weathering by wind and air, and to strengthen the dam against earthquakes.

The extra embankment, which is referred to as a camber, is provided on top of the embankment in provision for the settlement of the dam and/or the foundation. The camber has usually a height of 1 to 2% of the dam height at the central section of the dam and linearly ceases at the dam abutment. Corresponding to the camber, the slopes of both sides of the dam are modified.

An adequate width at the dam crest is provided for the construction and the usage after the completion of the dam

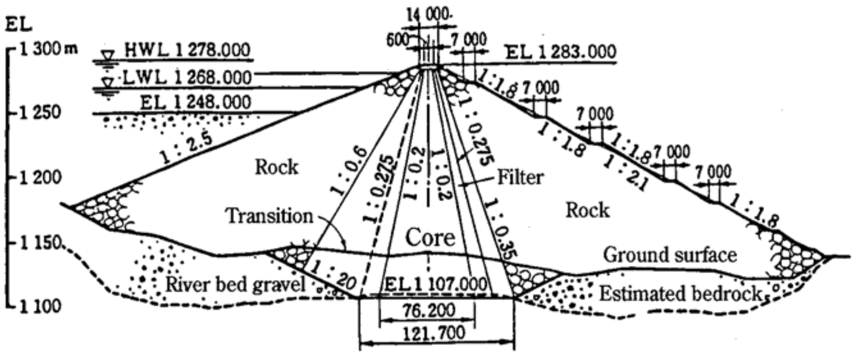


Figure 11-4 Zoned Fill Dam (Takase Dam)

2) Stability against sliding

The stability of a fill dam against sliding is generally analyzed by a slip circle method. In the case of an impervious material facing fill dam, the same method can be applied with the water pressure acting on the upstream facing.

In the following sections, the methods of the stability analysis applied in Japan, the Federal Energy Regulatory Commission (FERC) and the US Bureau of Reclamation (USBR) are presented. Because these three methods are based on the different ideas of load conditions, material strength, and safety factor, it is forbidden to apply them combined with one another.

(a) Japan

Considering a certain slip surface, the loads acting on the slip surface are self weight of the material, seismic inertial force inside the slip surface. Buoyancy and pore pressure are incorporated in the estimation of self weight. The seismic inertia is estimated by multiplying the material weight inside the slip surface and the design seismic coefficient *k*.

To consider the various conditions of a dam, several combinations of the water level and seismic coefficient *k* are incorporated in the analysis.

Full intensity, 100% of design seismic coefficient, k is applied at a constant seepage flow, and at normal high or medium water level. When the water level conditions continue for a short period only (immediately after dam completion, surcharge water level, or design flood water level), the seepage pressure corresponding to each water level condition and the reduced design seismic coefficient are combined. Sliding safety factor should be more than 1.2 in all loading conditions. The conditions for stability calculation are described in Table 11-2.

Table 11-2 Load Combination at Each Water Level

Condition	Slope considered in calculation	Design seismic intensity (%)	Water level	Pore pressure
(1) At normal water level	Upstream and downstream	100	Normal water level	Steady seepage pressure
(2) Immediately after completion	Upstream and down steam	50	-	Residual pore pressure during construction
(3) At intermediate water level	Upstream	100	Level between normal high and low water levels	Steady seepage pressure
(4) At surcharge level	Upstream and downstream	50	Surcharge water level	Steady seepage pressure
(5) At design flood level	Upstream and downstream	0	Design flood water level	Steady seepage pressure
(6) At rapid drawdown				
a) Dams normally subject to rapid drawn down	Upstream	100	Normal high water level ⇒low water level	Residual pore pressure
b) Other dams	Upstream	50	Surcharge water level ⇒ limiting water level in flood Season Normal water level⇒ low water level	Residual pore pressure

When the dam is founded on relatively weak foundation, such as an alluvium layer, stability analysis by slip plane is required to pass through this layer. In this case, it is necessary to consider a linear and linear combined slip surface or a linear and curved combined slip surface.

(b) FERC

Conditions for stability calculation and minimal safety factors against sliding are described in Table 11-3.

Table 11-3 Design Condition and Safety Factor (FERC)

Loading Condition	Minimum Factor of Safety	Slope to be Analyzed	Shear Strength *4
End of construction	1.3	U/S and D/S	Q for impervious zone, S for pervious zone
Sudden drawdown from maximum pool	> 1.1	U/S	R (S for rapidly drained material)
Sudden drawdown from spillway crest or top of gate	1.2	U/S	R (S for rapidly drained material)
Steady seepage with maximum storage pool	1.5	U/S and D/S	(R+S)/2 for R<S, S for R>S
Steady seepage with partial storage pool *1	1.5	U/S	(R+S)/2 for R<S, S for R>S
Steady seepage with surcharge pool	1.4	U/S	(R+S)/2 for R<S, S for R>S
Earthquake with seismic loading using a pseudo static lateral force coefficient (for steady seepage and/or sudden drawdown conditions *2,*3)	> 1.0	U/S and D/S	The same as steady seepage and/or sudden drawdown condition

Notes:

*1) To be analyzed for various pool elevations
*2) To be checked for critical conditions without earthquake
*3) To be checked for frequent drawdown associated with pumped storage project
*4) Q: unconsolidated undrained (UU) strength, R: consolidated undrained (CU) strength, S: consolidated drained (CD) strength

The aseismic stability of a dam on the condition shown in Table 11-3 is calculated by the seismic coefficient method. Although the detailed seismic load conditions are not described in Table 11-3, Seed's theory regarding the seismic coefficient method is frequently applied, as follows.

Based on research showing that displacement by seismic vibration remains within the tolerance at 0.75g acceleration at the dam crest, Seed recommends that the seismic coefficient method be applied to embankments constructed with materials which do not generate an excess pore pressure nor lower the strength more than 15 % during an earthquake. These materials are usually cohesive soil such as clay, silt clay, and sandy clay, or extremely dense non-cohesive soil. Seed proposes the following standards;

Magnitude	Horizontal seismic coefficient	Factory of safety
6.5	0.10g	1.15
8.25	0.15g	1.15

(c) USBR

Conditions for stability calculation and minimum safety factors against sliding are described in Table 11-4.

Table 11-4 Design Conditions and Safety Factor (USBR)

Loading condition	Shear strength parameters	Pore pressure characteristics	Minimum factor of safety
A. End of construction		Generation of excess pore pressure in embankment and foundation materials with laboratory determination of pore pressure and monitoring during construction	1.3
	1. Effective	Generation of excess pore pressure in embankment and foundation materials and no field monitoring during construction and no laboratory determination	1.4
		Generation of excess pore pressure in embankment only with or without field monitoring during construction and no laboratory determination	1.3
	2. Undrained strength		1.3
B. Steady-state seepage	Effective	Steady-state seepage under active conservation pool	1.5
C. Operational conditions	Effective or undrained	Steady-state seepage under maximum reservoir level	1.5
		Rapid drawdown from normal water surface to inactive water surface	1.3
		Rapid drawdown from maximum water surface to inactive water surface	1.3
D. Unusual	Effective or undrained	Drawdown at maximum outlet capacity	1.2

According to USBR Standards, the stability analysis by Spencer is specified for all calculations to check the circle that gives the minimum safety factor. Regarding the slip circle passing through non-cohesive material (c=0), stability analysis by finite plane sliding is also possible according to the Standards.

USBR also introduces the design standards for aseismic design and analysis method. These standards consist of displacement analysis and liquefaction analysis. The purpose is to ensure stability of the embankment and foundation against seismic activity.

3) Structural detail of facing rockfill dam

(a) Concrete facing rockfill dam

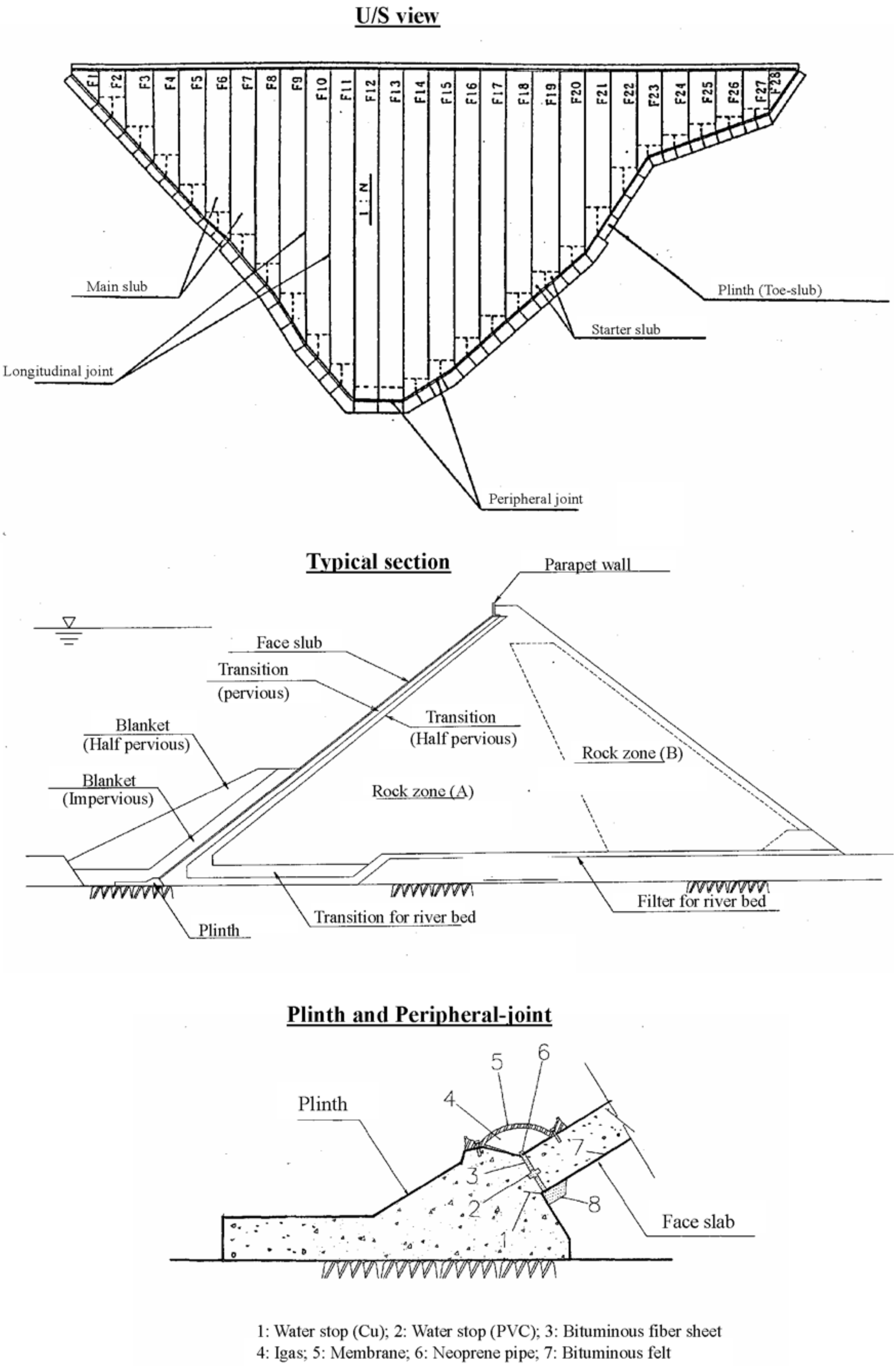
Concrete facing rockfill dams (CFRDs) have been one of the major rockfill dams since later 1970th due to the economic advantage and the establishment of design and construction method for CFRD. Major CFRDs and the typical sections are shown in Figure 11-5 and major dams are listed in Appendix A-11-2.

The major components of CFRD are rockfill, a face slab, a plinth and joints. The concrete slab is reinforced concrete 15 m wide separated by the transverse joints to ensure the water-tightness of the upstream of the dam. It is 0.3m thick at the dam crest and increasingly thicker with the depth of water at the rate of 0.3 %.

The slope of the dam is 1 to 1.4 on the average ranging from 1 to 1.3 to 1 to 1.6, taking the adequacy of the construction into account.

The width of the dam crest ranges from 6 to 8m to provide the construction area on the dam crest. A higher dam shows a tendency to incorporate wider crest width to enhance the seismic performance.

A plinth is founded directly on the foundation to connect the concrete slab to the foundation. A peripheral joint between the plinth and the concrete slab resist against high water pressure and may be a structural defect due to the differential behavior of the plinth and the concrete slab. Therefore special attention is essential in designing the detail structure and selecting the adequate material. The self-healing structure developed for the peripheral joints is a prudent design on the occasion of much leakage. Generally CFRD is not constructed with a grouting gallery. Therefore the design of joints against leakage and the adequate foundation treatment during the construction are essential.



1: Water stop (Cu); 2: Water stop (PVC); 3: Bituminous fiber sheet
4: Igas; 5: Membrane; 6: Neoprene pipe; 7: Bituminous felt
8: Asphalt

Figure 11-5 Typical Sections of CFRD

(b) Asphalt facing rock filldam

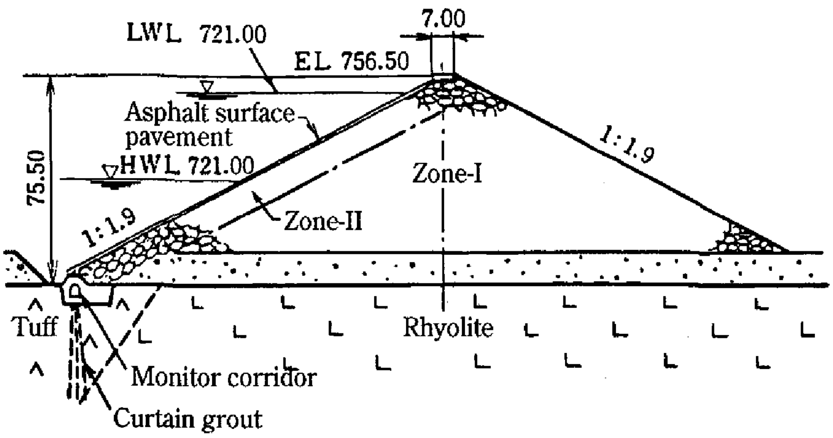
An asphalt facing consists of a base (leveling) layer, a drainage layer, and an impervious layer. Protective coating is applied to its surface. A structure with no drainage layer is also recorded. The middle drainage layer consists of coarse-grained asphalt concrete. Water leaking from the upper facing is led to a gallery in the cut-off to monitor leakage water. The gallery is utilized for foundation treatment. Reliability of the water-tightness of surface membrane is a precondition for a dam with no intermediate drainage layer.

In many cases, the membrane thickness is approximately 30 cm. Where a dam does not have an intermediate drainage layer, its thickness varies from 10 cm to 30 cm. In many cases, it is 10 cm to 20 cm. The slope of AFRD is milder than CFRD due to the construction limitation of asphalt concrete on the dam slope. It ranges from 1 to 1.7 to 1 to 2.0.

The remedial work is relatively easy when the membrane is broken.

The major fill dams with asphalt facing membrane are listed in Appendix A-11-3. The typical section of AFRD is shown in Figure 11-6.

Typical section



Gallery

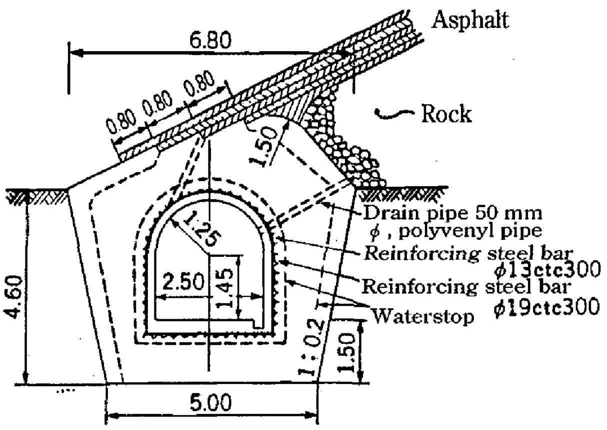


Figure 11-6 Asphalt Facing Rockfill Dam (Miyama Dam)

4) Inspection gallery

The galleries constructed under the impervious zone are generally called an inspection gallery.

The need of an inspection gallery is determined based on the scale of dam, dam design, foundation condition and construction conditions.

The major purposes of an inspection gallery are described below.

- To carry out the grouting work effectively without hindering dam construction
- Maintenance and repair of dam foundation (additional grouting after impounding)
- Safety management of dam and foundation (seepage observation and exploratory drilling)

An inspection gallery can be constructed on the foundation of an impervious core as an open type, in the bedrock as a tunnel type, or as a combination of the two types. The type is determined according to its purpose, and the topography and geology at the site.

Where differential settlement is expected due to a weak stratum in the foundation, although grouting from the ground surface is preferable, the gallery may be constructed in sound rock.

An inspection gallery is generally a reinforced concrete structure to be designed taking account of water pressure, a dam load and grout pressure, and foundation conditions. Such design conditions frequently impose much reinforcing. An example is given in Figure 11-7.

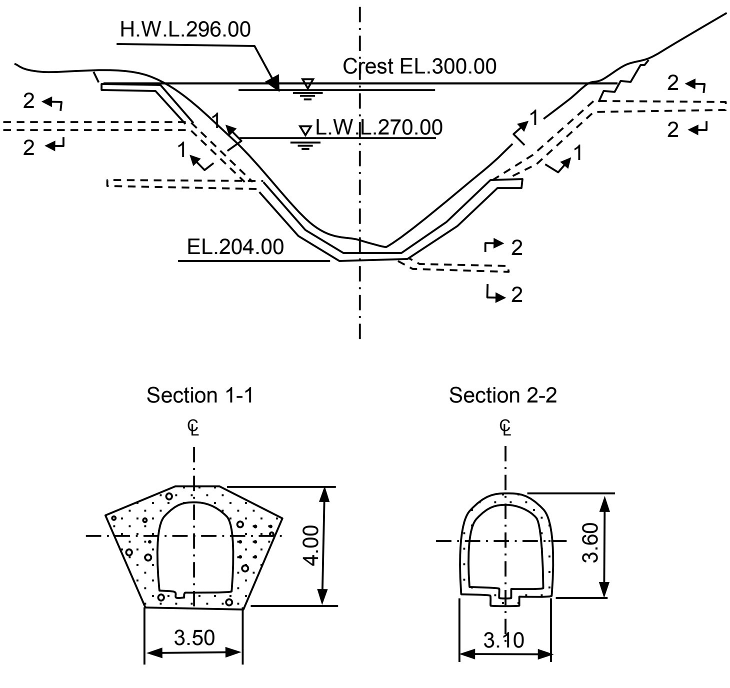


Figure 11-7 Inspection Gallery (Kisenyama Dam)

11.1.2 Spillway

(1) General considerations of design of spillway

A basic requirement of a spillway is to route safely the design flood into a dam without any damage to the dam and its foundation. A structure of the spillway is designed to meet the requirement.

Regarding the design flood, the maximum water level of the reservoir is set as the design flood water level during discharging it through the spillway. A water volume stored in a large reservoir during a flood can be incorporated in the design of the spillway. It is estimated by the following formula.

$$dV/dt = I - O$$

$$\frac{V_2 - V_1}{\Delta t} = \frac{(I_2 - O_2) + (I_1 - O_1)}{2}$$

where,

V	: Reservoir volume
I	: Inflow of flood
O	: Outflow through spillway
t	: Differential time
Suffix 1 and 2	: Present value and succeeding value

(2) Spillway arrangement

A dam spillway is classified based on its relation with a dam, that is, an integrated type, which is a part of a concrete dam or, an adjacent type where it is constructed adjacent to a fill dam as shown in Figure 11-8, and a separate type where it is installed away from a dam as shown in Figure 11-9.

A spillway is a kind of waterway and consists of an inlet, a flow channel, and an energy dissipator. Table 11-5 describes the components of these sections.

Table 11-5 Spillway Components

<table>
<tr><th colspan="2">Inlet section</th><th>Training section</th><th colspan="2">Energy dissipator</th></tr>
<tr><td rowspan="4">Overflow type</td><td>Straight overflow type</td><td rowspan="5">Chute type
Tunnel type
Dam surface overflow type</td><td rowspan="4">Jump type</td><td>Horizontal apron type</td></tr>
<tr><td>Curved overflow type</td><td>Inclined apron type</td></tr>
<tr><td>Side channel type</td><td>Bucket type</td></tr>
<tr><td>Morning glory type</td><td>Forced jump type</td></tr>
<tr><td colspan="2">Orifice type</td><td colspan="2">Ski jump type
Free fall type</td></tr>
</table>

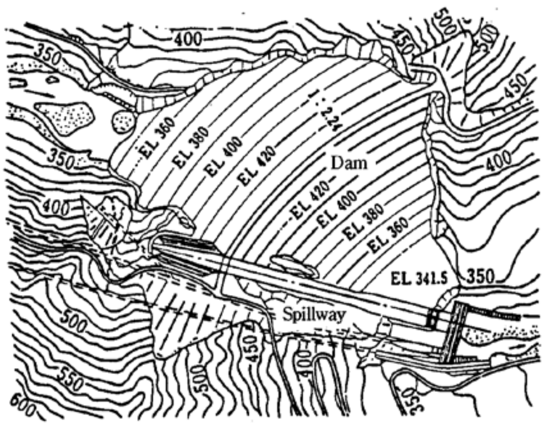


Figure 11-8　Adjacent Type spillway

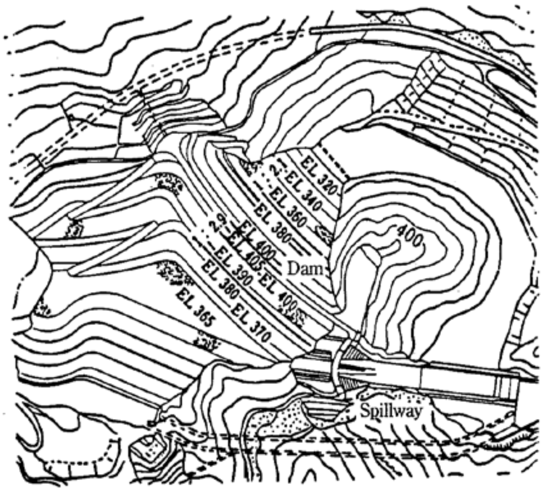


Figure 11-9　Separate Type Spillway

(3) Inlet

1) Inlet channel

An adjacent type or a separate type spillway may be necessary to guide a flow to a flow control area through a channel, depending on the topographic conditions. Configuration of the channel must provide a small velocity and gentle flow variation since a large velocity increases inflow loss, thereby reducing the discharge capacity.

Velocity in the channel is recommended to be under 3 to 4 m/sec to prevent the inflow of sediment in the channel.

2) Ogee crest

The flow from the inlet channel changes its characteristics from a subcritical flow to a supercritical flow over an ogee crest. The profile of the ogee crest is shaped as an over flow profile through a sharp edge weir to enable the maximum discharge without generating negative pressure on the flow surface during flood discharging. The Harold's profile is frequently used for the ogee crest profile. (Figure 11-10)

A total width of the spillway roughly corresponds to one of the river downstream. When spillway gates are arranged, the study on the number of the gates is necessary from an economical point of view, considering the gate dimensions technically.

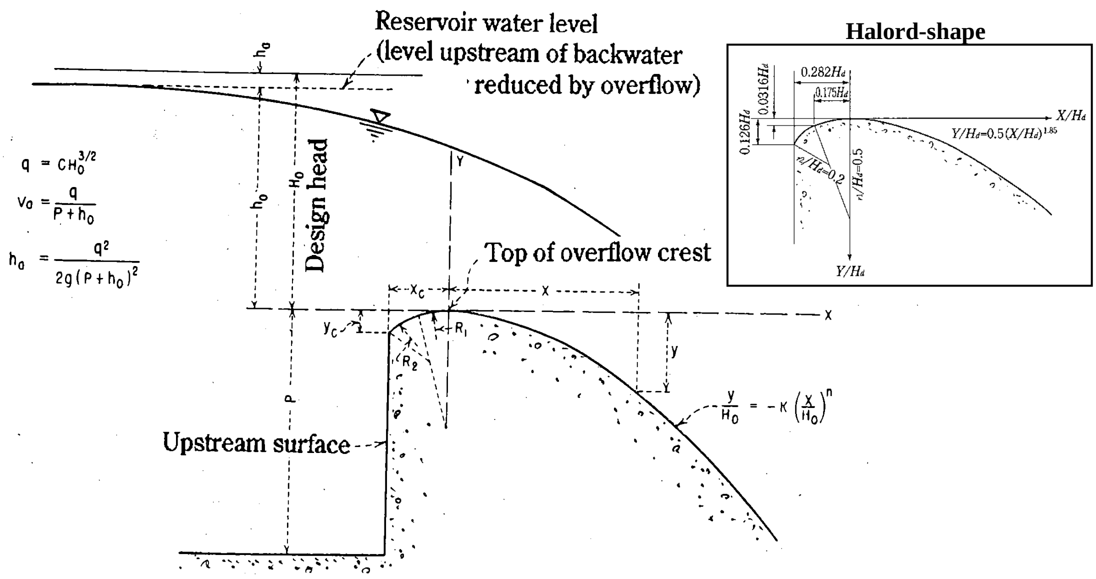


Figure 11-10 Typical Crest Profile

(4) Flow channel

A flow channel is classified into a dam body type which utilizes the concrete dam releasing water at the downstream end, and a chute type or a tunnel type in which the channel is constructed along or inside the bank. Since the flow in a channel is a supercritical flow, it is designed in a linear line with a uniform channel width and an almost fixed longitudinal slope. If a non-linear or curved surface profile of the channel -section is to be partially inserted, it is necessary to consider an impulse wave, a water level rise, and an uneven flow.

When changing the longitudinal slope, it is necessary to insert a gentle curve to enable the flow to run smoothly without much friction to the bottom. Aeration grooves may be designed on the bottom to supply sufficient air to prevent cavitation. In this case, a flow in the channel is an air entrained flow. The water depth becomes larger than that acquired from the model test or calculation, and requires a correction accordingly.

(5) Energy dissipator

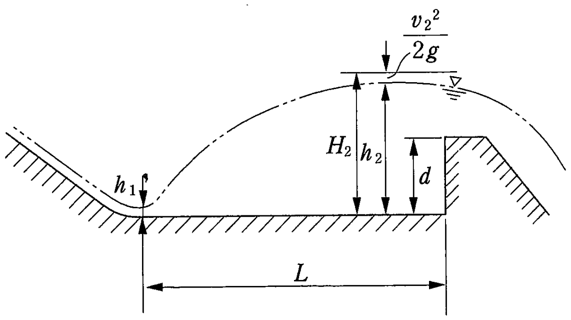
Because of the high velocity of water flowing down a spillway, it is necessary to install an energy dissipator at the end of the spillway to prevent scouring and erosion downstream of the dam and to protect the adjacent structures. Energy dissipators are classified into a hydraulic jump type, a ski jump type, and a free fall type. The hydraulic jump type is frequently applied to gravity dams and filldams (Figure 11-11). The free fall type is applied to arch dams.

It is necessary to consider the following factors when selecting the type of energy dissipator.

- Hydraulic characteristics of an energy dissipator
- Physical relations between a dam and an energy dissipator such as the distance and elevation

- Hydraulic and structural characteristics of spillway
- Topography, geology and hydraulic characteristics near the energy dissipator
- Location and importance degree of a downstream river and structures

In Japan, the design discharge for the energy discipator can be reduced to 1/1.4 to 1/1.5 than the design flood inflow of the dam by taking the risk of the structures downstream into account.



where, d: sill height
L: apron length

Figure 11-11 Example of Energy Dissipator

11.1.3 Outlet Works

(1) Purpose of outlet works

The outlet works other than a spillway are as follows.

- Water usage outlet works: To supply water for irrigation, tap water, and industry.
- Flood control outlet works: To control the water level within a restricted level and to carry out preliminary discharge during flood.
- Flow control outlet works: To maintain the required river flow downstream of a dam.
- Reservoir management outlet works: To drawdown the reservoir water level for the safety of a dam and reservoir surroundings.

Dam structure, river channel conditions in the site, reservoir objectives, reservoir water level operation plan and flood characteristics in the river basin are all considered when designing the dam outlet works to ensure the above described performance.

(2) Design detail of outlet works

The type and capacity of the outlet works are selected based on the operation purposes, operation frequency, discharge and installation conditions, and maintenance. The maximum discharge capacity is determined to meet the required discharge for water use, or to enable drawdown of the reservoir water level to the required level in reservoir operation during a flood or for the inspection and/or repair of the dam against incidents.

It is recommended that the outlet works be located at the lower elevation. This location is determined at where it is free from sedimentation.

The outlet work of a concrete dam is frequently integrated with the dam. The outlet work of a fill dam to be separately installed from the dam may be arranged in the river diversion tunnel or the tunnel for the construction from an economical point of view. It is necessary for it to be located at a site convenient for maintenance and operation. Examples are shown in Figure 11-12and Figure 11-13.

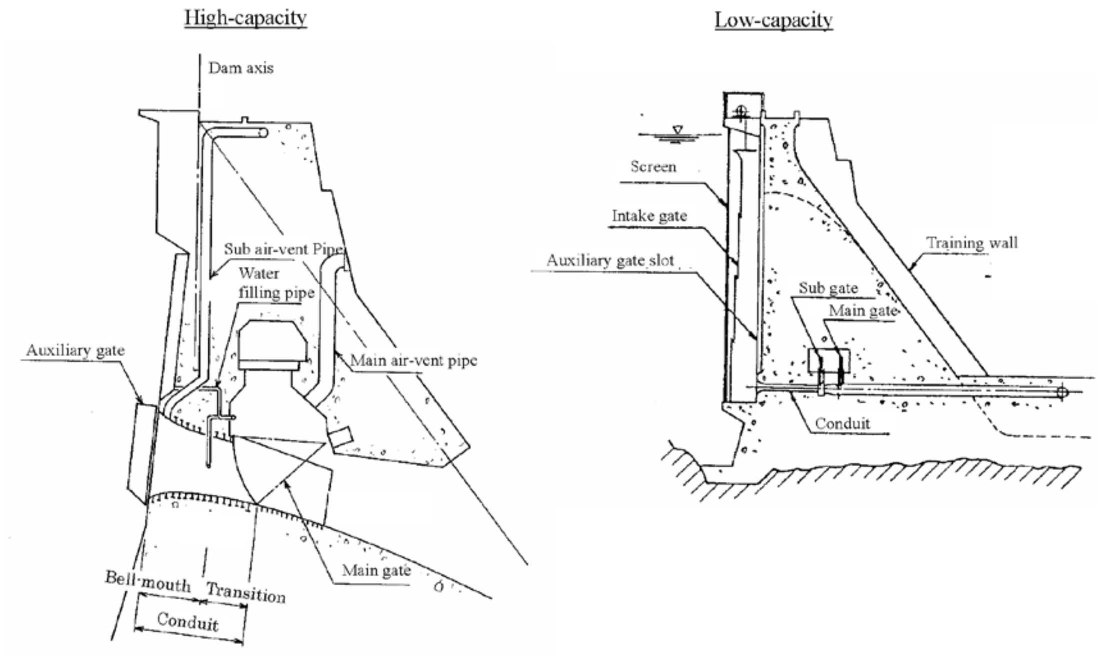


Figure 11-12　Outlet Works of Concrete Gravity Dam

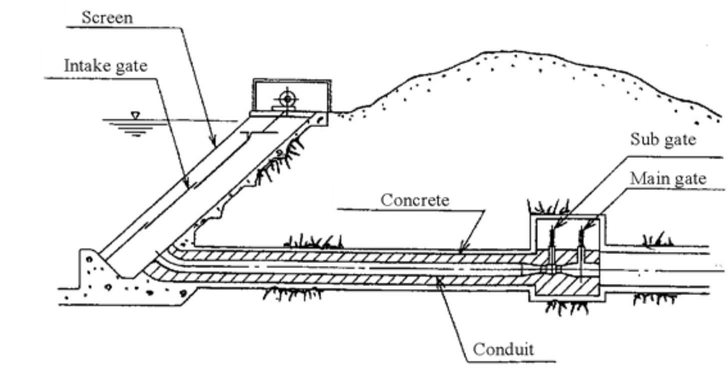


Figure 11-13　Outlet Works of Rockfill Dam

(3) Conduit configurations

The outlet works are generally pressurized conduits with high velocity, even having smaller discharge than one of the spillway. When designing the conduit, it is necessary to consider that the high velocity occasionally makes a significant pressure drop in the conduit, causing problems such as cavitation.

Where the cross section changes at the pressure conduit inlet or at its curved area or grooves, an inadequate configuration change generates a local pressure drop due to a separated streamline, which could cause cavitation. Cavitation may also be caused by a slight change of the conduit configuration during the manufacture and construction process despite a safe design configuration. These factors must be considered when designing a pressure conduit so that the pressure applied to the conduit, gate, and other auxiliary facilities may be retained at the normal value. Generally, the minimum design pressure is designed so that it does not become a negative pressure.

(4) Gates and valves

The types of the gate and the valve for the outlet conduit are determined considering the discharge conditions, pressure conditions, adaptability to the dam, outlet conduit structure, and maintenance/operation. Generally, a slide gate, fixed roller gate, radial gate, jet flow gate, hollow jet valve and cone valve are applied to dams.

Gates or valves with a head of approximately 25m or larger, and with a discharge of approximately 5 m^3/sec or larger are called high pressure gates or high-pressure valves. Their materials must be resistant against corrosion and cavitation.

Gates and valves must open and close safely and fully. Manual operation devices and auxiliary power facilities are installed as required considering the possibility of troubles with the power source due to storms or earthquakes.

11.1.4 Countermeasures against Sediment

Countermeasures against sediment can contribute to reduce the sediment volume of the reservoir, which make the lower dam height and sustainable use of the dam and the reservoir.

The countermeasures are classified to reducing the sediment into the reservoir and discharging the sediment out of the reservoir. Examples are a bypass tunnel and a sediment weir, and flushing, sluicing and dredging respectively. In the following sections, a bypass tunnel and flushing of the sediment are described.

The facilities for the countermeasures require a high cost to construct additionally in the existing reservoirs. Therefore, it is recommended to arrange adequate countermeasures against sediment in the design stage of the project which is located in the area of much sediment anticipated.

(1) Bypass facility

A bypass facility for sediment consists of a diversion weir, a bypass tunnel and a dissipating basin. The diversion weir, locating upstream, diverts entirely or partially floods into the bypass tunnel. The bypass tunnel, locating in the mountain besides the reservoir, route the sediment from the reservoir to the river downstream of the dam as a mixed flow of the flood water and sediment. Flow energy of the sediment mixed flow is dissipated at the downstream basin to release the water further downstream of the river.

The countermeasures against the erosion on the surface of the bypass tunnel are essential for the design of the bypass facility. The diversion tunnel of the dam construction may be utilized for the bypass tunnel.

An example of the bypass facility is shown in Figure 11-14.

(2) Flushing facility

A flushing facility releases the sediment in the reservoir by enhancing the flushing capacity of the sediment at a lowered water level of the reservoir. A typical flushing facility consists of a sluice gate and a conduit incorporated in a concrete gravity dam.

Easy recovery of the reservoir water level is an important condition of the adoption of the flushing facility. A concern of the designer is countermeasures against scouring of the flushing facility as well as the sediment bypass facility.

An example of the flushing facility is shown in Figure 11-15.

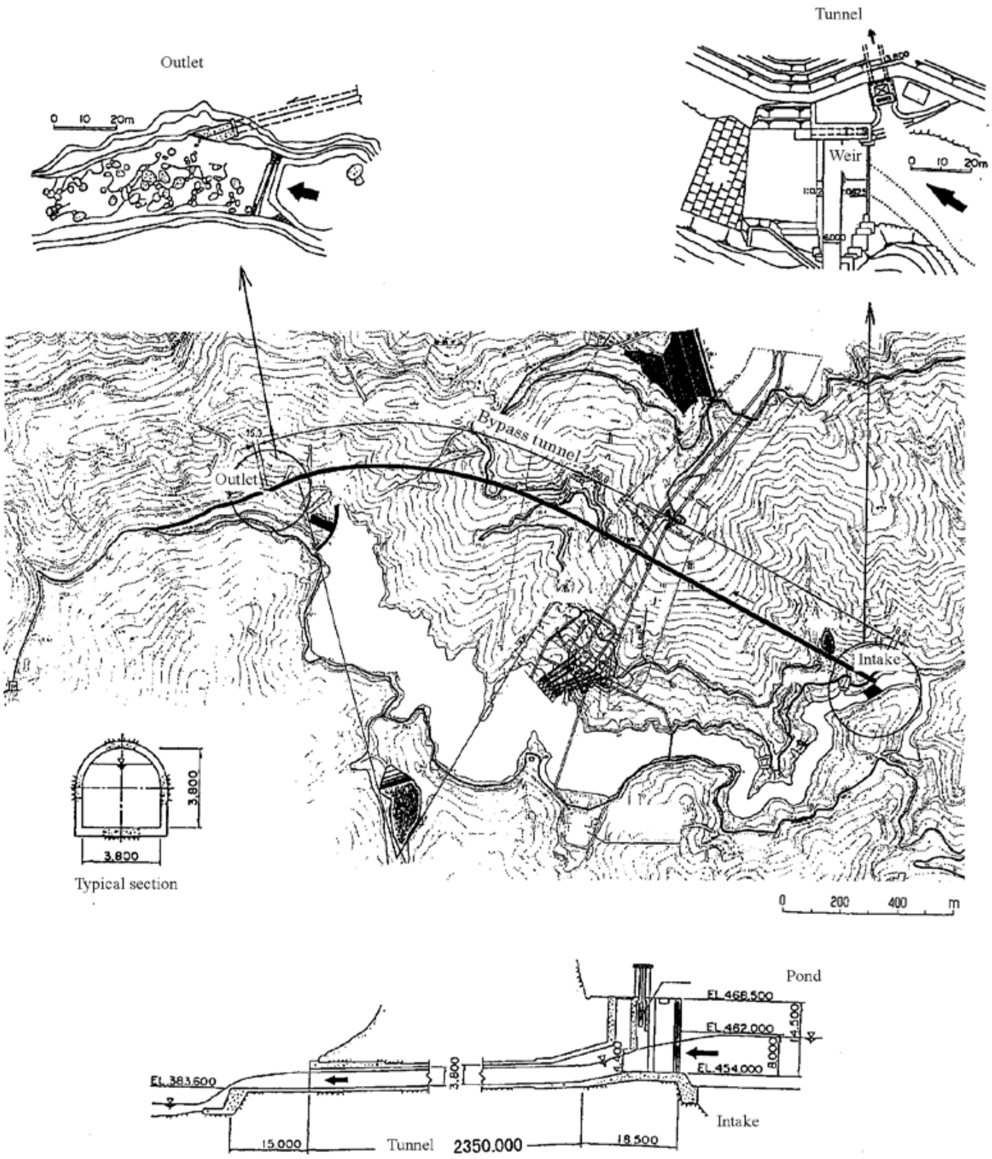


Figure 11-14 Example of a Bypass Facility (Asahi Dam in Japan)

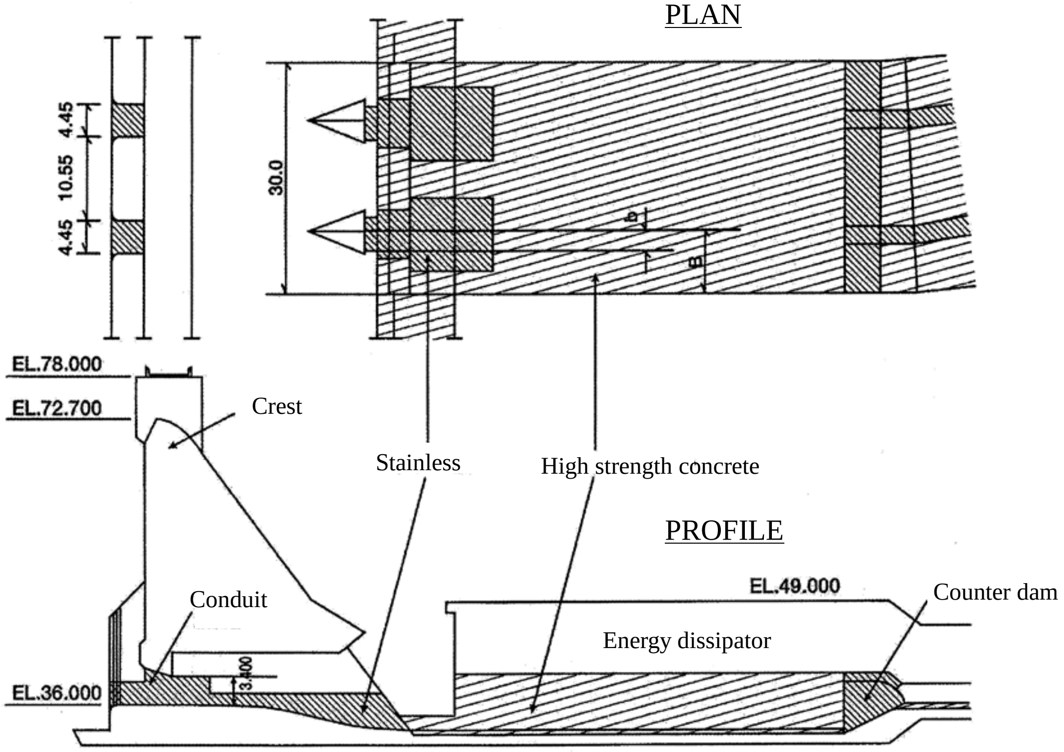


Figure 11-15 Example of Flushing Facility (Masudagawa Dam in Japan)

11.1.5 Intake Weir

An Intake weir, distinguished from a dam before described, is a low dam which collects the river water of a run-of-river type power plant.

(1) Structure of weir

The following conditions are considered to select an intake weir site.

- The foundation should be a sound rock with little sediment deposit.
- When the weir site is situated on deep river deposit, a relatively small volume of excavation may reach a fairly well consolidated layer suitable for foundation of the weir.
- After intake weir construction, least scouring is expected in the river bank and river bed immediately downstream.
- The river diversion and cofferdam works are easy.
- The cross section is wide enough to safely release the design flood discharge despite future sedimentation.
- The intake weir is usually installed at a right angle to the river. Where it must be installed diagonally, however, preventive measures are taken against scouring of the banks during floods.

Intake weirs are classified into gated weirs and overflow weirs without gates which are referred to as ungated weirs. In many cases, ungated concrete gravity weirs are used as shown in Figure 11-16. However, where it is necessary to reduce the damage in an upstream area due to the effect of back water during a flood and it is difficult to flush the sediment by a sand flush structure only, gated weirs with a roller gate, an inflatable rubber gate or a steel-rubber combined inflatable gate (SR gate) are used.

In gated weirs, sediment will not reach to the dam crest, and is convenient to maintain the water depth required for water intake.

The intake weir should be high enough to enable the intake to collect the maximum plant discharge. In addition, the effect of backwater on the upstream river structures and existing power plants if any must be taken into consideration.

Intake weirs are constructed on a foundation rock. Where an alluvial layer on the riverbed is thick and the foundation rock is deep, a floating dam which does not rest on the bedrock may be constructed as shown in Figure 11-17.

Where the contact surface between the weir and the bed rock is suspected of water seepage, a cut-off at upstream end of the weir should be set deep into the foundation rock. Also, an apron of an appropriate length must be installed to prevent riverbed scouring in the downstream of the weir by overflowing water.

(2) Appurtenant facilities

1) Sand flush structure

Sediment starts to take place when a weir or a small dam is installed on a steep river. A sand flush structure is, therefore, required to ensure the intake of water and to prevent sediment from flowing into a waterway. A sand flush structure is generally installed near the intake and water is released to flush the sediment when the flood flow recedes. Sill height of the sand flush structure is arranged slightly lower than the intake sill height. Where the weirs are gated, one gate near the intake can be used as a sand flush gate. The dimensions of the sand flush gate must provide sufficient velocity to flush the sediment. The entire sill may be covered with stones or a steel sheet to prevent severe surface erosion.

A slide or roller gate as shown in Figure 11-16 is adopted so that it can be reliably raised with sediment behind the gate and closed even during discharging water containing some ediment.

2) Fish way

A fish way may be arranged in the weir where migrating fish are found in and around the project site.

Fish ways are waterways which allow the passage of fish migrating upstream to spawn. They are classified into a inclined type, a ladder type, and an elevator type depending on their structures. The fish way should be a minimum one-meter (1 m) wide with a gradient of 1 in 10, to 1 in 15. The velocity of flow should be approximately 1.2 to 1.5 m/sec as shown in Figure 11-18.

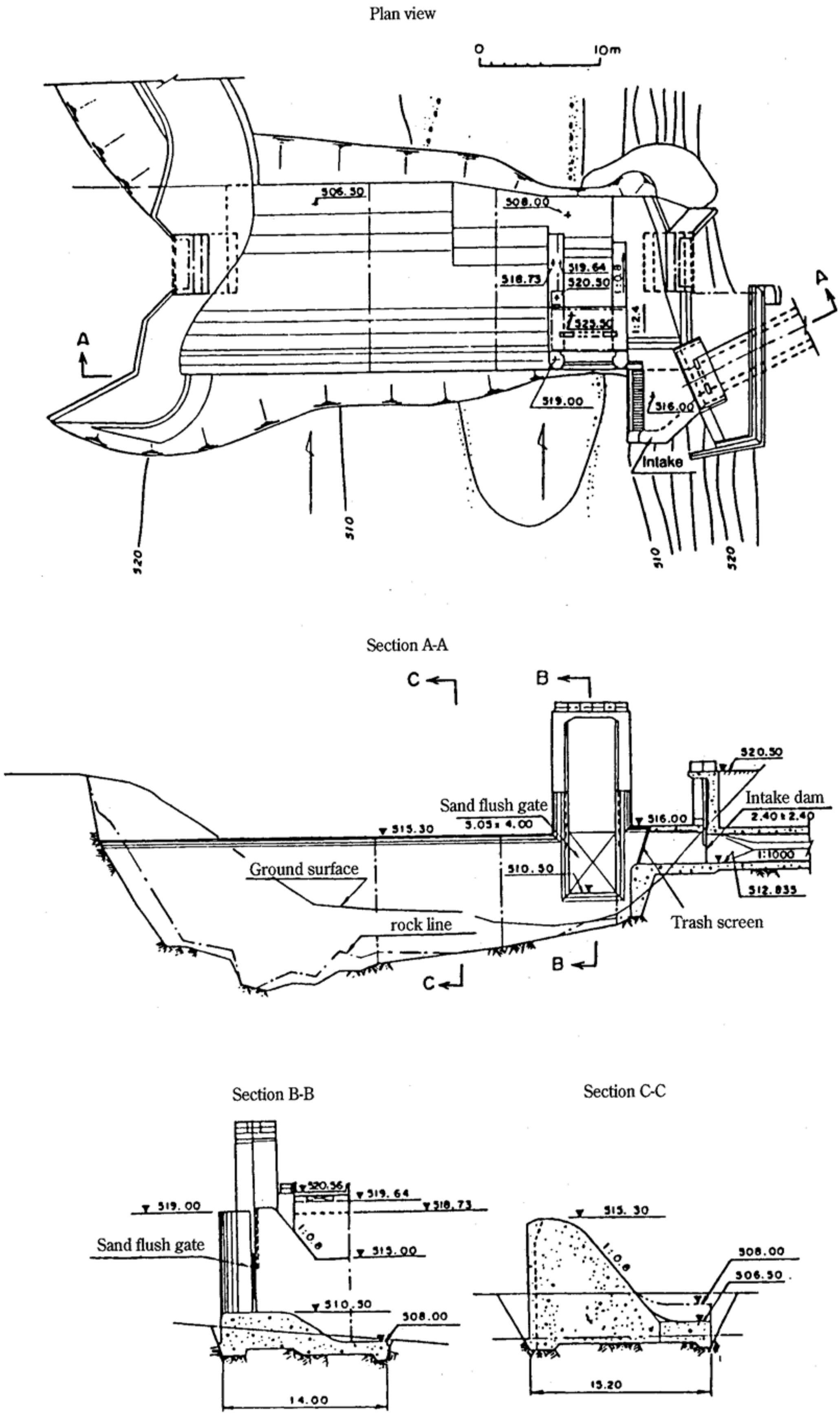


Figure 11-16 Intake Weir

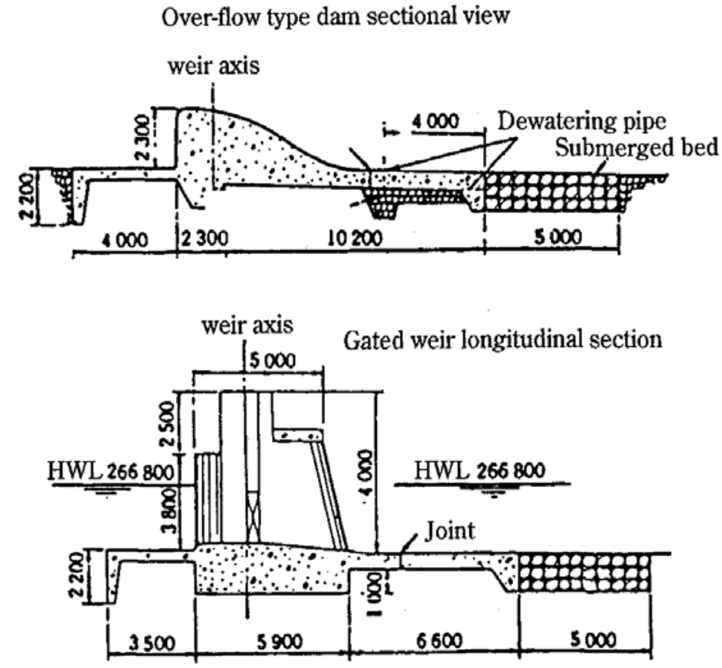


Figure 11-17 Example of Floating Dam

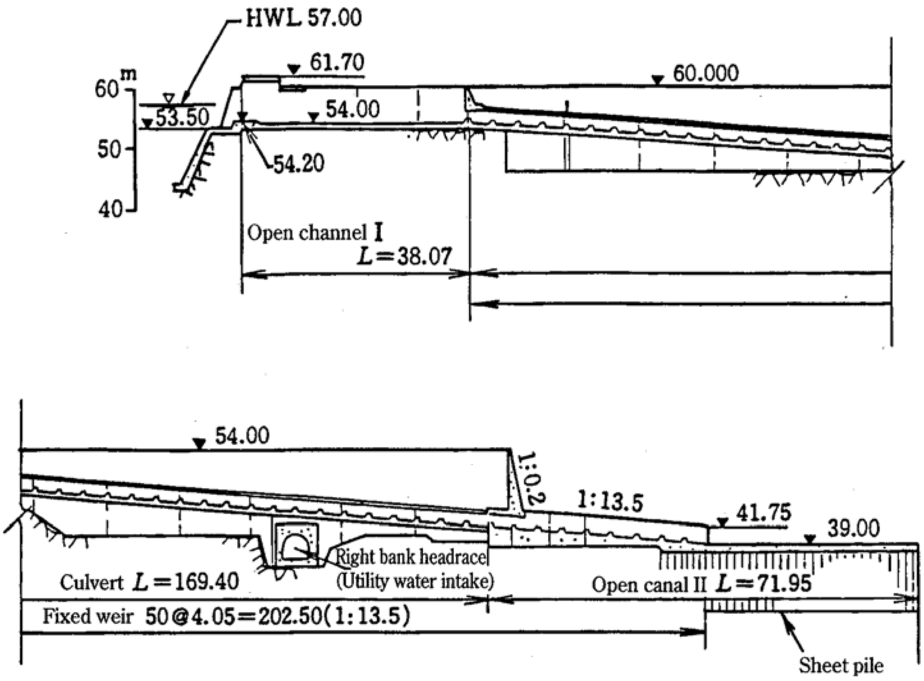


Figure 11-18 Example of Fish Way

11.2 Intake

11.2.1 Structure and Type of Intake

An intake is a structure to conduct water from a river, reservoir or regulating pond to a waterway. Its location, alignment, structure and appurtenant facilities must satisfy the following conditions.

- The design discharge is constantly available and can be controlled as required.
- The head loss is small.
- Inflow must be smooth and must not cause a vortex and entrainment of air.
- Sediment, driftwood or leaves must not flow into the waterway.
- Intake must be safe from damage by flood or landslide.
- The intake of a pumped storage power plant functions as the outlet during pumping operation.
- All maintenance works after completion must be easy.

The intake is often constructed on the slope of riverbank or lake shore. Therefore, it must be structurally stable and located at a site safe against a landslide, a rock fall and an avalanche.

Intakes are classified into a non-pressure type and a pressure type according to the hydraulic conditions of the waterway. Flow into the structure is horizontal, inclined or vertical.

Its structure may be an integral part of a dam (weir) or independent from a dam (weir). Independent intakes may be constructed on the riverbank slope, or as a self-standing tower.

11.2.2 Intake for Non-Pressure Waterway

The intake for non-pressure waterway is generally installed near the weir. The site of the intake should be selected so that the design discharge can be drawn from the river without being affected by sediment behind the weir, free from damage caused by flood flows and driftwoods.

The intake should be aligned at a right angle or at a slightly lesser angle to the river. The site should also provide a space for a settling basin between the intake and a headrace inlet.

The intake must be designed with an inflow velocity of approximately 0.3 to 1.0 m/sec. The intake sill height should be approximately 1m higher than that of the sediment flush structure to prevent the entry of sediment into the waterway. A submerged weir should also be constructed in the front of the intake to prevent entry of sediment. In addition, a trashrack with bars spaced 5 cm to 15 cm apart are installed at the front of the intake. At the inlet of the waterway, a control gate is installed to adjust the discharge and to close the waterway for inspection and repair as shown in Figure 11-20. To minimize head loss, the cross section from the intake to the waterway should be free from abrupt changes and is often designed as a bell mouth.

11.2.3 Intake for Pressure Waterway

The intake for pressure conduit enables intake of water at any water level within the available water depth.

Where the intake is constructed inside the reservoir or the regulating pond judging from the small amount of sediment, topography, energy of flood water, etc., its position and direction can be selected comparatively freely. The structures are an inclined type on the riverbank or a tower type as shown in

Figure 11-21 inside the reservoir, or a dam body type incorporated in a dam. The velocity at the intake is approximately 0.3 to 1.0 m/sec to minimize an inflow loss and to facilitate sediment removal from the trashrack

It is desirable that water depth above the intake sill be approximately 1.5 to 2 times the headrace diameter to prevent entrainment of air during intake of water. A trashrack is installed in the front of the intake, and a bellmouth shaped inlet is adopted to minimize a head loss. The intake sill height is determined considering the expected sediment depth.

A multi-stage gate may be installed to permit intake of surface water only for irrigation in the downstream area, or a selective intake gate may be installed to draw water from an optional depth as a countermeasure to reduce water turbidity in the downstream.

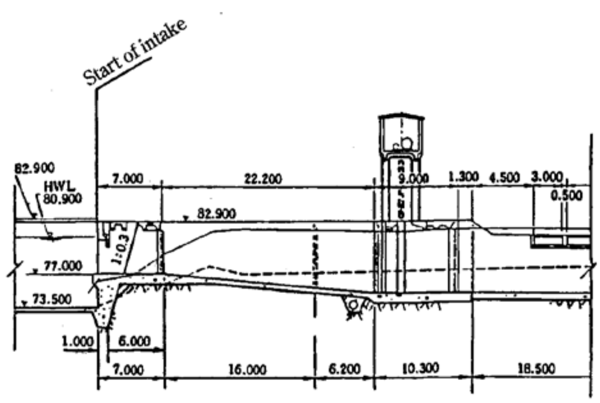


Figure 11-19 Intake for Non-Pressure Waterway

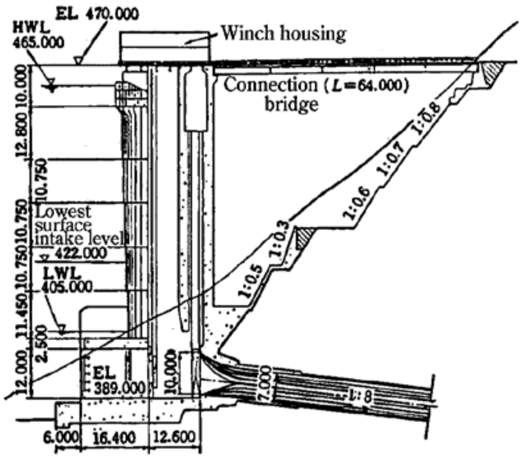


Figure 11-20 Intake for Pressure Waterway

11.2.4 Appurtenant Facilities

(1) Control gate

The control gate is used to keep a waterway empty for the inspection and repair. It is also used to control the inflow corresponding to the intake water level and discharge fluctuation. In a case where a powerhouse is located near a dam, an inlet valve of a turbine may not be installed and its function is conducted by a control gate.

A slide gate or a fixed roller gate is often applied to a non-pressure waterway. A fixed roller gate or a caterpillar gate is applied to a pressure waterway. However, the caterpillar gate is now only rarely used because fixed roller gates are available for large, high pressure conduits.

(2) Trash prevention facility

A trash prevention facility is installed to prevent the entry of trash, driftwoods, etc. into a headrace. It is mainly classified into a trash boom installed upstream of the intake across a reservoir, a trashrack installed at the front of intake, and a clearing machine.

A trash boom is installed away from the intake across a reservoir to primarily prevent large driftwoods from directly hitting the trashrack. In many cases, the trash boom consists of drums or other empty containers secured to a rope, or waterproof floats made of styrofoam which are tied together. The trash boom is set anchored to the banks or a dam so as to correspond to the variable water surface. A pontoon bridge is placed where appropriate.

Trashracks are installed at the front of intake to prevent the intrusion of driftwoods and other trash. The trashracks are made of flat steel bars placed vertically at even spacing and connected tightly by tie-bars and spacers, and firmly fixed to a support beam. Although a vertical trashrack is of a simpler structure, a tilted trashrack is easier in removing the trash. A small amount of trash may be removed manually. However, a mechanical rake is often used as it automatically removes trash efficiently and safely.

11.3 Settling Basin

River water contains a certain volume of suspended sediment. During floods, sediment concentration increases substantially. In a run-of-river plant, suspended sediment deposits in the waterway, and chokes its sectional area. It is also the cause of erosion of the penstock and turbine. To settle and flush the sediment, it is necessary to install a settling basin close to the intake.

The settling basin must be sufficiently long to allow fine sand particles to fall from the water surface gradually and finally reach the bottom at the end of the basin, i.e.

$$L = \frac{d}{W_{sk}} V$$

where,

L : minimum required length of settling basin (m)

V : the average water velocity in settling basin (m/sec)

d : depth of settling basin (m)

W_{sk} : marginal settling speed of sediment (m/sec)

$$W_{sk} = \left(\sqrt{\frac{2}{3} + \frac{36\nu^2}{sgd_k^3}} - \sqrt{\frac{36\nu^2}{sgd_k^3}} \right) \times \sqrt{sgd_k} \qquad \text{(Ruvey formula)}$$

ν : dynamic viscosity coefficient of water (m^2/sec)

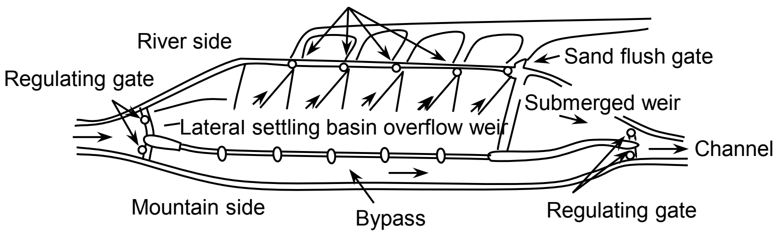
s : sand weight in water

The above formula is a theoretical one. Considering the effects of vortexes and sub-flows, the length of the settling basin is often more than twice the calculated length. Water depth *d* is designed slightly deeper than that of the headrace, and the channel width is designed to provide the average velocity of approximately 0.3 m/sec.

The alignment of the settling basin should possibly be the same as the direction of the water inflow to provide a uniform velocity in the basin. The width should also be expanded at a slight angle from the approaching section. Training walls are constructed as required to prevent a vortex flow.

The bottom of the settling basin should be sloped to facilitate movement of the deposited sand by water energy to a location in the downstream area and remove it through a sand flush gate.

Where the sediment must be removed without interrupting the power generation, the settling basin may be divided into two sections. Here, the water flow in one section is closed and the sediment removed, or, a side canal may be provided to switch the river water to the side canal when the river water is clean to discharge the sediment as shown in Figure 11-21.



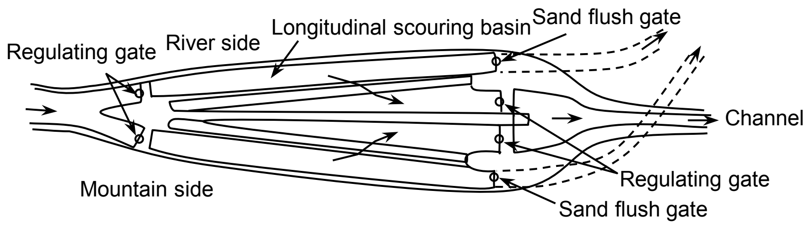


Figure 11-21   Settling Basins

11.4 Water Conveyance System

A water conveyance system covers the distance from an intake to a turbine. In the case of a run-of-river type power plant, it consists of a headrace, a head tank and a penstock. In the case of a dam and waterway type power plant, it consists of a headrace, a surge tank and a penstock.

11.4.1 Type selection of Headrace

A headrace is classified into a pressure waterway and a non-pressure waterway. In terms of hydraulics, a non-pressure waterway is an open channel and a pressure waterway is a conduit.

The headrace structures are an open canal, covered canal, culvert, tunnel, aqueduct, inverted siphon, etc. Figure 11-22 shows an example of typical types of structure, an open canal (channel) and a tunnel.

An open canal is often used when the headrace passes on a flat terrain. In general, the construction cost of an open canal is the most economical. However, locations prone to landslides must be avoided. A covered canal and a culvert are used in locations where the ground cover is too thin to construct a tunnel, and where a stability problem may occur on the high excavated slope. The excavated height is generally limited to lower than 10 to 15m.

An aqueduct or an inverted siphon is constructed when the waterway crosses a river.

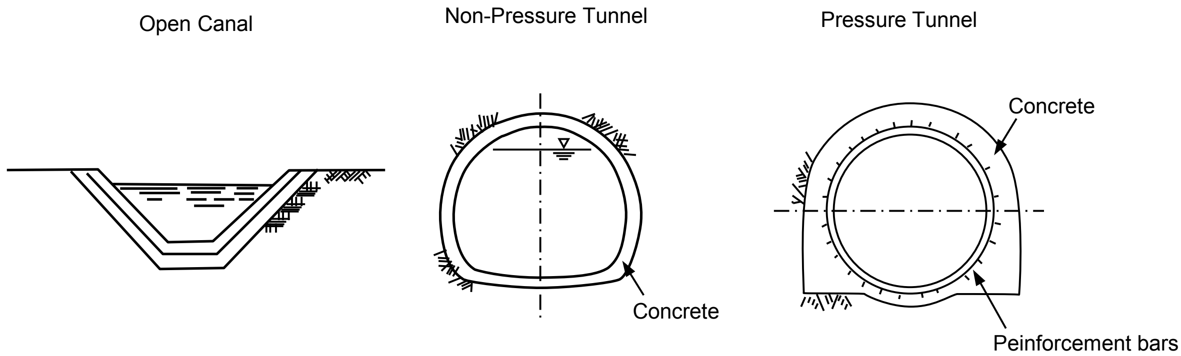


Figure 11-22 Cross Section of Headraces

11.4.2 Waterway Gradient and Cross Section

(1) Non-pressure waterway

The cross section of the non-pressure headrace canal is determined in relation to the headrace gradient. When the discharge is fixed, a steeper headrace gradient provides a smaller canal cross section, thereby reducing the waterway construction cost. This is, however, not always economical because a friction loss in the waterway increases and an electricity generating output decreases. Contrarily, a gentler waterway gradient makes the waterway cross section larger and the electricity generating output increases due to the decreased friction loss. Here, however, the waterway construction cost increases and the sectional area of the canal decreases due to sediment settling in the waterway.

It is, therefore, necessary to determine the most economical waterway gradient and waterway cross section in consideration of the waterway construction cost and power output and energy generation.

The velocity of a flow in a non-pressure waterway is generally 2 to 3 m/sec. The waterway gradient is generally 1:1,000 to 1:2,000 in the case of an open canal and 1:500 to 1:1,500 in the case of a tunnel.

Typical cross sections of a non-pressure headrace are rectangular, trapezoidal, horseshoe-shaped or circular. For an open channel a trapezoidal cross section is generally used, however the shape may be rectangular for a small flow. A horseshoe-shaped cross section is generally used for the culvert and tunnel due to easy construction, high strength against external forces, and low cost. The side wall is easier in construction if the side arch has a larger radius close to a straight line.

Therefore, where the geological conditions are good, the side wall is constructed perpendicularly or almost straight.

Where the geological conditions are unfavorable, a cross section close to a circle is used to resist external pressure. When studying the cross section, it is convenient to use hydraulic characteristics curve showing the relation between the water level and a velocity and a volume of flow.

(2) Pressure Waterway

The velocity in the pressure conduit has no relation to the waterway gradient. It is related to the hydraulic gradient. To put it in more detail, if the pressure conduit is designed so that the entire conduit is positioned below the hydraulic gradient line connecting the design lowest water level of the intake to the design lowest water level of the surge tank, water in it flows down by hydraulic gradient with no relation to the waterway gradient.

If the cross section of the pressure tunnel is reduced, construction cost decreases but a head loss increases and a power output and energy generation decrease. Contrarily, if the tunnel cross section is increased, construction cost increases but a head loss decreases and a power output and energy generation increases. Several cross sections of the pressure tunnel should be compared and the most economical one should be selected by the relation between the construction cost and benefit.

The velocity in a pressure tunnel is generally 2 to 4 m/sec in the case of a conventional hydropower type and 5 to 6 m/sec in the case of a pumped storage type.

In many cases, the waterway has a circular cross section which is structurally the most rational shape. The horseshoe shape is used when the internal pressure is not so high or when the geological conditions are good. When, the plant discharge is small, the headrace cross section should be the smallest possible cross section that can be constructed at the expense of the economy.

11.4.3 Discharge in Headrace

A Manning's formula is used for both the non-pressure waterway and the pressure waterway to calculate the discharge in concrete or steel lined waterway.

$$V = \frac{1}{n} \times R^{2/3} \times I^{1/2}$$

$$Q = V \times A$$

where,

V	: Average velocity in waterway (m/sec)
R	: Hydraulic radius (m)
I	: Hydraulic gradient
Q	: Discharge (m^3/sec)
A	: Flow sectional area (m^2)
n	: Manning’s roughness coefficient depending on unit system

The values of Manning’s roughness coefficient are shown for various surfaces of the headraces. Values from 0.017 to 0.029 are applied to the shotcrete lining. The combined value of Manning’s n is used for the headrace surface consisting of two or more materials.

Table 11-6 Manning's Roughness Coefficient

Lining Material	n
Conduit constructed with steel forms and lined by good quality concrete	0.011 - 0.014
Conduit lined by conventional concrete	0.012 - 0.016
Tunnel lined with invert concrete only	0.020 - 0.030
Totally un-lined tunnel	0.030 - 0.040
Welded steel pipe	0.010 – 0.014
Shotcrete	0.017 – 0.029

11.4.4 Design of Tunnel Support

In a tunnel excavated by NATM, the surrounding rock reinforced with shotcrete and rockbolts basically ensure the stability of the tunnel. However a waterway tunnel is lined for the following reasons.

- To prevent a collapse of a tunnel and maintain the cross section.
- To assure watertightness and prevent water leakage.
- To construct a smooth inner surface thereby reducing a head loss.

(1) Non-pressure tunnel

The lining of a non-pressure tunnel must give adequate strength against external pressure, although it is difficult to theoretically determine the concrete lining thickness. The lining of 20cm to 30cm thick is typically adopted.

(2) Pressure tunnel

The reinforced concrete is generally used for pressure tunnel lining, however plain concrete may be used when the internal pressure is small. Hydrostatic pressure, water hammer pressure, surging pressure, etc. are internal pressure and may be considered by classifying the design load into a long-term load and a short-term load. External water pressure, grouting pressure, ground pressure, etc. are external pressure. Grouting pressure is usually dominant over all of others.

There are various concepts regarding the lining design method i.e., whether the internal pressure should be distributed to the surrounding rock, whether the tensile strength of the lining concrete should be counted, or whether pre-stressed lining concrete should be used. Many calculating equations are also proposed. The formula referred to as Otto-Frey-Baer, which is based on the multi-walled cylindrical structure theory, is frequently used for the design of the lining of the tunnel considering the surrounding rock.

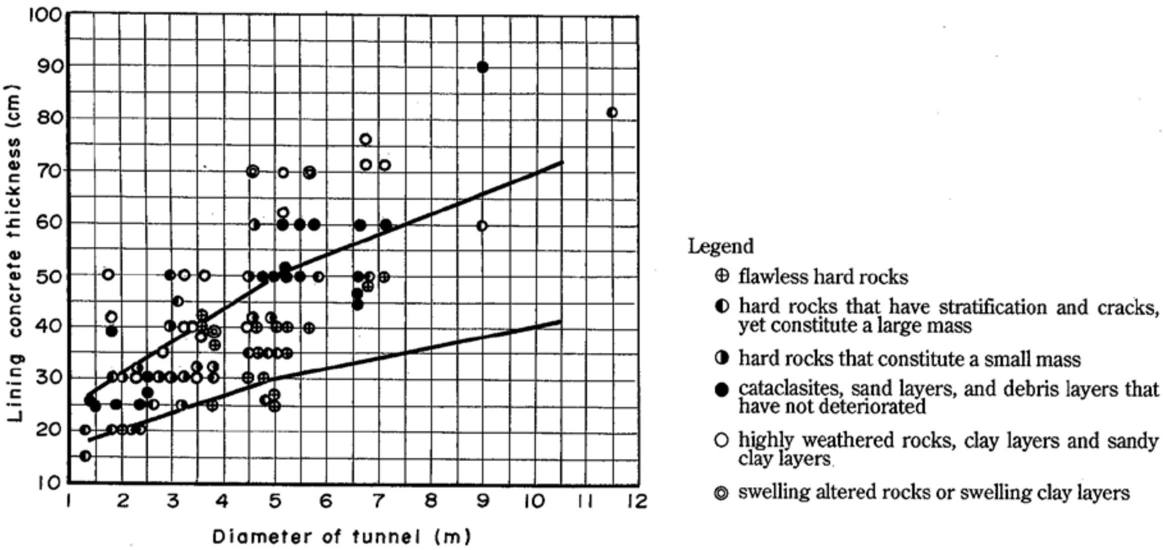


Figure 11-23 Relation Between Pressure Tunnel Diameter and Lining Thickness

Steel lining may be used to prevent water leakage in areas near the ground surface such as the intake or surge tank, the branching point of waterway or geologically weak locations. In this case, the following precautions are required.

- Consideration must be given to prevent buckling by remaining external water pressure when internal pressure is removed.
- Water leakage may occur at the joints of the concrete tunnel and steel lining due to difference in deformation. Therefore, a waterstop is inserted or it is designed to absorb the deformation.

11.5 Head Tank and Surge Tank

A head tank and a surge tank are constructed at the intersection of the headrace and penstock or near the upstream end of the tailrace to prevent the adverse effect of water hammer pressure or flow variation due to turbine load fluctuation on the headrace, tailrace and other structures. It is called a head tank and a surge tank when the headrace is a non-pressure type and a pressure type respectively. It is called a tailrace surge tank when constructed in a tailrace.

The head tank consists of a main body, regulating gate, flushing gate, trashrack, trash rake, air supply pipe and other peripheral facilities. It is also equipped with a water level recorder, warning system and other peripheral equipment.

11.5.1 Head Tank

A head tank supplies water when a power plant load increases rapidly and diverts excess water when the load decreases. The head tank regulates the difference between a penstock flow and a headrace flow due to the load change. It also settles and flushes sediments from the running water to prevent troubles to the penstock and the turbine.

The functional conditions of the head tank are shown below.

(1) The head tank capacity must be large enough to continue operation for 1 to 2 minutes at maximum plant discharge without supply of water from the headrace.

(2) The water surface area of the head tank must satisfy the following against rapid change of water surface or wave action during normal operation.

$$A/Q \geqq 50$$

where,

A : Storage area (m^2)

Q : Maximum plant discharge (m^3/sec)

(3) Water level fluctuation against load fluctuation must be within the permissible upper and lower limits.

Consideration must be given to the following in designing a head tank.

- i) Rapid section change is avoided to prevent a vortex of the river water flowing into a head tank.
- ii) To accumulate and remove settled sediment, a 1:15 to 1:50 sloping gradient is used at the bottom. A groove is provided at its end and a flushing gate is equipped to flush sand.
- iii) The elevation of the intake sill is 1.0 to 1.5m higher than the lowest part of the head tank to prevent sand inflow to the penstock.
- iv) The water depth at the mouth of the penstock is twice or more that of the inside diameter of the penstock to prevent air entrainment into the penstock with a vortex.
- v) The spillway channel is designed so that the maximum plant discharge can be safely discharged at the water level to the extent that no pressure is exerted in the headrace when all loads are shut down.
- vi) The velocity in the head tank is to be 0.4 to 0.6 m/sec.
- vii) A trashrack is provided in the front of the penstock. Large debris is not anticipated so the spacing of bars is smaller than that installed in the intake.
- viii) A control gate is installed when no inlet valve is equipped at the turbine inlet. The control gate is set at the back side of the screen or directly at the mouth of the penstock. The penstock could be damaged by the negative pressure if the control gate at the mouth of the penstock is suddenly closed and shuts off the flow. To prevent this, an air valve or pipe must be installed at the rear of the control gate.
- ix) In determining the capacity of the head tank 1 to 2 minutes of operation at the maximum plant discharge is considered. However, some examples adopt a small capacity head tank in consideration of power plant operation and turbine closing time, etc.

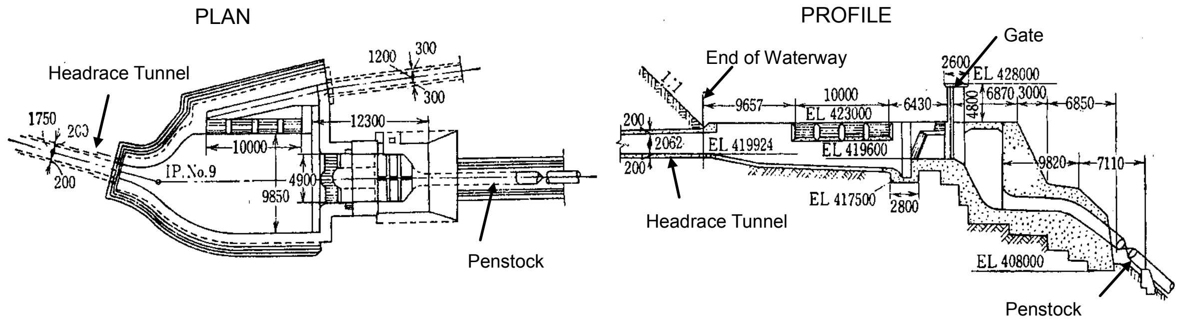


Figure 11-24 Example of Head Tank

11.5.2 Surge Tank

(1) Design condition

The following factors must be considered when designing the surge tank.

i) Stability characteristics: It must return to the state of equilibrium after a lapse of time against water level fluctuation due to load fluctuation.

ii) Shut-down at a full-load condition: The highest water level caused by shut-down of a full load must always be below the top end of the surge tank in the case of the highest water level in the reservoir.

iii) Load sudden increase condition: When the load suddenly increases from a half load to a full load, the lowest water level must be above the crown of the headrace in the case of the lowest water level in the reservoir.

iv) Attenuation characteristics: Water level fluctuations in the surge tank during normal operation should be of a degree not to affect flow regulation of a turbine.

When operation of automatic frequency control (AFC) and automatic load regulation (ALR) is adopted for the hydro power plant in the power system, sudden load increase and decrease is repeated at all times. Resonance occurs when the load fluctuation synchronizes with the water surface vibration of a surge tank and extraordinarily large surging may, therefore, occur. In the case of a pumped storage power plant, it is necessary to study a pump trip as well as an input shut-off condition. A down surge when input to the pump is shut down during pumping and the valve is kept fully open becomes a very severe condition, which is peculiar to a pumped storage power plant.

(2) Types of surge tank

A simple type, orifice type, differential type, chamber type, etc. are used for surge tanks.

i) The simple type surge tank is the simplest to construct and directly connects the vertical shaft with the headrace. A water surface rise and fall is gentle and the water hammer pressure in the penstock is significantly reduced. However, this type requires a relatively larger capacity than the other types. Figure 11-25 shows an example of a simple type surge tank.

ii) The orifice type surge tank connects the surge tank and headrace with the regulating port. Water passage through the regulating port according to the changes in plant discharge makes a head loss, thus reducing the fluctuation of the water level in the surge tank. Therefore, the surging damping capacity is good but the water hammer pressure absorption is small. It is noted that a rise of water level when the load is shut off is larger at partial plant discharge than at maximum plant discharge. Figure 11-26 shows an example of an orifice surge tank.

iii) The differential type surge tank is directly connected to the headrace by erecting a riser with almost the same sectional area as that of the headrace. Its construction is more complicated than that of other types. Compared with the orifice type, this type is excellent in that pressure rise can be controlled below the riser height. It also provides better water hammer pressure absorption. Compared with the simple type surge tank, this type can be reduced to 1/1.5 to 1/2 in sectional area and about half in volume. Figure 11-27 shows an example of the differential type surge tank.

iv) The chamber type surge tank consists of a vertical shaft which is connected to water chambers. The vertical shaft has the minimum sectional area to satisfy the stability condition of fluctuation of water level. Water chambers are arranged at the highest and lowest water levels to restrict the amplitude of fluctuation. The sectional area to satisfy the stability condition becomes larger in the case of a low head. This type is, therefore, suitable for medium or high head cases. Figure 11-28 shows an example of the chamber type surge tank.

The above surge tank types are mainly classified from a hydraulic factor and are further classified into an exposed type and an embedded type from the aspects of installation conditions. The exposed type is constructed of reinforced concrete, pre-stressed concrete and steel. The embedded type is mainly constructed of reinforced concrete. If the geological condition is good, part of the internal pressure is taken by the surrounding rock. Steel plate lining may be provided to prevent water leakage.

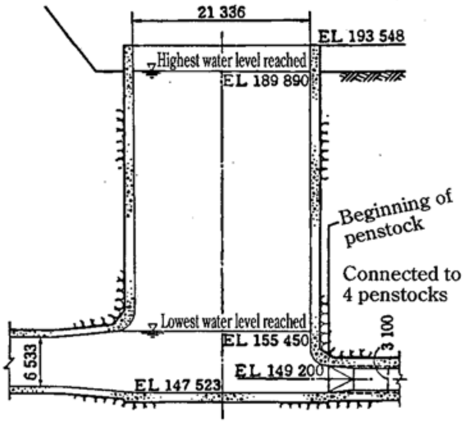


Figure 11-25 Simple Type Surge Tank

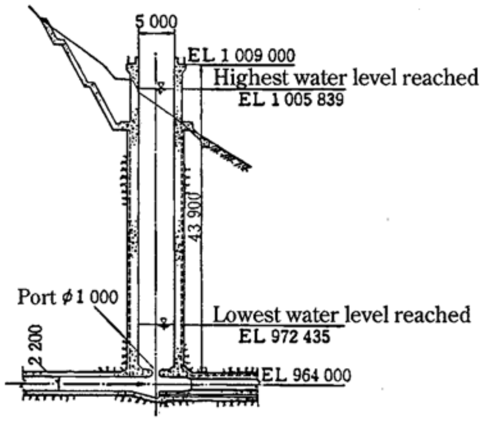


Figure 11-26 Orifice Type Surge Tank

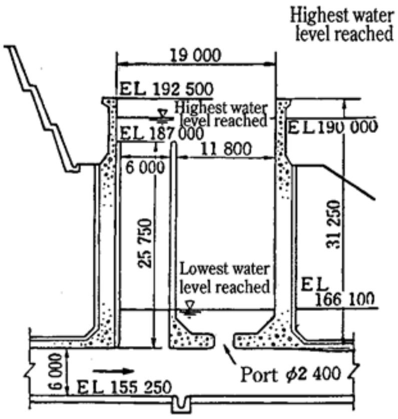


Figure 11-27 Differential Type Surge Tank

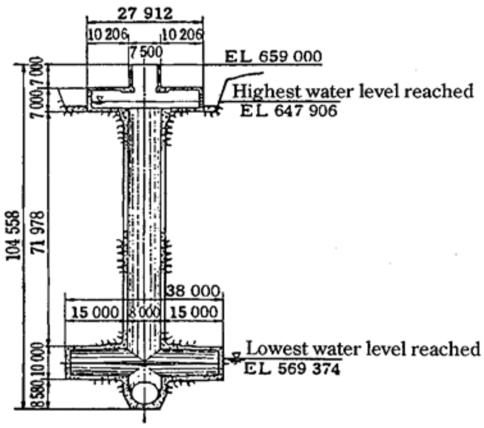


Figure 11-28 Chamber Type Surge Tank

11.6 Penstock

11.6.1 Types and Materials of Penstock

A penstock is classified into an exposed type and an embedded type. The former is exposed on the ground surface as shown in Figure 11-30. The latter is embedded in the ground or bedrock as shown in Figure 11-31. Some exposed steel pipes are installed in a tunnel with no concrete backfill. Embedded penstocks are often used in a dam type or a pumped storage power plant.

Particularly in the pumped storage power plant, the center of the pump turbine must be lower than the low water level of the lower pond due to the suction head required during pump operation, therefore, the powerhouse is an underground type with the penstock embedded in a tunnel.

A steel pipe, reinforced concrete pipe, fiber reinforced plastic (FRP) pipe are used as the material of the penstock pipe. In the case of a large pumped storage power plant, a large diameter and a high head are required of the penstock. Here, the material used is high-tension strength steel of 600N/mm^2 to 800N/mm^2. In addition, up to 100mm thick steel plate has been used up to present.

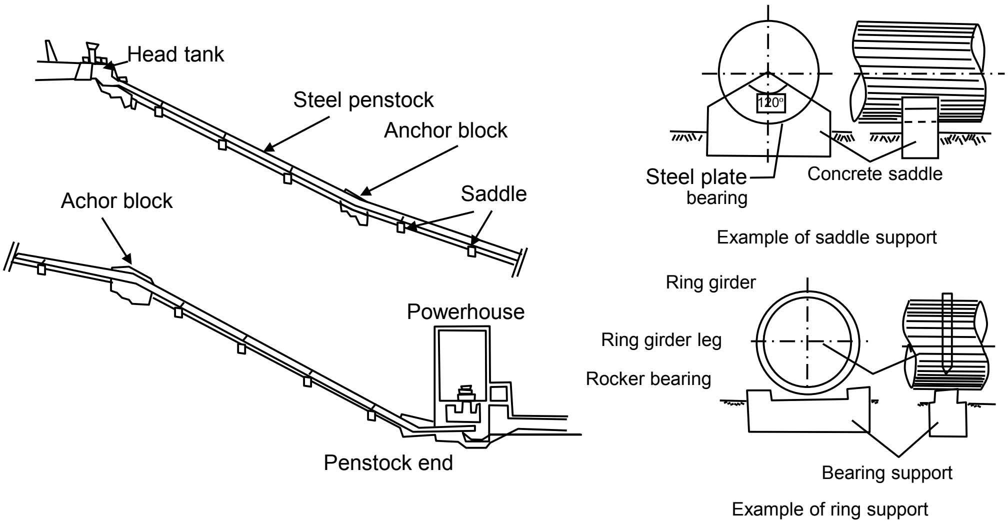


Figure 11-29 Penstock (Exposed type)

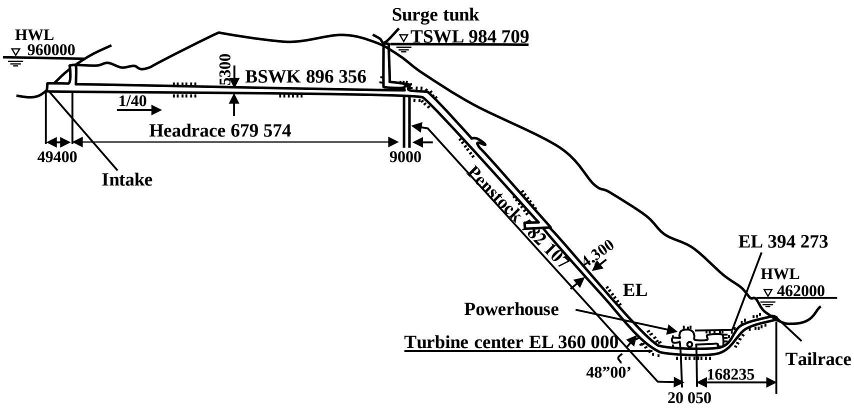


Figure 11-30 Penstock (Embedded Type)

11.6.2 Penstock Alignment

(1) Exposed penstock

It is necessary that an exposed type penstock as shown in Figure 11-30 be planned to run straight over the shortest distance along a ridge with thin overburden which is free from a landslide or an avalanche. The penstock is fixed at its bends with concrete anchor blocks. When steel pipe is used, the anchor blocks are spaced at 50 to 100m intervals.

For a FRP penstock, more bends are provided than for a steel penstock in order to maintain the stability of the natural ground by minimizing the excavation. However, the size of the anchor blocks is smaller than that for steel penstock, because the support of each FRP pipe by a saddle anchor can reduce the thrust on the anchor blocks. It gives a FRP penstock better economical performance.

In the case of a steel pipe, a straight section between the anchor blocks is supported by ring girders or concrete saddles. When saddles are employed, the support interval is about 3 to 5 times the pipe diameter and, in many cases, it is 6m. For ring girders because the rigidity of steel pipe is increased by the reinforcing rings of the ring girders, the interval is 10 to 20 times the pipe diameter and, in many cases, it is 18 m. In determining the length of a unit pipe, the transportation conditions to the installation location and temporary installation conditions are taken into consideration so that the joints should not be set on the saddles.

(2) Embedded penstock

An embedded penstock is employed in locations where an adequate rock cover is secured and the geologic conditions are good. An embedded penstock can be aligned straight with the minimum length to the powerhouse regardless of the topography. It contributes greatly to the economy in comparison with an exposed penstock depending on the location of the powerhouse. In some cases, therefore, an embedded penstock may be substantially more economical.

There are two types of embedded penstock, an inclined shaft and a vertical shaft. The inclined shaft as shown in Figure 11-30 generally has a gradient of 45 degree or steeper to facilitate a fall of excavation muck through the excavated portion of a penstock tunnel.

11.6.3 Design of Penstock

(1) Diameter of Penstock

When selecting the most economical pipe diameter, the change in construction cost due to pipe diameter and change in electric power and energy due to a head loss are taken into consideration in the same way as the headrace cross section is determined.

A velocity in a penstock is usually 2 to 4m/sec. In the case of a power plant with a low head, it is generally more economical to increase the pipe diameter and consequently to decrease the velocity as well as the head loss. In the case of a power plant of a high head, it is generally more economical to decrease the pipe diameter.

(2) Thickness of Penstock

The following formula is applied to determine the thickness of a penstock.

$$t = \frac{PD}{2\sigma} + \varepsilon$$

where,

t	: Thickness
P	: Design head
D	: Diameter
σ	: Allowable stress of material
ε	: Margin for corrosion

1) Exposed penstock

With an exposed penstock, the pipe thickness is designed in consideration of the internal pressure of the pipe filled with water, weight of the steel pipe, weight of the water in the pipe and a temperature change. Other loads such as seismic force, wind pressure and a snow load may also be considered. When the pipe is not filled with water, 0.2 kg/cm^2 is considered as the external pressure working on an empty pipe.

2) Embedded penstock

In the case of an embedded penstock, as the steel pipe is continuously supported by concrete in its axial direction, it is unnecessary to consider the weight of the steel pipe and the water in the pipe. When the pipe is filled with water, internal pressure and a temperature change are studied.

When the rock around the penstock is good, it can be designed that surrounding rock of the penstock supports partially the internal pressure acting on the penstock.

When the pipe is empty, external pressure is studied. To be considered as external pressure are placing concrete and grouting during construction and seepage water of the ground after construction.

(1) Fiber reinforced plastic penstock

A fiber reinforced plastic (FRP) pipe may be preferable to the penstock with low inner pressure and a small diameter less than 3 m in terms of the construction and the maintenance.

The design of a FRP penstock is similar to that of a steel pipe. Because it is an anisotropic compound material, the compound fracture stress must be taken into account in addition to the usual allowable stress in consideration of the creep. A FRP pipe is generally used for an exposed penstock. A fiber reinforced plastic mortar (FRPM) pipe, which is made of the plastic mortar between the inner and outer FRP, is used for an embedded penstock due to its greater rigidity.

11.7 Spillways for Settling Basin and Head Tank

A spillway discharges an overflow from a spillway outlet of a settling basin or a head tank into the river. An open canal, covered canal and pipe conduit (steel pipe, reinforced concrete pipe, fiber reinforced plastic pipe, etc.) are used.

As the spillway is often located on a steep slope, considerations are given to the following items during the design.

- The spillway route and its structure are determined by a survey of the topographic and geologic features from the head tank to the river, together with the surrounding environmental conditions.
- As a flow in the spillway is a supercritical flow in a steeply graded waterway, it could cause an impulse wave or cavitation at bends or at discontinuous slope parts of the waterway, and therefore the waterway should be as straight as possible.
- As the water surface may swell due to entrainment of air, the cross section of the spillway must be designed with due attention to this phenomena.

- In the case of the pipe conduit, an air hole is provided at the bends to replenish the air carried away by high-speed water flow.
- Even if the spillway is an open canal or a covered canal, cut-off projections are constructed on the bottom in the same way as the anchor block of the pipe conduit to prevent sliding as well as erosion by leaking water.
- An energy dissipater is installed at the spillway end to safely discharge water downstream.
- When water from the spillway is discharged directly to the river, attention is paid to its alignment to avoid an adverse effect on the river including excessive scouring of the river bed.
- The spillway for a head tank is generally installed together with the penstock. When a gully near the head tank can be used, the spillway can be shortened by using an open canal.

In the case of a small plant discharge, the headrace section becomes the minimum section of the construction and the headrace has a surplus capacity. In this situation, the empty capacity is used to absorb surplus water when the load suddenly decreases and the spillway may be omitted. For a plant with Pelton turbine, the spillway of the head tank can be omitted by gradual closing of the intake gate and an operation of the deflector of the turbine.

11.8 Powerhouse

A powerhouse consists of a turbine and generator hall, and auxiliary machine rooms which house switchgears, control boards, accessory equipment, etc. Dimensions of the powerhouse depend on the equipment arranged.

11.8.1 Selection of Powerhouse Location

The powerhouse location is decided by considering the following;

- A location having good foundation
- A location not susceptible to flood damage, not directly hit by a flood flow
- A location free from a landslide, an avalanche and a similar potential hazard
- A location convenient for an outdoor switchyard and transmission line arrangement
- A location providing easy transportation for construction materials and equipment and easy future operation/maintenance

11.8.2 Type of Powerhouse

According to its position, a powerhouse is roughly classified into an open type and an underground type. The open type is classified into an indoor type and semi-outdoor type, according to a type of building.

The indoor type powerhouse as shown in Figure 11-31 is more generally used. The turbine and generator are housed in the powerhouse building and are assembled and disassembled by using an overhead traveling crane. In the case of the semi-outdoor type as shown in Figure 11-32, the elevation of the generator room ceiling is set at the ground level. Equipment for the assembly and disassembly of the turbine and generator is carried in and out through an overhead hatch. An outdoor crane is used for assembly and disassembly.

The underground type is shown in Figure 11-37 and explained in 11.8.4.

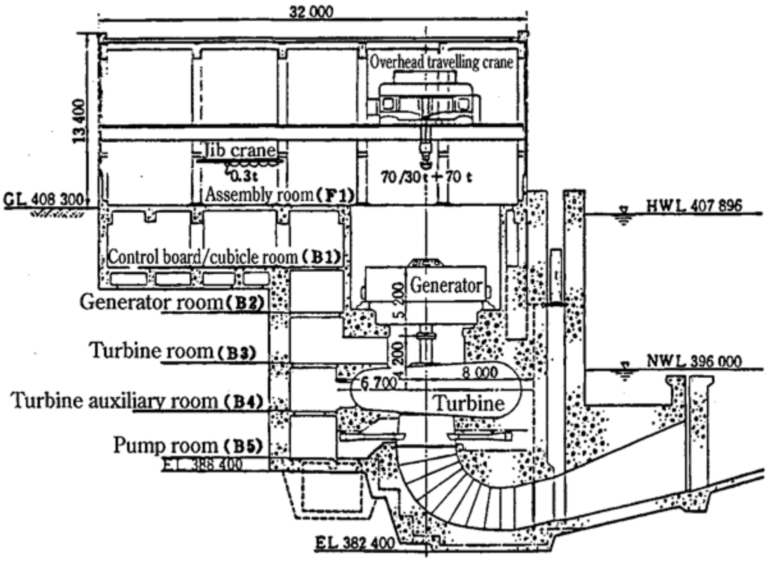


Figure 11-31 Indoor Powerhouse

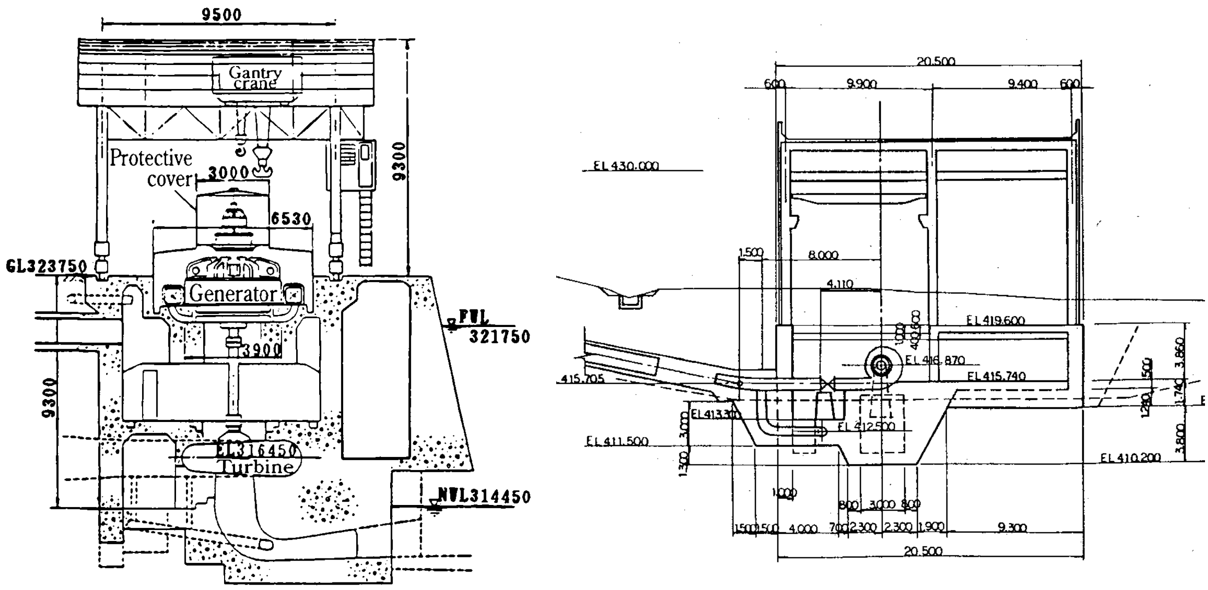


Figure 11-32 Semi Outdoor Powerhouse

11.8.3 Design of Powerhouse

As a general rule, when the revolving speed of the turbine and generator is increased, small, lightweight equipment can be adopted and the size and construction cost of the powerhouse building can be reduced. However, it requires the larger suction head. As a result the substructure of powerhouse becomes deeper and the construction cost increases.

Considering the above features of the powerhouse, a comprehensive study for the design of the powerhouse is necessary taking account of the revolving speed of the turbine and generator, and the construction cost of the powerhouse building and substructure. The turbine revolving speed is determined first, and the powerhouse design is proceeded with as shown below.

- The turbine center elevation is determined from the tailrace water level and the suction head required by turbine characteristics.
- The generator room floor elevation is determined from the required dimensions of the turbine and generator. The height of powerhouse building is determined by the lifting height of the overhead crane necessary for unloading, assembly and disassembly of the turbine and generator.
- Plan dimensions of the powerhouse are determined after full considerations of the layout of the main and auxiliary equipment as well as the operation, maintenance, and installation work.

A powerhouse substructure varies greatly depending on the turbine type. It must be capable of supporting the steel penstock, turbine and generator, fixing the draft tube and withstanding the vibration caused by the operation. The turbine and the generator are heavy weight equipment and must be supported adequately. Barrel support, which is a vertical cylindrical structure with thick wall, and beam support are commonly adapted to large-scale powerhouse and small-scale powerhouse respectively. These examples are shown in Figure 11-33 and Figure 11-34.

A structure of a large-scale powerhouse is often a multistory type having many floors. Each floor is designed to have the strength required to bear the load of various equipment. All rooms are fully ventilated and evacuation routes in the event of an accident or an emergency are fully considered.

To minimize the size of the powerhouse building, a study may be done, the items of which are shortening of the generator shaft, increasing the turbine revolving speed, improvement of the crane, and the rational combination of accessory equipment.

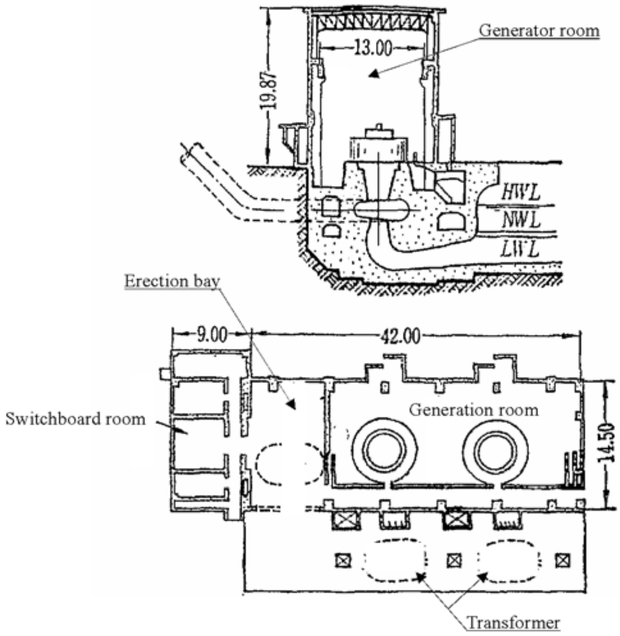


Figure 11-33 Barrel Type Support

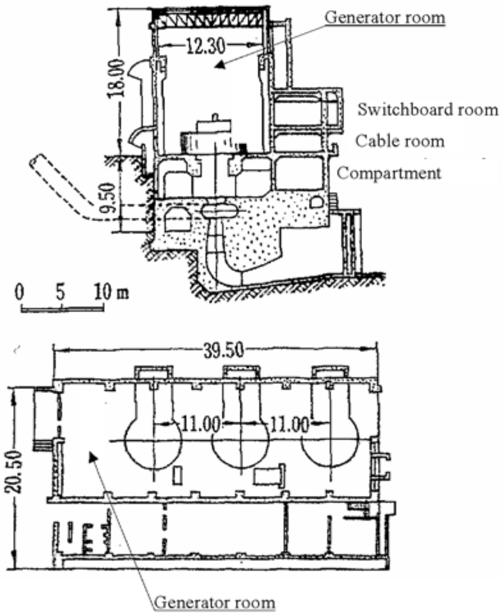


Figure 11-34 Beam Type Support

11.8.4 Underground Powerhouse

(1) Features of the underground powerhouse

The features of an underground powerhouse are shown below.

- The location can be selected as desired without limitation of topography and the head can be fully utilized. The underground powerhouse is often advantageous when the turbine center elevation must be low in order to secure a suction head, particularly in the case of a pumped storage power plant.
- There is no damage by snow and ice in cold regions. It is safe against landslides.
- No structure appears on the ground surface and the natural environment is hardly influenced.
- Construction work can be done throughout the year without regard to climate or temperature.
- Because a large cavern is constructed in the ground, the construction cost is controlled by the geologic conditions.
- Special consideration must be given to ventilation, drainage and lighting both during construction and after completion.
- A tunnel to transport electric equipment and materials or to transmit generated power to the outgoing switchyard is required.
- Because large-scale underground excavation is necessary, the construction cost of the

powerhouse is high.

(2) Arrangement of underground powerhouse

There are three types of arrangement of an underground powerhouse, a head type, a tail type and an intermediate type, according to its location in the total waterway system.

The head type powerhouse is constructed near the intake and has a relatively long tailrace tunnel. The headrace surge tank may be omitted. However, to meet load fluctuation the tailrace tunnel section must be increased or a tailrace surge tank must be installed at the starting point of the tailrace tunnel when the tailrace tunnel is long. In many cases of this arrangement, the access tunnel to the powerhouse may become a long inclined or vertical shaft. Figure 11-35 shows an example of the head type powerhouse.

The tail type powerhouse is installed near the outlet and has a relatively long headrace tunnel. The surge tank for headrace is constructed, but in many cases the surge tank for tailrace is omitted. The access tunnel to the powerhouse may be short.

The intermediate type combines the features of the head type and tail type. The headrace surge tank and tailrace surge tank may be either retained or omitted.

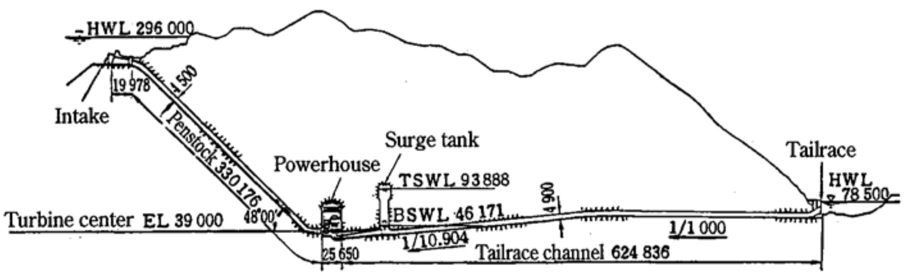


Figure 11-35 Head Type Powerhouse

(3) Shape and Size of Underground Powerhouse

The underground powerhouse shown in Figure 11-36 is roughly classified according to its cavern cross section into the type having an arch-type ceiling and vertical side walls (Mushroom shaped), and the type having an egg shaped cross section or a bullet cross section with the ceiling arch and side wall forming a continuous line. The selection of the cross section depends on the dimension of the powerhouse and the mechanical properties of surrounding rock. Due to the development of a supporting technology, such as a NATM and a PS anchor method, and an analytical method of underground caverns, an oval type and a bullet type are commonly aplied to undergournd powerhouses, of which cross area exceeds 1500 m^2.

Thorough investigation must be made to select the powerhouse location where the geologic structure is good with little faults. The layout must be determined by considering the direction of the main joint, dominant ground pressure, etc.

The size of the building shown in Figure 11-37 is determined by the capacity and number of units of the generator, the layout of the auxiliary equipment, etc. in the same manner as in the case of an above ground powerhouse. Less excavation, particularly for the underground powerhouse, results in less construction cost and construction safety. Therefore, the position of the main machinery and transformers and other various equipment must be fully studied. The main transformer is installed in the same cavern as the main machine hall or in an adjacent cavern for itself. The switchyard is installed outdoors in many cases, but may be installed underground according to topographic and geologic conditions, weather conditions, environmental problems, etc. When another cavern is constructed apart from the carven of the turbine and generator hall, attention must be paid to the distance between these caverns. If the distance is too small, they will affect each other and the damaged area between these in terms of rock properties is amplified as compared with a single cavern.

The turbine and generator erection bay is located at one end or at the center of the turbine and generator hall. When it is located at the center, the bed rock of the erection bay forms a type of island between the open spaces on both sides. Therefore, a form of strut effect can be expected to support the side walls.

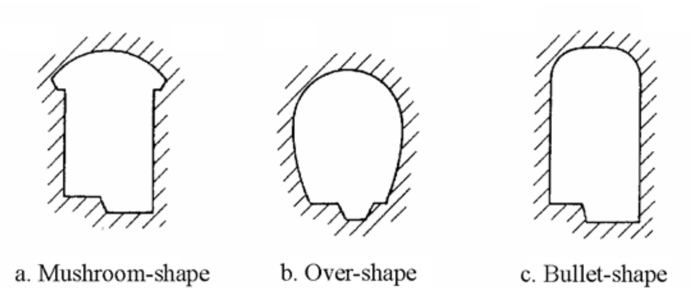


Figure 11-36 Sections of underground powerhouse

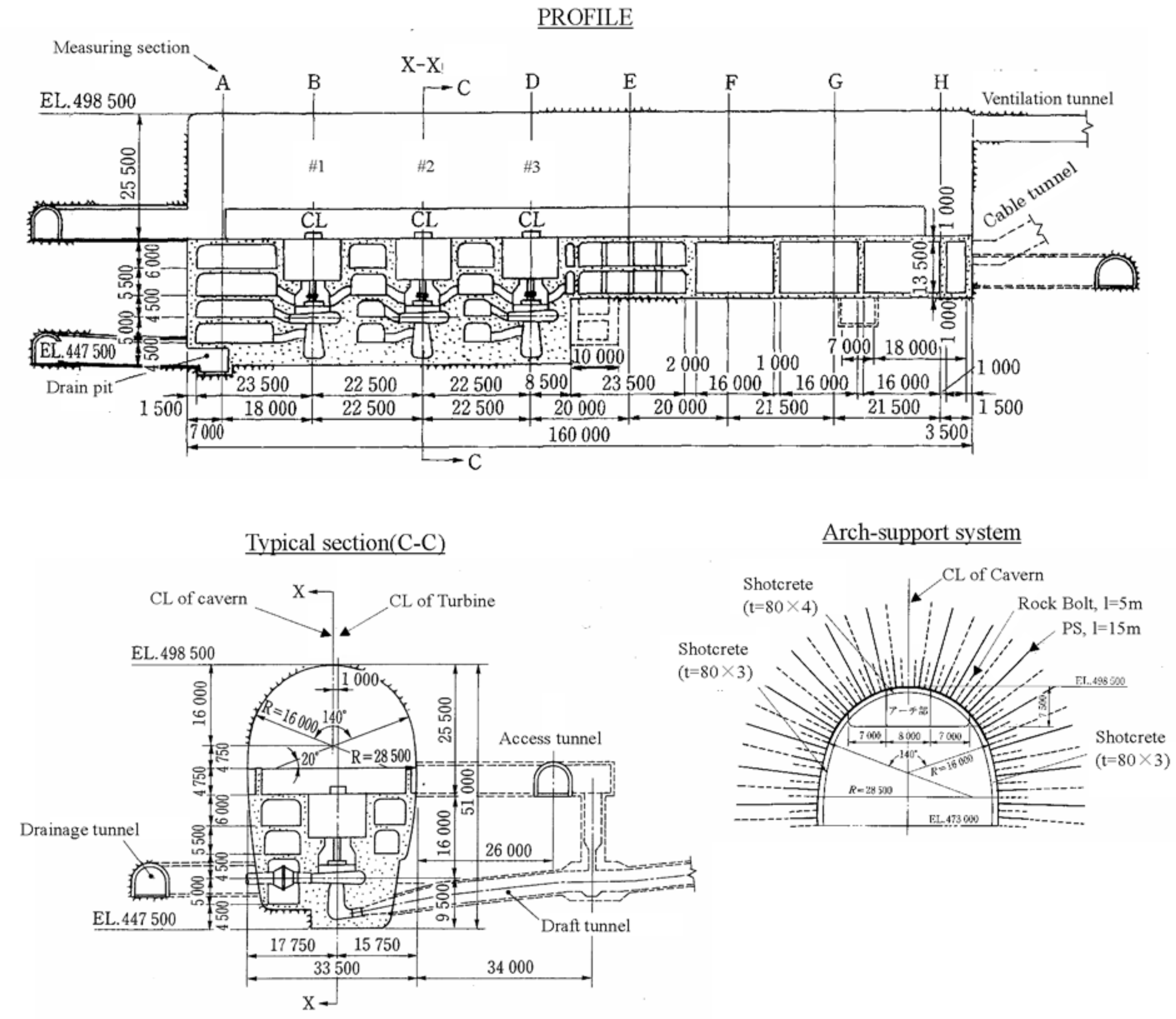


Figure 11-37 Example of Underground Powerhouse (Imaichi PS in Japan)

(4) Design of Supports

An underground powerhouse carries a load caused by the excavation. The load on the underground powerhouse concentrates and becomes large due to the large size of the carven. Therefore providing adequate supports on the ceiling and wall of the cavern is essential to secure the stability of the surrounding rock of the underground powerhouse.

To design the supports of the underground powerhouse, the deformation and the damaged area of the surrounding rock, of which mechanical properties are deteriorated due to excessive stress by the excavation, should be studied taking the process of stepped excavation into account. For this purpose, a two-dimensional FEM analysis is commonly applied. A seismic load is not considered in this study because of the underground structure having less effect of the seismicity.

Shotcrete, rockbolts and PS anchors are adapted to the support members of the surrounding rock. In addition, cable bolting, using PC strands embedded in the surrounding rock with mortal grouting like rockbolts, is promising support from an economical point of view. The specifications, such as spacing, length and preset stress, are determined according to the above study. Reinforced concrete (RC) may be applied to the lining concrete of the cavern. Instead of RC lining, shotcrete with steel fiber is sometimes applied with a larger thickness.

11.9 Tailrace and Outlet

A tailrace is a part of waterway which guides the water discharged from the turbine to the river. It consists of a tailrace bay, a tailrace and an outlet. The water level of the tailrace depends on the outflow from the turbine and the water level of the river during a flood. The Pelton turbine does not use a draft tube and the nozzle center position should be set 30cm higher than the water level of the river during flood. Both the Francis turbine and Kaplan turbine use a draft tube and the tailrace level is set not lower than the outlet of the draft tube, even at minimum plant discharge.

A tailrace bay is provided near the upstream end of a rather long tailrace as a tail tank or a kind of a mini-surge tank to mitigate the adverse effect due to the change in power discharge. An enlarged section some meters long is designed as appropriate.

The velocity in the tailrace is usually set at 1 to 2 m/sec.

An outlet is the exit to the river and is protected with concrete and/or wet masonry from the river flow and sediment according to surrounding topographic features.

The tailrace location must be selected in consideration of the following factors.

- There is no possibility of the exit being blocked by sedimentation.
- The river flow does not directly strike the tailrace.
- The water level does not rise sharply during a flood and there is no river-bed movement by floods. There is no possibility of the tailrace being damaged by floods.
- The river width does not decrease near the downstream of the tailrace.

In the case of a pumped storage power plant, a tailrace becomes an intake during pumping and the flow of water is reversed from that during power generation. Therefore, the same considerations as to the intake must be given to the tailrace.

Reference of Chapter 11

[1] Guide Manual for Development Aid Programs and Studies of Hydroelectric Power Projects, New Energy Foundation, 1996

[2] Fill dam engineering, Japan Electric Power Civil Engineering Association

[3] Design and construction of underground structures of power facilities, Japan Electric Power Civil Engineering Association

[4] Construction of multipurpose dam, Dam Engineering Center

[5] Technical Guideline of Grouting, Japan Institute of Construction Engineering

Chapter 12

Design of Electro-mechanical Equipment

Chapter 12 Design of Electro-mechanical Equipment

12.1 Selection and Design of Turbine

Design parameters such as water levels of intake and tailrace (Maximum, Normal, and Minimum), and maximum generating discharge, etc. are to be selected as shown in Table 12-1 followed by general design for electro-mechanical equipment. The following describes selection and design methods for turbines, generators, auxiliary equipment and electrical circuits.

Table 12-1 Design Parameters for Electro-Mechanical Equipment

Item	Unit	Planed		
Maximum generating discharge water	m^3/sec			
Normal effective head	m			
Number of Unit				
Power System Frwquency	Hz			
		Maximum	Normal	Minimum
Water level of Intake	EL. m			
Water level of Tailrace	EL. m			

12.1.1 Classification of turbine

Turbines to be applied for small scale hydropower projects can be classified, according to fundamental mechanism of utilizing water energy, into an impulse turbine and a reaction turbine. Turbine classification can be further organized as shown in Figure 12-1. The electrical designer selects a turbine that can produce as much generating electricity as possible taking into consideration the effective head and available maximum discharge of the project site.

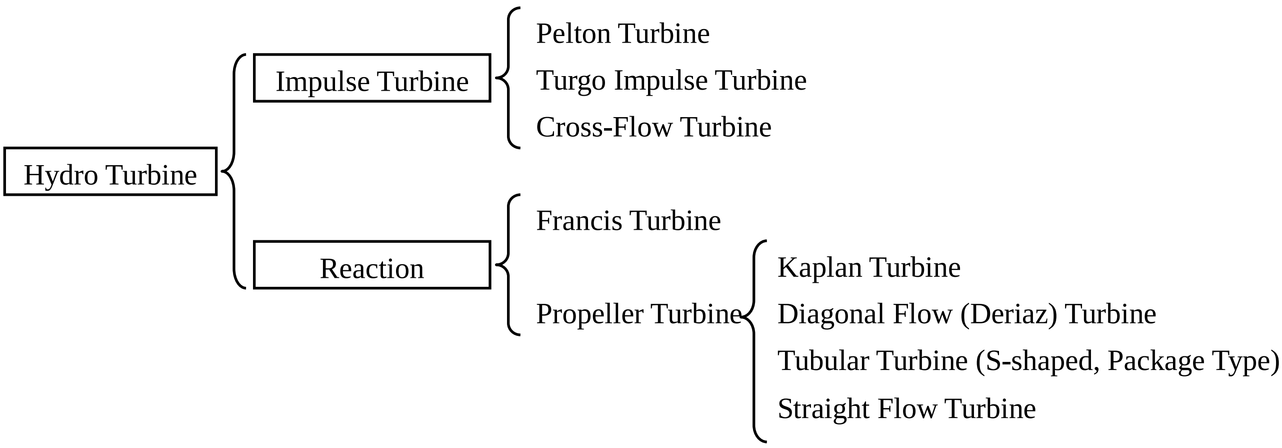


Figure 12-1 Classification Tree of Turbine

(1) Impulse turbine

In an impulse turbine, water having the pressure head is ejected from the nozzles to convert all of its energy into the velocity head, and the turbine runner is driven by the velocity of the water jet.

(2) Reaction turbine

In a reaction turbine, water having the pressure head acts on the turbine runner and the water pressure drives it directly.

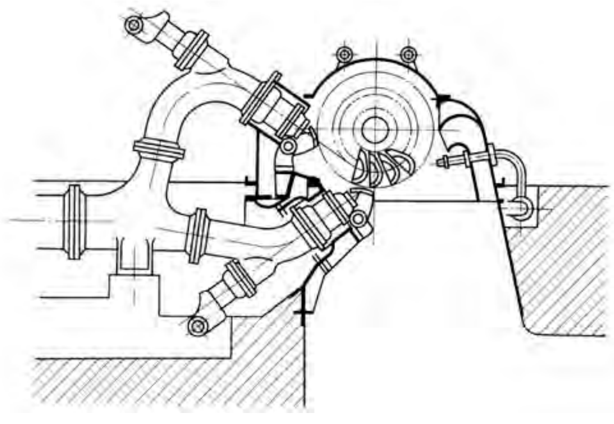
(3) Pelton turbine

Pelton turbine is classified an impulse turbine. The Pelton turbine has been invented by the American mining engineer Pelton in 1870 and now is the most popular turbine for the high head hydro power project. The turbine consists of one or more free jets of water discharging into an aerated space and impinging on a set of buckets attached around periphery of a disk. The buckets vary in some details of their construction, but in general are bowl-shaped and have a central dividing wall, or splitter, extending radially outward from the shaft. This splitter divides the stream and the bowl-shaped portions of the bucket turn the water back, imparting the full effect of the jet to the runner. The free jet is formed by the water passing through the nozzle pipe, the needle nozzle, and thence through the nozzle tip.

The size of the jet and thus the power output of are controlled by a needle in the center of the nozzle and needle tip. The movement of the needle is controlled by the governor. A jet deflector is located just outside the nozzle tip to deflect the jet from the buckets to effect sudden load reductions.

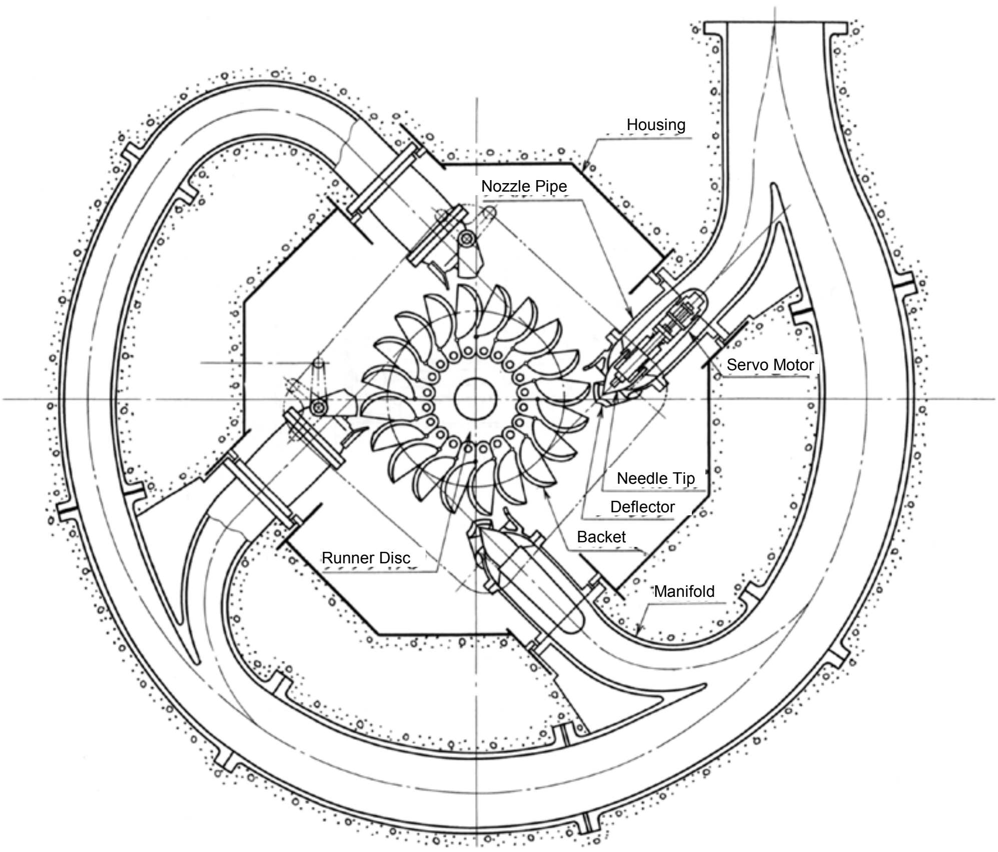
Pelton turbines are generally utilized when the head is too high for the practical use of Francis turbines, which is normally applied to the head height of 200m above. They have been installed in projects having heads as high as 1,750m. Pelton turbine is roughly as follows.

Generating output	: 100～315,000 (kW)
Discharge	: 0.2～55 (m^3/sec)
Effective Head	: 75～1,300 (m)



Source: JIS-B0119 Hydraulic turbine and reversible pump turbines-Vocabulary

Figure 12-2 Horizontal Pelton Turbine（2 Jet Nozzle）



Source: JIS-B0119 Hydraulic turbine and reversible pump turbines-Vocabulary

Figure 12-3 Vertical Pelton Turbine (4 Jet Nozzle)

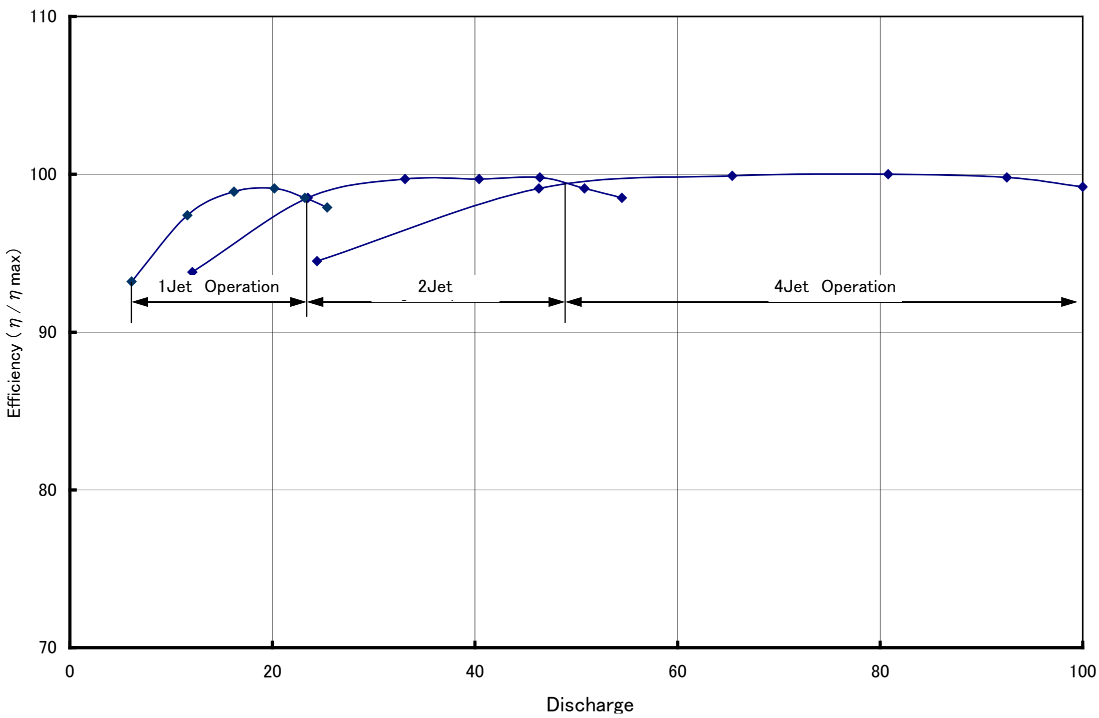


Figure 12-4 Efficiency vs. Discharge of Pelton Turbine (at 1 to 4 Jet Operation)

The horizontal Pelton turbine has generally one or two nozzles. Even if the turbine has been designed as a 2 jet nozzle type, a single nozzle operation technology is developed in response to a low river flow in the dry season, which ensures efficient operation at low flow rate as well.

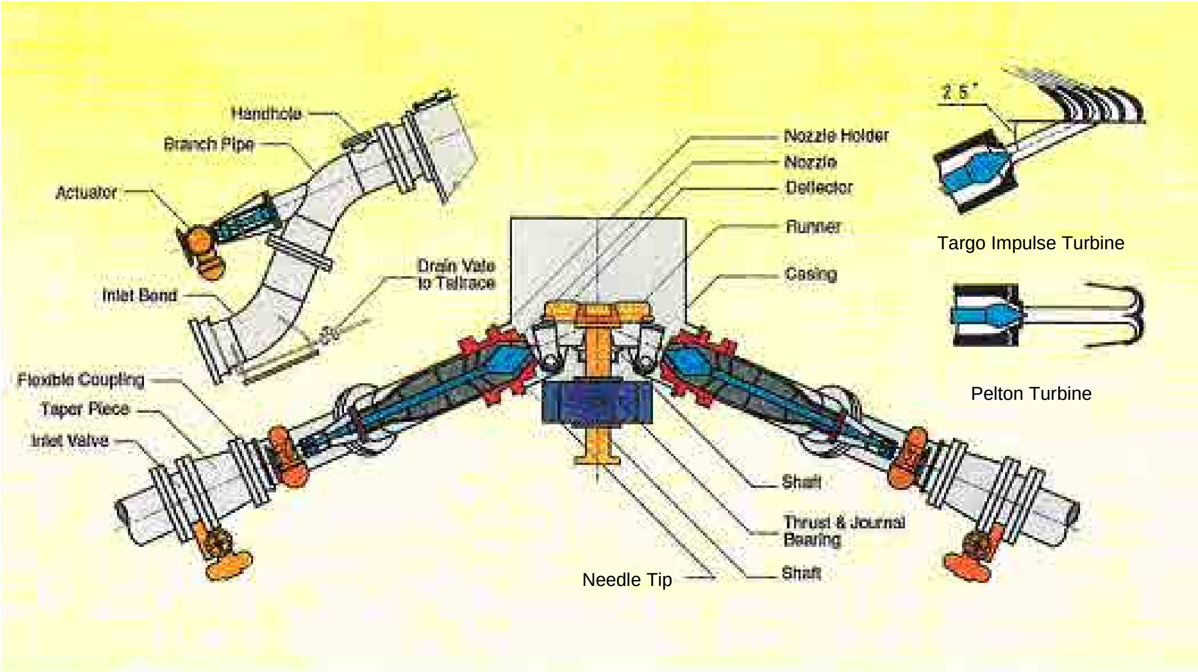
Water jetted from the nozzles collides with the Pelton turbine runner in the air. Therefore, the installation height of the Pelton runner must always be higher than the tailrace water surface. The difference between the tailrace water surface and the Pelton turbine runner center is a head loss and it is not utilized for generation. It is advantageous on this point to set the installation height of the turbine as closer to the tailrace water surface as possible, however, it is necessary to keep a safe height not to have the turbine runner inundated during floods.

The Pelton turbine equipped with the deflector, in a case where the control or shut-off of the generator output is required due to a sudden load change or a transmission line failure, can respond to it with the deflector as well by changing the direction of the jet water acting on the turbine runner, by which a similar effect to that by closing the nozzle is achieved. It is also possible for the Pelton turbine to close the plant discharge rather slowly using the deflector. This will mitigate a pressure increase inside the penstock as well as a revolving speed increase in the turbine when the plant discharge is cut off rapidly, which are the problems with the Francis turbine. In addition, a technology to omit the spillway in a head tank has been also developed through the deflector discharge for a considerable period of time. It is said that the Pelton turbine is suitable for a run-of-river hydro generating scheme that has a large effective head and a large change in river flow.

(4) Turgo impulse turbine

A Turgo impulse turbine is classified into an impulse turbine which a British turbine manufacturing company, Gilkesu (Gilbert Gilkes & Gordon Ltd.) has developed. The turbine is composed of a runner and nozzles as well as a Pelton turbine. Water jets from the nozzles are diagonally sprayed on three or four blades simultaneously at a flat surface of the runner to drive the turbine runner. Shown in Figure 12-4 is the turbine structure including the relationship between the nozzle and the runner compared with a Pelton turbine. This turbine is used for a hydropower scheme with a little lower head than that for a Pelton turbine. The coverage of a Turgo impulse turbine is roughly as follows.

Generating output	: 100～10,000 (kW)
Discharge	: 0.2～8.0 (m^3/sec)
Effective Head	: 75～400 (m)



Source: Gilkes Co.ltd. TURGOIMPULSE TURBINE Catalog

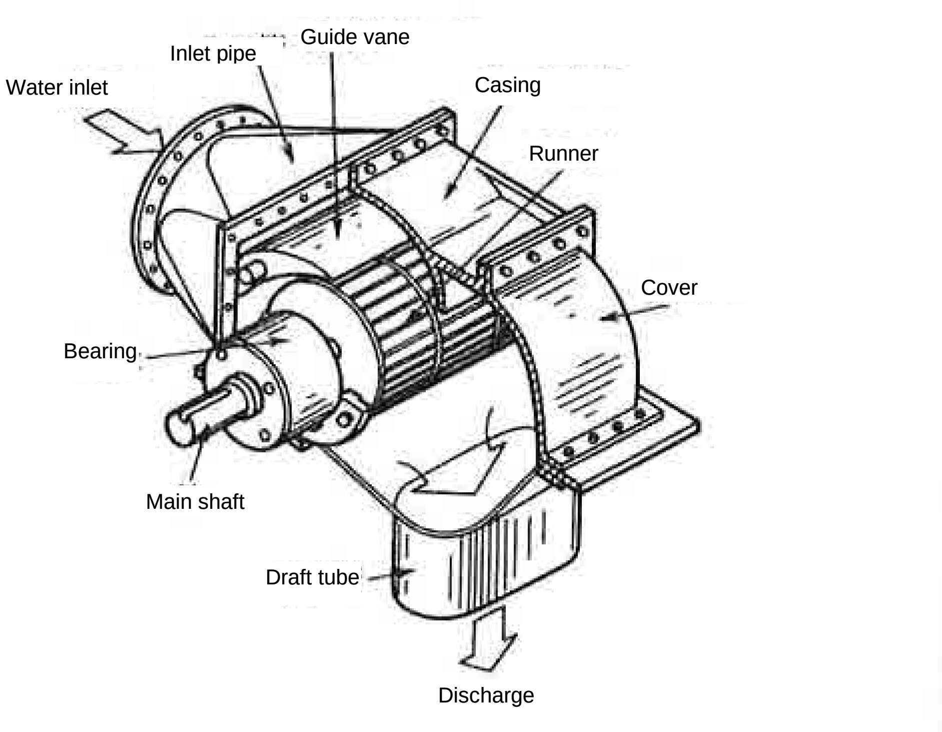
Figure 12-5 Turgo Impulse Turbine（Comparison with Nozzle Flow of Pelton Turbine and Turgo Impulse Turbine）

A Turgo impulse turbine, as compared with a Pelton turbine with the same head and the same discharge, can set a higher revolving speed, which decreases not only the turbine dimension and weight but also the generator weight. As a result it is probable to reduce the overall cost. This turbine, like a Pelton turbine, enables highly efficient operation at a partial load by switching the nozzles, reduction of the water hammer pressure and rotation speed due to a sudden closure of the plant discharge, and the omission of the spillway in a head tank through the deflector discharge.

(5) Cross flow turbine

A cross flow turbine is simple in structure and easy in production to be applied to many cases of small scale hydropower development. This turbine is also called Ossberger turbine from the name of the manufacturer developed. This turbine is suitable for a run-of-river type hydropower plant with the medium to low effective head, and the low plant discharge with a large change in river flow. The coverage of cross flow turbines is roughly as follows.

Generating output	: 50～1,000 (kW)
Discharge	: 0.1～10 (m^3/sec)
Effective Head	: 5～100 (m)

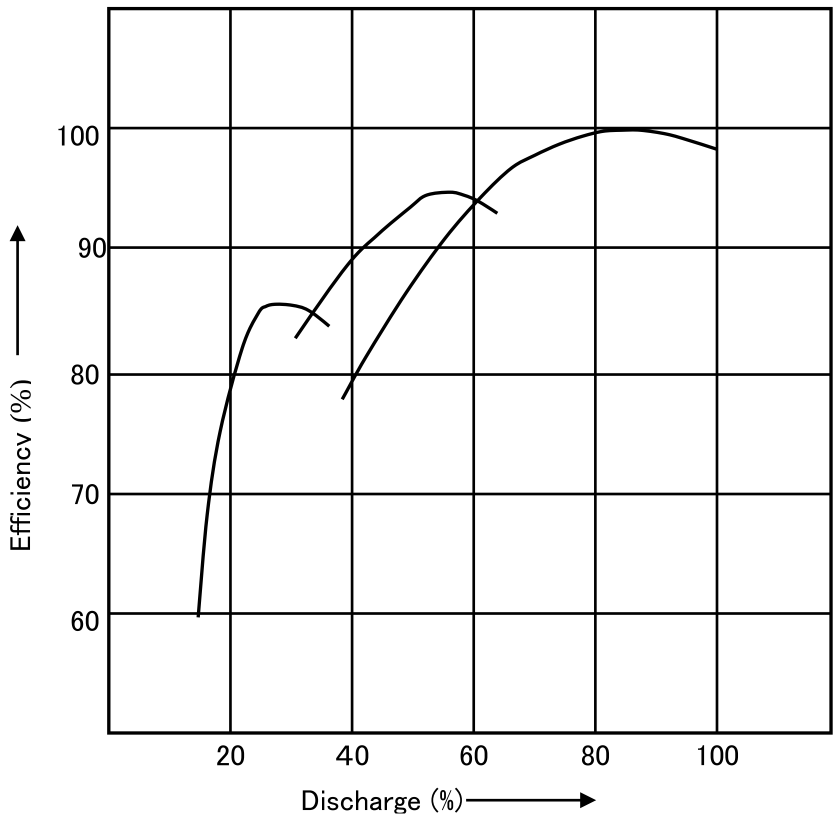


Source: NEF Medium and small scale hydropower guidebook (5th revised edition)

Figure 12-6 Cross Flow Turbine

A cross flow turbine is featured by easy operation and maintenance from its simple structure, improved efficiency during partial operation with the two-part split guide vanes, and an inexpensive price for the equipment. A water flow from the penstock is adjusted in quantity by the guide vanes and acts on the runner blades before and after running through the center of the runner. The guide vanes 1/3 and 2/3 split are manipulated individually in response to the flow and load. As a result, keeping high efficiency for a wide range of the discharge as shown in Figure 10-6, the cross flow turbine can be operated as well as the Pelton turbine under the nozzle switching operation. However, the maximum efficiency of cross flow turbine is approximately 80% regardless of the specific speed.

A cross flow turbine is classified into an impulse turbine and the installation height of the runner must always be higher than the tailrace water surface like a Pelton turbine. The runner length and diameter ratio (aspect ratio) of the cross flow turbine is larger in proportion to the specific speed. Therefore, if a designer selects a large specific speed for the cross flow turbine, the long runner may pose the problems of deflection etc., which needs to be carried out carefully.



Source: NEF Medium and small Scale hydropower guidebook (5th revised edition)

Figure 12-7　Cross Flow Turbine Efficiency with Two Guide Vane Paces

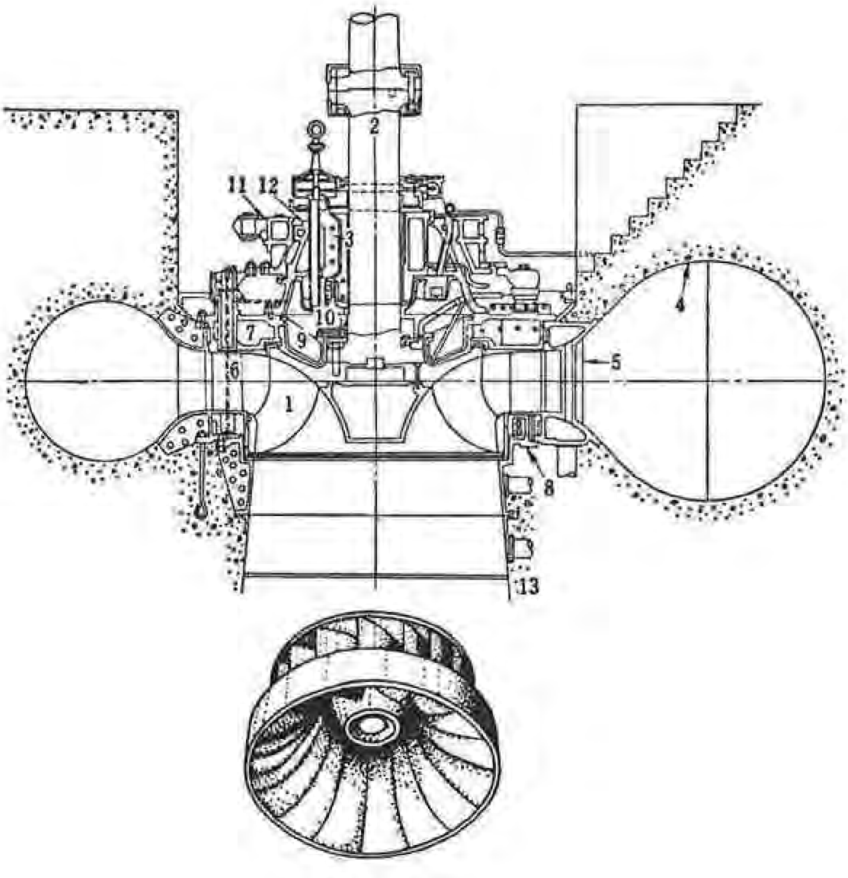
(6) Francis Turbine

Francis turbine is the most popular hydro turbine. The Francis turbine was invented by the American engineer Francis in 1855. This turbine is now most popular one by the high efficiency at the full power and the simple mechanical construction. The strucyure is shown in Figure 12-8.

Water enters the spiral case from intake passages or penstocks, passes through the stay ring, guided by the stationary stay-ring vanes, thence through the movable guide vanes, through the runner and into the draft tube, through which it flows into tailrace or re-regulating reservoir. The movable guide vanes with axis parallel to the main shaft control the flow of water to the runner and thereby control the power output of the turbine.

Francis turbine runners usually have the upper ends of the runner vanes attached to a crown and the lower ends attached to a band, thus completely enclosing the water passageway through the runner. This turbine is used for the wide range from a high head to a low head scheme and large to small capacity scheme. The coverage of Francis turbines is roughly as follows.

Generating output	: 200～715,000 (kW)
Discharge	: 0.4～700 (m^3/sec)
Effective Head	: 15～500 (m)



Source: NEF Medium and small scale hydropower guidebook (5th revised edition)

Figure 12-8 Vertical Francis Turbine

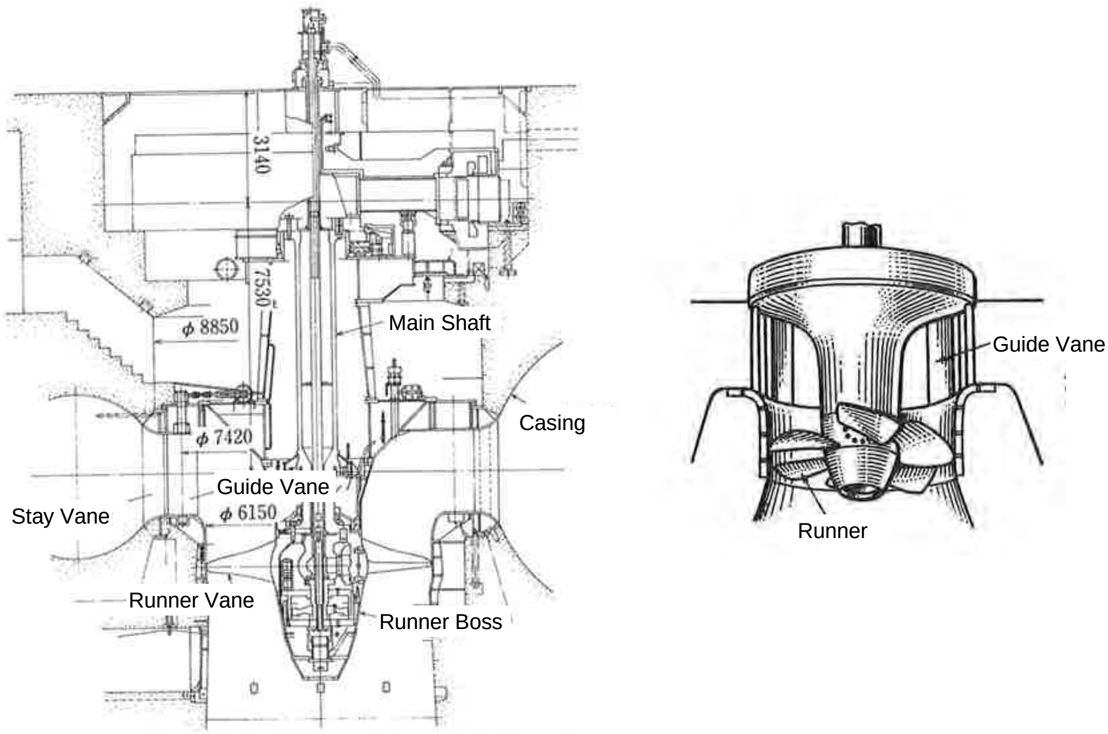
(7) Propeller turbine

A propeller turbine is applied in an area of 80m or less low effective head. The shape of runner looks like a screw of ship, and the runner vanes vary from 3 to10 in less number with the lower head. Water flows axially and drives the turbine runner. A propeller turbine is classified into a Kaplan turbine, a diagonal-flow turbine, a tubular turbine, and a straight flow turbine in terms of structure.

1) Kapran turbine

A Kaplan turbine is a type of a propeller turbine which can adjust correlatively the opening angles of guide vanes and runner vanes so that optimal efficiency can always be obtained in response to the effective head and plant discharge. This turbine is applied to a range of head from 5m to 80m. Most of the propeller turbine is a Kaplan turbine, some of which exceed 200,000 kW. The coverage of Kapran turbine is roughly as follows.

Generating output	: 5,000～250,000 (kW)
Discharge	: 0.4～1,300 (m^3/sec)
Effective Head	: 6～130 (m)



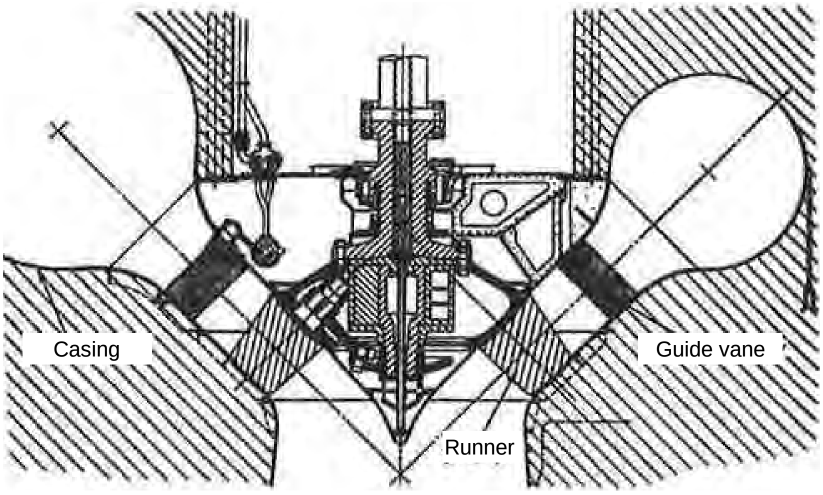
Source: Hydro turbine

Figure 12-9 Propeller Turbine (Kapran Turbine)

2) Diagonal-flow turbine

Another type of a propeller turbine, in which the axis of the runner blades is diagonal with the main shaft, is called diagonal flow turbine. The runner blades and the guide vanes are generally adjustable like Kaplan turbine in response to the effective head and discharge. This turbine is applied to approximately 40 to 130m of effective head, some of which exceed 50,000 kW. The coverage of Diagonal-flow turbine is roughly as follows.

Generating output	: 5,000～80,000 (kW)
Discharge	: 6～150 (m^3/sec)
Effective Head	: 50～130 (m)



Source: NEF Medium and small scale hydropower guidebook (5th revised edition)

Figure 12-10 Diagonal-Flow Turbine

3) Tubular turbine

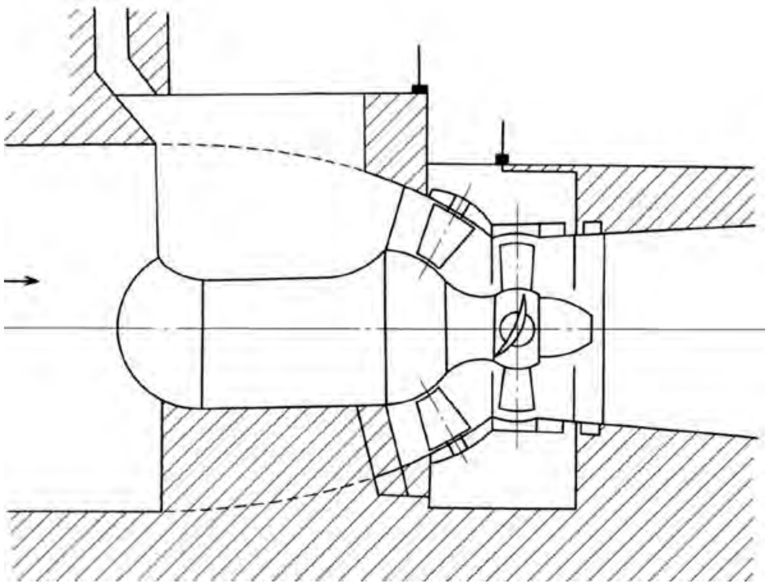
One of propeller turbines that has a cylindrical casing (tubular) instead of a spiral casing is called a tubular turbine. This turbine is applied to the range of low effective head from 3 to 25m, the output of which varies from a few hundred kW to more than 60,000kW. The large capacity tubular turbine is equipped like Kaplan turbine with an adjustable system of guide vanes and runner blades in response to the effective head and plant discharge.

(a) Bulb turbine

This is one of tubular turbines which installs a generator inside the bulb in a waterway. This turbine is often applied to the low effective head generating scheme, the output of which varies from a few hundred kW to more than 60,000kW. The coverage of a bulb turbine for a small scale hydropower scheme is roughly as follows.

Generating output	: 100～70,000 (kW)
Discharge	: 3.0～400 (m^3/sec)
Effective Head	: 3～30 (m)

A diameter and a flywheel effect of the generator are restricted due to its installation in the bulb in the waterway. In addition, a cooling system may be required in order to release the heat from the generator.



Source: JIS-B0119 Hydraulic turbine and reversible pump turbines-Vocabulary

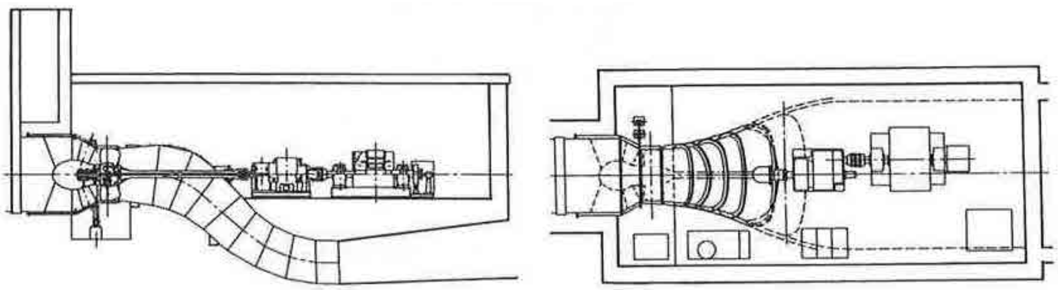
Figure 12-11　Bulb Turbine

(b) S shaped tubular turbine

S shaped tubular turbine is to be applied for the scheme of large change in the water flow and low effective head, the coverage of which is shown below.

Generating output	: 50～20,000 (kW)
Discharge	: 1.5～70 (m^3/sec)
Effective Head	: 3～36 (m)

S shaped tubular turbine has a S shaped waterway in order to install the generator outside the waterway. The runner part is the same structure as other tubular turbines. But the turbine shaft penetrates the channel to be connected to the generator located above the draft tube with S shaped bend. This configuration has advantages in easy maintenance and easy installation works for flywheel, speed-up gear, etc., and almost no restriction on the generator design, compared with other tubular turbines to have a generator installed in a tube or a bulb.



Source: NEF Medium and small scale hydropower guidebook (5th revised edition)

Figure 12-12　S Shaped Tubular Turbine

(c) Vertical shaft tubular turbine

A vertical shaft tubular is a S shaped tubular turbine installed vertically (or diagonally), and the basic performance and turbine structure are the same. The application area of the turbine is shown below.

Generating output	: 1,000～30,000 (kW)
Discharge	:25～150 (m^3/sec)
Effective Head	:5～25 (m)

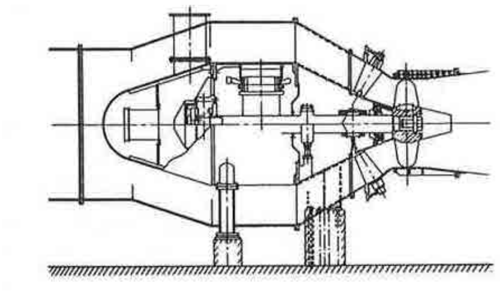
By adopting the vertical shaft tubular turbine, a generator and a speed-up gear can be placed on the top of the cylindrical casing, which may reduce the size of power plant building. In recent years, there has appeared an example of a new technology where the bulb turbine coupled with the elbow draft tube is installed in a vertical position for the purpose of reducing the power house dimensions.

(d) Package Bulb turbine

A package bulb turbine is classified into a Bulb turbine, and they are the same structure. A package bulb turbine is applied to a small capacity bulb turbine. Here, a channel, a bulb, a generator, a turbine, and support structures are integrally assembled into a unit in a factory, which is to be joined to the waterway with a flange in the construction site. Thus, the package bulb turbine is aimed at shortening the construction period, streamlining civil works, and saving installation space. It is suitable for a relatively low head and large discharge scheme and is applied to the idle head of water lines such as waterworks, industrial water service, etc. The application area of the turbine is shown below.

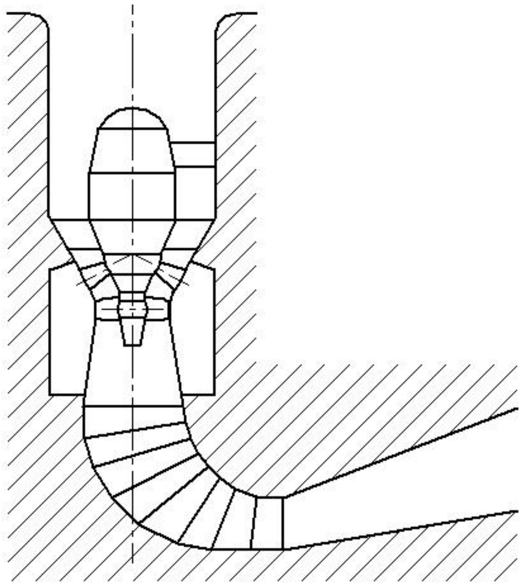
Generating output	: 150～3,500 (kW)
Discharge	: 4～25 (m^3/sec)
Effective Head	: 4～20 (m)

A diameter and a flywheel effect of the generator are restricted due to its installation in the bulb. In addition, a cooling system (such as groove or corrugation) may be required to release the heat from the generator.



Source: NEF Medium and small scale hydropower guidebook (5th revised edition)

Figure 12-13 Packeage Bulb Turbine



Source: JIS-B0119 Hydraulic turbine and reversible pump turbines-Vocabulary

Figure 12-14 Vertical Shaft Tubular Turbine

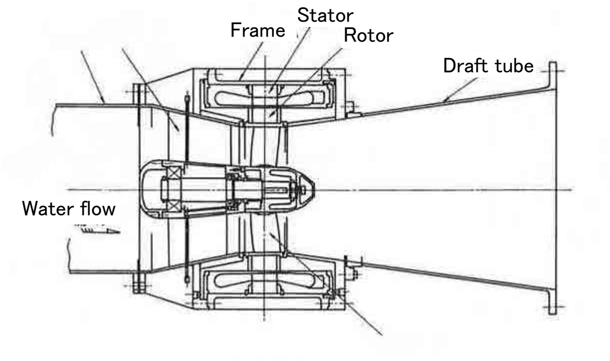
(e) Straight flow turbine

It is a water turbine which has been adopted for a long time historically in Europe when there is a relatively large river flow and a low effective head. It is the structure that a generator is installed in the outer periphery of the runner of tubular turbine. Because a generator is installed outside of the waterway, it is advantageous in easy maintenance, no restriction on the flywheel effect, and a compact plant building compared with bulb turbine and Kaplan turbine. Each set of seal devices is necessary at the outer periphery of the turbine upstream and downstream between fixed parts of the turbine and the peripheral ring where a generator rotor is set. This is disadvantageous in its technical difficulty coping with a high circumferential speed of the generator rotor, and the time-consuming maintenance works. The improvement of seal materials has recently been advanced, and an adjustable runner vane system of the straight flow turbine has been developed. An application area of the straight flow turbine spreads to 20,000kW in output and 120m^3/sec in discharge.

An integrated water turbine generator has been developed for a small scale hydropower scheme by adopting an induction generator and omitting the outer ring seal systems. An application area of the integrated water turbine generator is shown below.

Generating output	: 10～20,000 (kW)
Discharge	: 0.5～120 (m^3/sec)
Effective Head	: 3～20 (m)

The turbine of this type has fixed guide vanes and runner vanes, and it may be necessary for a few turbines to be installed to respond to the varying discharges by operating number of units. A start- up and a stop of this turbine are controlled by either inlet valve or intake gate.



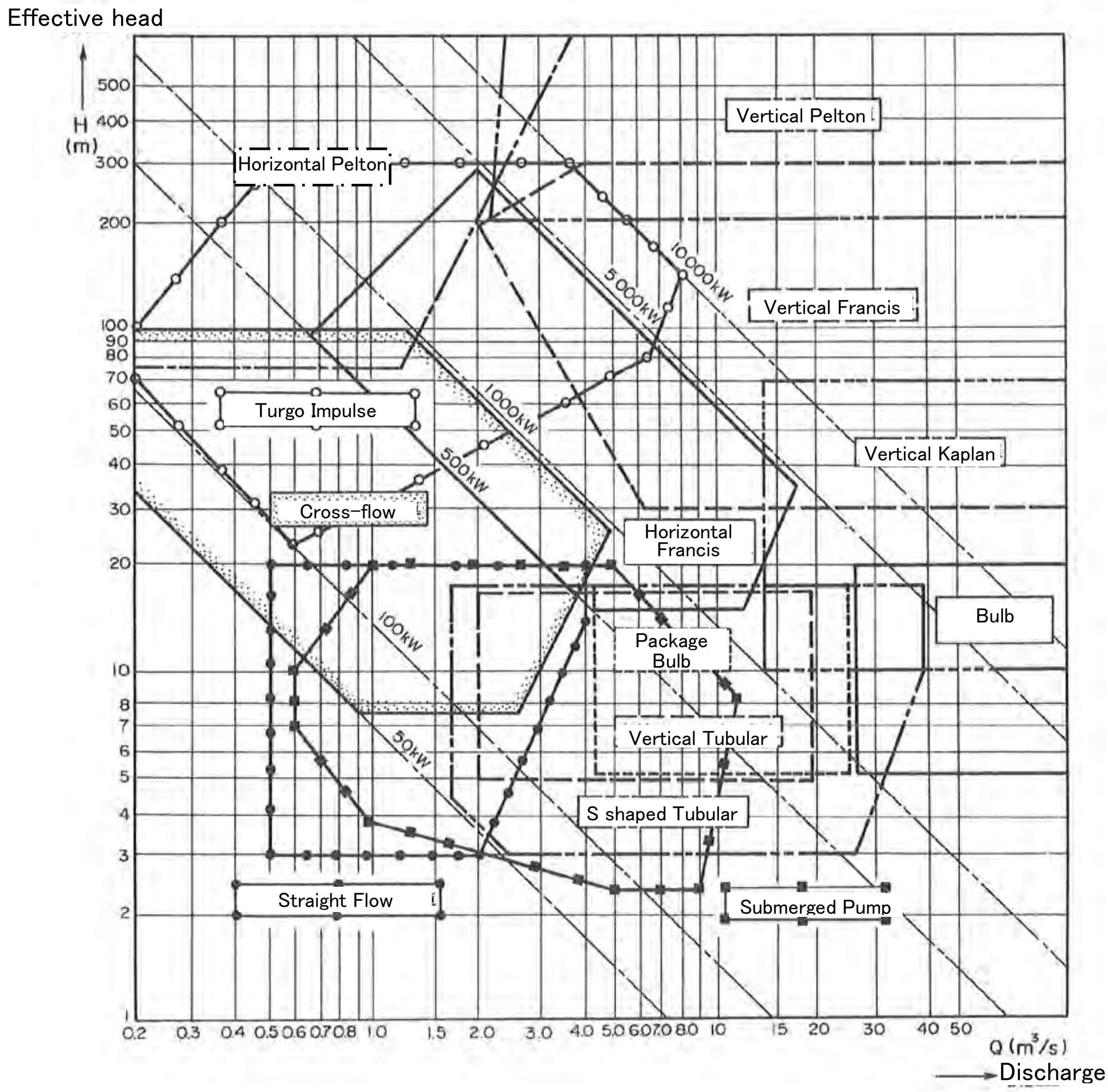
Source: NEF Medium and small scale
hydropower guidebook (5th revised edition)

Figure 12-15 Straight Flow Turbine

12.1.2 Turbine Type Selection

The turbine type is selected on the basis of the effective head and turbine discharge, while considering such factors as a river flow, operation of the reservoir and regulating pond (head fluctuation and flow fluctuation) and distributing demand. When two or more turbine types are available, the best one is selected by a comprehensive study of cost, efficiency, maintenance, etc.

Various turbines have limitations on their respective heads and applicable specific speeds. The application range is determined by their adaptability, strength, and characteristics including cavitation with respect to head and discharge variations. Shown in Figure 12-16 and Figure 12-17 below are the coverage of water turbines including the turbines of the foregoing paragraphs. Generally, the Pelton turbine is applied to a high head with a small flow and the propeller turbine to a low head with a large flow. The Francis turbine is applied to a medium to high head with a large to medium flow.



Source: NEF Medium and small scale hydropower guidebook (5th revised edition)

Figure 12-16 Coverage of Water Turbines (Less Than 10MW)

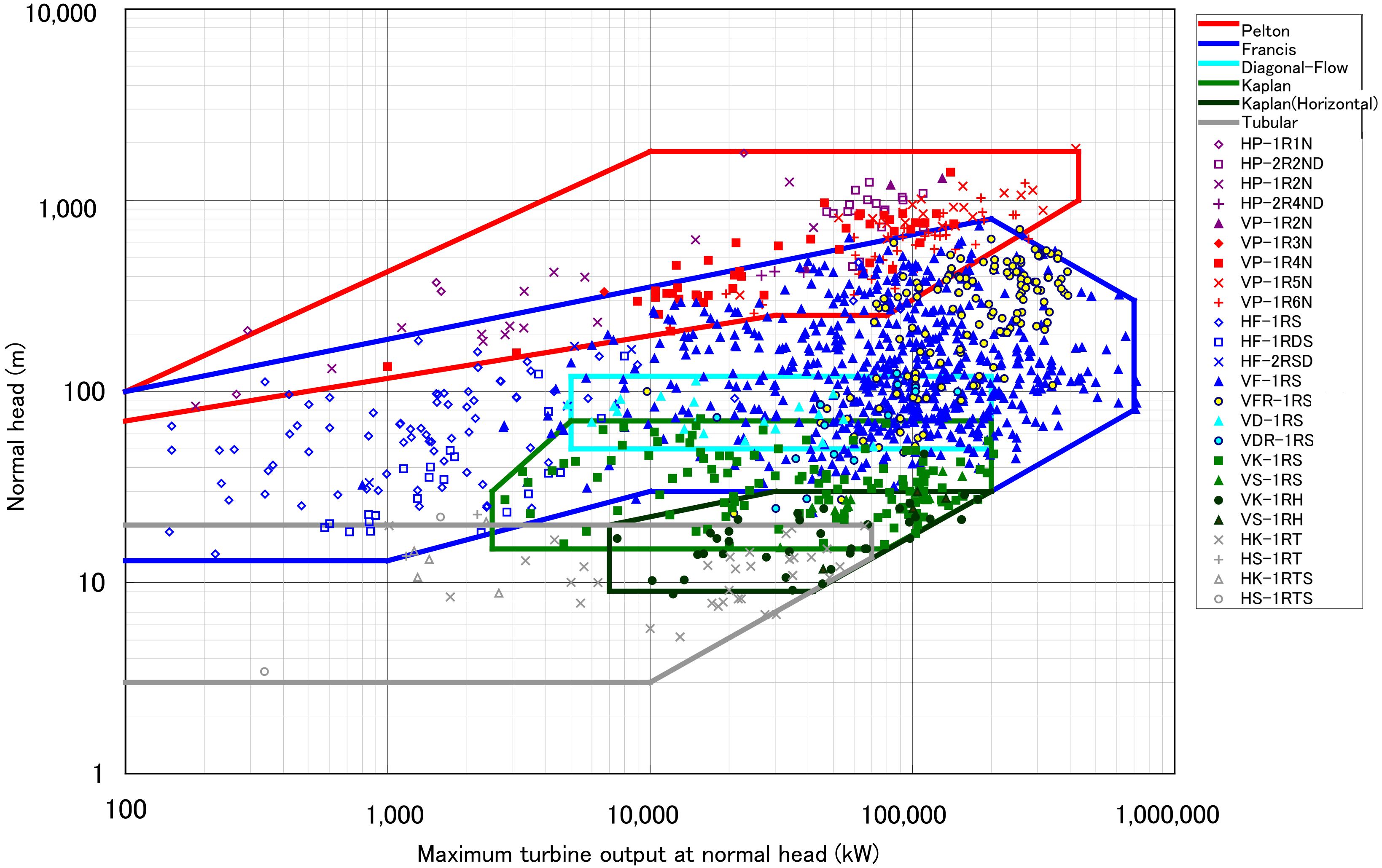


Figure 12-17 Coverage of Water Turbines (More Than 10MW)

12.1.3 Designing the Water Turbine

The turbine design in a feasibility study stage can be carried out by the following flow.

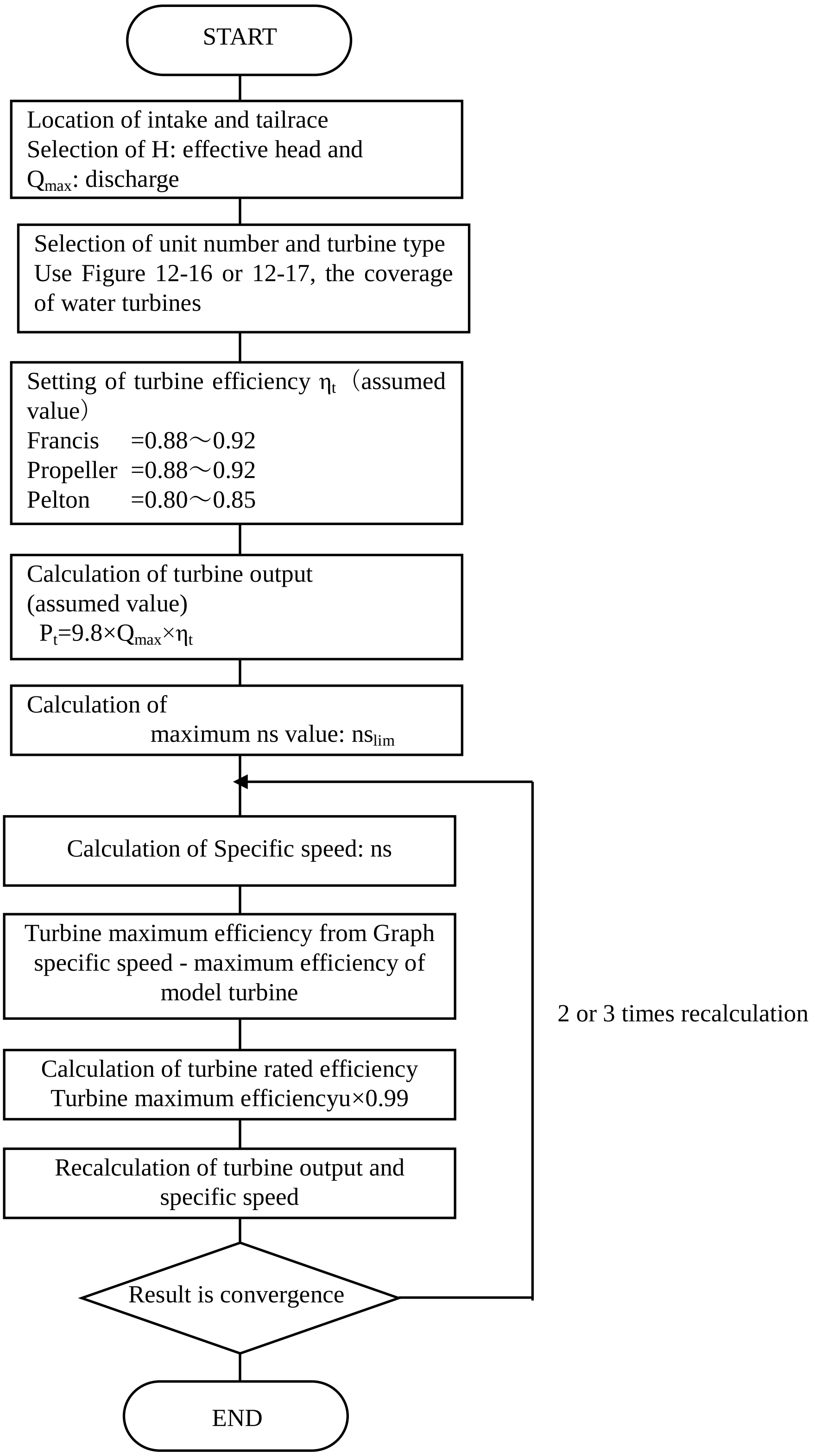


Figure 12-18 Design flow of a water turbines

(1) Set of water turbine output (assumed value)

In the first estimation, it is necessary to set the water turbine output for the calculation of specific speed, rotary speed, and efficiency. Therefore, the designer sets an assumed efficiency shown below for an initial value for each water turbine type, and calculates the water turbine output, specific speed, revolving speed. And the designer recalculates them from calculation results again as needed and improves the accuracy. A theoretical formula for water turbine output is shown below.

$$P = 9.8 \times Q_{max} \times H_e \times \eta_t$$

where,

P_t	: Maximum turbine output at the effective head H (kW)
Q_{max}	: Maximum plant discharge at effective head H (m^3/sec)
H_e	: Effective Head (m)
η_t	: Water turbine efficiency

Assumed water turbine efficiency

Pelton turbine	: 0.88～0.92
Francis turbine	: 0.88～0.92
Diagonal flow Turbine	: 0.88～0.92
Propeller turbine	: 0.80～0.85

(2) Calculation of the Specific speed of water turbine

The specific speed of a certain turbine is the rotating speed of a hypothetical turbine which is homologous to the original turbine and produces unit output (1kW) under unit head (1m). When the effective head and turbine output are kept constant, it is called the specific speed because this expresses the pitch (small or large) of the revolving speed of the turbine. The specific speed is given by the following formula.

$$ns = n \times \frac{\sqrt{P_t}}{H_e^{5/4}} \qquad\qquad n = \frac{ns \times H_e^{5/4}}{\sqrt{P_t}}$$

where,

ns	: Specific speed (m-kW)
n	: Revolving speed (min^{-1})
He	: Effective Head (m)
P_t	: Maximum turbine output at effective head H_e (kW)

Given the head and output, the specific speed is proportional to the revolving speed and the selection of a higher specific speed or higher revolving speed of the turbine makes it smaller together with a generator and a powerhouse building. There is, however, a limit to the specific speed because of cavitation occurrence and equipment strength. The maximum specific speed is given by statistics according to the following formulae.

Pelton turbine $\quad ns \leq \dfrac{4{,}300}{H_e + 200} + 14$

Francis turbine $\quad ns \leq \dfrac{23{,}000}{H_e + 30} + 40$

Diagonal flow Turbine $\quad ns \leq \dfrac{21{,}000}{H_e + 20} + 40$

Propeller turbine $\quad ns \leq \dfrac{21{,}000}{H_e + 16} + 50$

Cross Flow turbine $\quad ns \leq \dfrac{4{,}000}{H_e + 14} + 16$

The maximum output per one runner is used to calculate the above specific speed of the Francis turbine, diagonal-flow turbine and propeller turbine and the maximum output per one nozzle is used for the specific speed of the Pelton turbine. The output defined by the following equation is used for the cross-flow turbine.

$$P_t = \frac{P_r}{B_g / D_1}$$

where,

P_t : The output used for calculation of the specific speed of Cross Flow turbine (kW)

P_r : Output per one runner (kW)

B_g : Width of the guide vane flow part (m)

D_1 : Runner diameter (m)

(3) Calculation of water turbine revolving speed

The designer selects a water turbine type, sets its assumed efficiency and calculates the revolving speed from the maximum specific speed. The revolving speed of water turbine and generator is estimated by the following formula with the frequency of the distributing power system and the number of magnetic poles of the generator except when adopting a speed-up gear or a direct current generator. It is common that the revolving speed is chosen among the standard revolving speeds of a generator shown in Table 10-2, which is originally obtained in terms of the generator design.

$$n = \frac{120 \times f}{p}$$

where,

f : Power system frequency (Hz)

p : Number of Poles

The upper limit of the revolving speed of the water turbine is calculated by the formula below using the maximum specific speed of each water turbine type as shown in the foregoing paragraph. Then the designer chooses a value from the table below which is close to the calculated value.

$$n = \frac{ns_{lim} \times H_e^{5/4}}{\sqrt{P_t}}$$

where,

n	: Revolving speed (min^{-1})
P_t	: Water turbine output used for calculation of the specific speed (kW)
ns_{lim}	: Maximum specific speed (m-kW)
H_e	: Effective Head (m)

Table 12-2 The Standard Revolving Speed of a Generator（JEC-4001）

Pole	50Hz	60Hz	Pole	50Hz	60Hz	Pole	50Hz	60Hz
4	1,500	1,800	28	214	257	60	100	120
6	1,000	1,200	30	200	240	64	94	113
8	750	900	32	188	225	70	86	103
10	600	720	36	167	200	72	83	100
12	500	600	40	150	180	80	75	90
14	429	514	42	143	171	84	71	86
16	375	450	48	125	150	88	68	82
18	333	400	50	120	144	90	67	80
20	300	360	54	111	133	96	63	75
24	250	300	56	107	129	100	60	72

An estimation example is shown below..

Effective head	: H_e= 100 (m)
Maximum discharge	: Q_{max}= 5.0 (m^3/sec)
Turbine Type	: Francis
Assumed efficiency	: η_t= 0.9 is applied
Turbine output	: $P_t = 9.8 \times Q_{max} \times H_e \times \eta_t$ = 9.8×5.0×100×0.9 = 4,410 (kW)
Maximum specific speed	: $ns_{lim} = \frac{23{,}000}{H_e + 30} + 40$ = 217 (m-kW)
Revolving speed	: $n = \frac{ns_{lim} \times H_e^{5/4}}{\sqrt{P_t}} = \frac{217 \times (100)^{5/4}}{\sqrt{4{,}410}}$ = 1,033 (min^{-1})

Therefore, from the standard revolving speeds of a generator of Table 12-2, 1,200 (min^{-1}) or 900 (min^{-1}) becomes the choice candidate of the water turbine revolving speed if it is a 60Hz district, and 1,000 (min^{-1}) or 750 (min^{-1}) if it is a 50Hz district. Using the revolving speed selected, the specific speed is calculated again and the estimate of the turbine efficiency is made.

(4) Turbine efficiency estimation (Francis and Propeller turbine)

The ratio of output and input of the water turbine is called turbine efficiency, and it is given $\eta_t = P_t/(9.8 \times Q_{max} \times H_e)$.

where,

P_t	: Turbine output (kW)
Q_{max}	: Discharge (m^3/sec)
H_e	: Effective head (m)

Generally, in consideration of partial to full load operation, a turbine is designed so that it can attain the highest efficiency at around 80% of the maximum discharge. In this case, the efficiency decreases as the discharge becomes higher or lower than that point. The efficiency curve is different depending upon the turbine type and the specific speed. The relations between the specific speed and model turbine efficiency for each major turbine types are shown in Figure 12-19 to Figure 12-21.

Homologous to the actual water turbine is the model turbine, which is made in a factory after the design of actual turbine for the designer to inspect its hydraulic performance, and validity of the design. Usually a performance curve of the model turbine is organized with respect to effective head of 1m and specific runner dimension of 1m.

By the specific speed calculated from the revolving speed selected from Table 10-2 as explained in a foregoing paragraph, the maximum turbine efficiency is set. The designer reads a maximum turbine efficiency of the similar (the same specific speed) model turbine and converts it into the value of actual planning project with the following conversion formula.

$$\eta_{tmax} = \frac{2 \times (\eta_{mmax} - 0.5(1 - (P_t/H_e{}^{1.5})^{0.1})}{1 + (P_t/H_e{}^{1.5})^{0.1}}$$

where,

P_t	: Turbine output (kW)
H_e	: Effective head (m)
η_{mmax}	: Model turbine maximum efficiency
η_{tmax}	: Maximum turbine efficiency

Because it is estimated that the turbine efficiency at the rated output (the maximum output) is at around 0.99 of the maximum turbine efficiency. Then the efficiency at the rated turbine output is given in the next formula.

Turbine efficiency :$\eta_t = 0.09 \times \eta_{tmax}$

When water turbine efficiency is estimated, the water turbine output is calculated again. Therefore, the designer recalculates the specific speed and sets the efficiency again in the same way. This procedure is carried out until a value of the turbine efficiency does not change greatly. Calculations are usually done around three times.

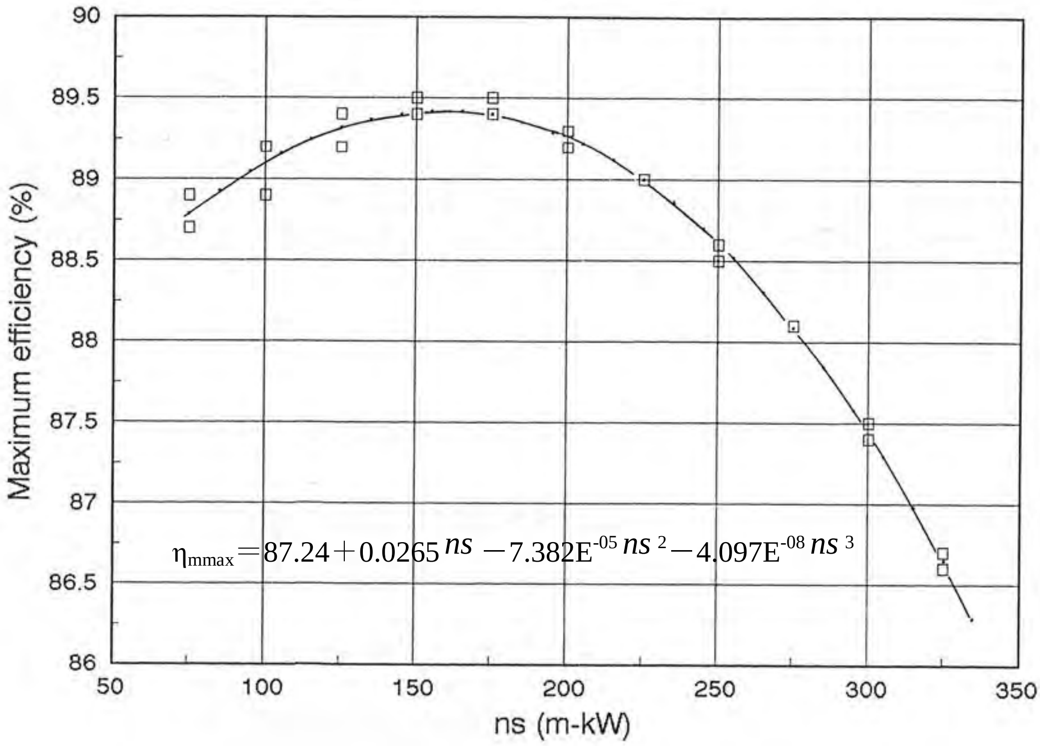


Figure 12-19　Relation of Maximum Model Turbine Efficiency and Specific Speed (Francis Turbine)

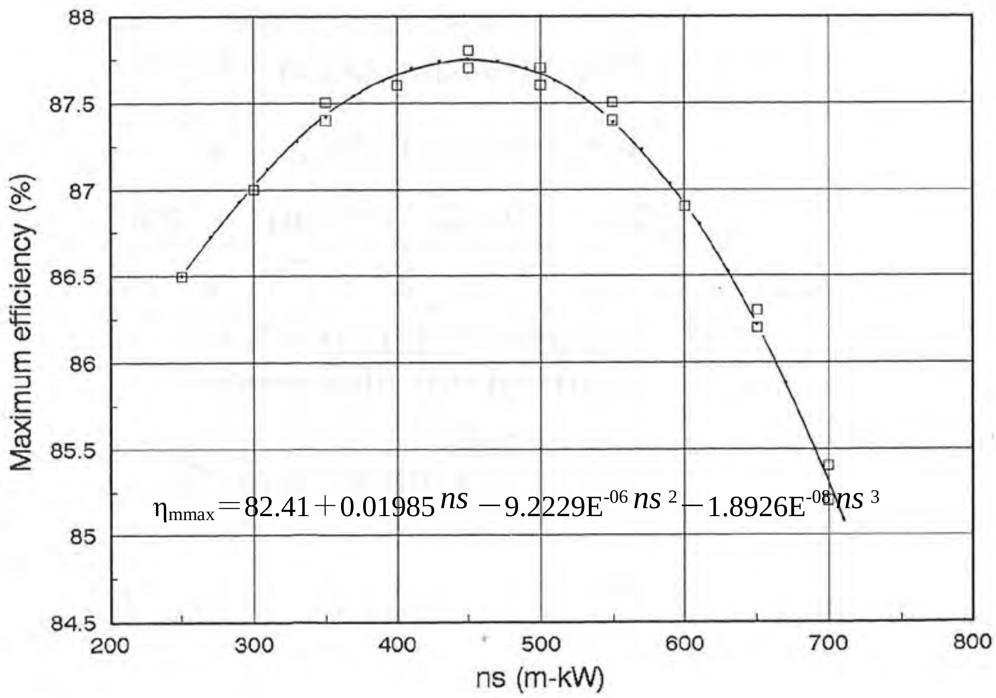


Figure 12-20　Relation of Maximum Model Turbine Efficiency and Specific Speed (Propeller Turbine)

The estimation example is shown in below.

Water turbine type	: Francis
Effective Head	: H_e=100 (m)
Maximum discharge	: Q_{max}=5.0 (m^3/sec)
Efficiency	: η_t=0.9 (assumed)
Frequency	: 50（Hz）

First calculation,

Turbine output : $P_t = 9.8 \times Q_{max} \times H_e \times \eta_t = 9.8 \times 5.0 \times 100 \times 0.9 = 4{,}410$ (kW)

Maximum specific speed : $ns_{lim} = \dfrac{23{,}000}{H_e + 30} + 40 = 217$ (m-kW)

Revolving speed : $n = \dfrac{ns_{lim} \times H_e^{5/4}}{\sqrt{P_t}} = \dfrac{217 \times (100)^{5/4}}{\sqrt{4{,}410}} = 1{,}033$ (min^{-1})

Standard revolving speed of 750 (min^{-1}) is chosen from the system frequency 50Hz district of Table 12 -2, and calculate specific speed.

Specific speed : $ns = n \times \dfrac{\sqrt{P_t}}{H_e^{5/4}} = 750 \times \dfrac{\sqrt{4{,}410}}{100^{5/4}} = 157.5$ (m-kW)

The model turbine efficiency η_{mmax} = 0.893 is given from Figure 12-19. The maximum actual turbine efficiency is converted from the model turbine efficiency.

Max. actual turbine efficiency : η_{tmax}

$$= \frac{2 \times (\eta_{mmax} - 0.5(1 - (P_t/H_e^{1.5})^{0.1})}{1 + (P_t/H_e^{1.5})^{0.1}}$$

$$= \frac{2 \times (0.893 - 0.5 \times (1 - (4{,}410/100^{1.5})^{0.1})}{1 + (4{,}410/100^{1.5})^{0.1}}$$

$= 0.901$

Rated turbine efficiency : η_t $= \eta_{mmax} \times 0.99 = 0.901 \times 0.99 = 0.891$

Second calculation,

Turbine output : $P_t = 9.8 \times Q_{max} \times H_e \times \eta_t = 9.8 \times 5.0 \times 100 \times 0.891 \approx 4{,}370$ (kW)

Specific speed : $ns = n \times \dfrac{\sqrt{P_t}}{H_e^{5/4}} = 750 \times \dfrac{\sqrt{4{,}370}}{100^{5/4}} = 156.8$ (m-kW)

The model water turbine efficiency η_{mmax} = 0.893 is given from Figure 12-19. The maximum actual turbine efficiency is converted from the model turbine efficiency.

Max. actual turbine efficiency : η_{tmax} $= \dfrac{2\times(\eta_{mmax} - 0.5(1-(P_t/H_e^{1.5})^{0.1})}{1+(P_t/H_t^{1.5})^{0.1}}$

$= \dfrac{2\times(0.893 - 0.5\times(1-(4{,}370/100^{1.5})^{0.1})}{1+(4{,}370/100^{1.5})^{0.1}}$

$= 0.901$

Rated turbine efficiency : η_t $= \eta_{mmax}\times 0.99 = 0.901\times 0.99 = 0.891$ (Convergent)

Because the calculation is convergent, the designed turbine ratings become as follows.

Rated turbine Efficiency	: 0.891
Turbine output	: 4,370 (kW)
Revolving speed	: 750 (min^{-1})

(5) Turbine efficiency estimation (Pelton turbine)

The efficiency estimation of a Pelton turbine is carried out by a similar method of a Francis and a propeller turbine. However, a conversion formula from a model turbine to an actual one is different from the above turbines and the specific speed of the Pelton turbine is calculated using the maximum output per nozzle. The conversion formula of the maximum turbine efficiency from a model turbine to an actual one of the Pelton turbine is shown below. The relation between the specific speed and the maximum turbine efficiency of the Pelton turbine is shown in Figure 12-21

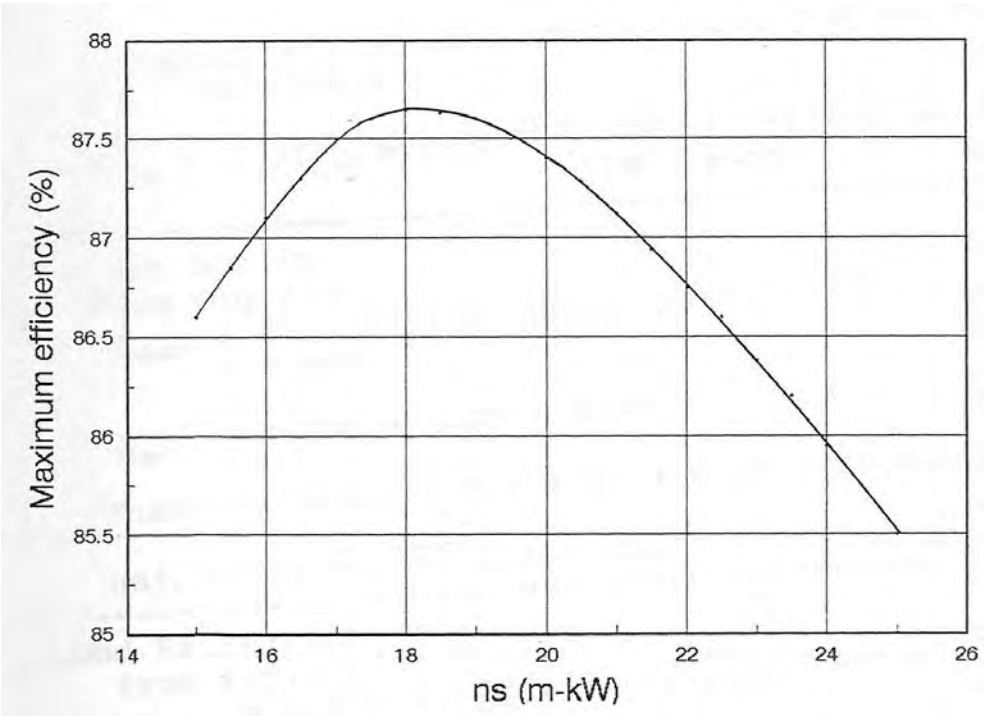


Figure 12-21 Relation of Maximum Model Turbine Efficiency and Specific Speed (Pelton Turbine)

$$\eta_{tmax} = \eta_{mmax} \times \left(\frac{P}{Noj \times 2500}\right)^{0.01375} \times \left(\frac{Noj}{4}\right)^{0.01475}$$

where,

P	: Turbine output (kW)
Noj	: Number of nozzles (jet)
η_{mmax}	: Maximum efficiency of model turbine
η_{tmax}	: Maximum turbine efficiency

The estimation example is shown below.

Turbine type	: 4jet Pelton turbine
Effective head	: H_e=300 (m)
Maximum discharge	: Q_{max}=5.0 (m^3/sec)
Efficiency	: η_t=0.85 (assumed)
Frequency	: 50 (Hz)

First calculation,

Turbine output : $P_t = 9.8 \times Q_{max} \times H_e \times \eta_t = 9.8 \times 5.0 \times 300 \times 0.85 = 12{,}495 \approx 12{,}500$kW

4jet Pelton turbine is adopted in the example, and the turbine output per nozzle Pj is calculated.

Turbine output per nozzle : $Pj = P_t/Noj = 3{,}130$kW

Maximum specific speed : $ns_{lim} = \dfrac{4{,}300}{H_e + 200} + 14 = 22.6$ (m-kW)

Revolving speed : $n = \dfrac{ns_{lim} \times H_e^{5/4}}{\sqrt{Pj}} = \dfrac{22.6 \times (300)^{5/4}}{\sqrt{3{,}130}} = 504$ (min^{-1})

Standard revolving speed 500 (min^{-1}) is chosen from Table 12-2 for the system frequency 50Hz district, and calculate specific speed.

Specific speed : $ns = n \times \dfrac{\sqrt{Pj}}{H_e^{5/4}} = 500 \times \dfrac{\sqrt{3{,}130}}{300^{5/4}} = 22.4$ (m-kW)

The model water turbine efficiency η_{mmax} = 0.866 is given from Figure 12-21. The maximum actual turbine efficiency is converted from the model turbine efficiency

Max. actual turbine efficiency : η_{tmax}

$$= \eta_{mmax} \times \left(\frac{Pj}{Noj \times 2500}\right)^{0.01375} \times \left(\frac{Noj}{4}\right)^{0.01475}$$

$$= \eta_{mmax} \times \left(\frac{Pj}{2500}\right)^{0.01375} \times \left(\frac{Noj}{4}\right)^{0.01475}$$

$$= 0.866 \times \left(\frac{3{,}130}{2500}\right)^{0.01375} \times \left(\frac{4}{4}\right)^{0.01475}$$

$$= 0.868$$

Rated turbine efficiency : η_t $= \eta_{mmax} \times 0.99 = 0.868 \times 0.99 = 0.859$

Second calculation,

Turbine output : $P_t = 9.8 \times Q_{max} \times H_e \times \eta_t = 9.8 \times 5.0 \times 300 \times 0.859 \approx 12{,}600$ (kW)

4jet Pelton turbine is adopted in the example, and the turbine output per nozzle Pj is calculated.

Turbine output per nozzle : $Pj = P_t/Noj = 3{,}200$ (kW)

Specific speed : $ns = n \times \dfrac{\sqrt{Pj}}{H_e^{5/4}} = 500 \times \dfrac{\sqrt{3{,}150}}{300^{5/4}} = 22.5$ (m-kW)

The model water turbine efficiency $\eta_{mmax} = 0.866$ is given from Figure 12-21. The maximum actual turbine efficiency is converted from the model turbine efficiency

Max. actual turbine efficiency: η_{tmax}

$$= \eta_{mmax} \times \left(\frac{Pj}{Noj \times 2{,}500}\right)^{0.01375} \times \left(\frac{Noj}{4}\right)^{0.01475}$$

$$= \eta_{mmax} \times \left(\frac{Pj}{2500}\right)^{0.01375} \times \left(\frac{Noj}{4}\right)^{0.01475}$$

$$= 0.866 \times \left(\frac{3{,}150}{2500}\right)^{0.01375} \times \left(\frac{4}{4}\right)^{0.01475}$$

$$= 0.868$$

Rated turbine efficiency η_t $= \eta_{mmax} \times 0.99 = 0.868 \times 0.99 = 0.859$ (Convergent)

Because the calculation is convergent, the designed turbine ratings become as follows.

Rated efficiency : 0.859
Turbine output : 12,600 (kW)
Revolving speed : 500 (min^{-1})

(6) Draft head

A Pelton turbine and a Targo Impulse turbine is an impulse turbine that uses the effective head at the nozzle exit entirely as a velocity head. Unlike the reaction turbine, the head from the nozzle to the tailrace level becomes ineffective. When the runner is installed near the tailrace water level, the water running outside the runner may be up with in the housing by water foaming to hit the bottom of the runner and causes the output reduction. The runner installation height from the tailrace water level varies depending on the runner shape and the specific speed, and is set generally at 2 to 3m.

When the tailrace water level rises only temporarily during a flood, the installation elevation of the Pelton turbine can be set at the normal water level. In such a case, however, compressed air may be sent inside the housing to lower the downstream water level and continue the operation throughout a flood.

As the cross-flow turbine runner receives water in the air in the same way as the Pelton turbine, the runner is installed higher than the tailrace water level. The runner installation height and an effective head vary depending on the presence or absence of a draft tube. When a draft tube is not installed and water running through the runner is discharged directly into the air, the runner installation height is determined to ensure that the foaming water does not strike against the bottom of the runner, similarly to the Pelton turbine. The head from the runner center to the tailrace water surface then becomes ineffective. If a draft tube is installed, a part of the head mentioned above is recovered.

A reaction turbine such as a Francis turbine, a diagonal flow turbine, and a Kaplan turbine is equipped with a draft tube to minimize the turbine loss and use the head between the runner center and the tailrace water level effectively, which is called a draft head. It is desirable that the turbine be installed as high as possible in view of flood protection and reduction of foundation excavation in powerhouse. However, if the draft head is raised above a certain level, negative pressure on the back of the runner vanes increases and causes cavitation as well as noise, vibration, efficiency drop, etc. Moreover it accelerates cavitation erosion of the runner itself.

A cavitation coefficient is used as an index to quantitatively express the conditions causing cavitation. The turbine draft head (Hs) is expressed by the following formula.

$$Hs = Ha - Hv - \sigma H$$

where,

Hs : Draft head (m)

Ha : Atmospheric pressure (water column m)

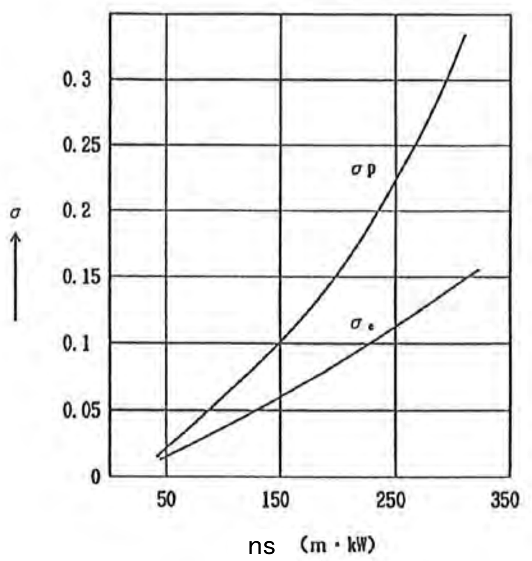
Hv : Saturated vapor pressure (water column m)

σ : Cavitation coefficient

(Obtained from the turbine specific speed (Ns in Figure 12-22 and 12-23)

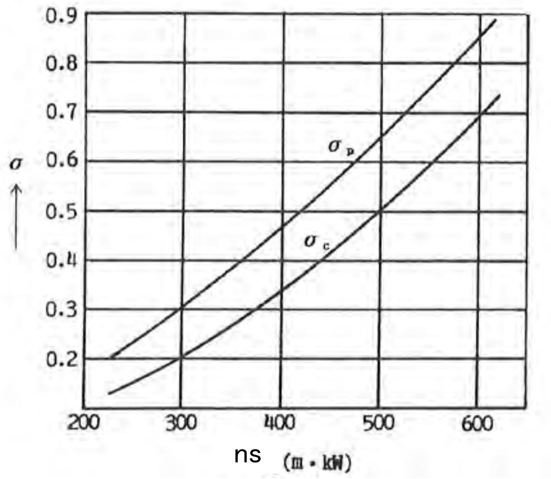
H : Effective head (m)

The cavitation coefficient is classified into a critical cavitation coefficient to indicate the point where the efficiency begins to drop during turbine operation and a plant cavitation coefficient to indicate the turbine draft head for the runner to be actually installed. The relations between σ and turbine specific speed ns of Francis and Kaplan turbines are shown in Figure 12-22 and Figure 12-23. The turbine draft head (Hs) is specified as difference of elevation between a specific position of the runner and the tailrace water surface. The positions of several types of turbines are shown in Figure 12-24. They are not necessarily in the center of the turbines and on the rather safe side against cavitation.



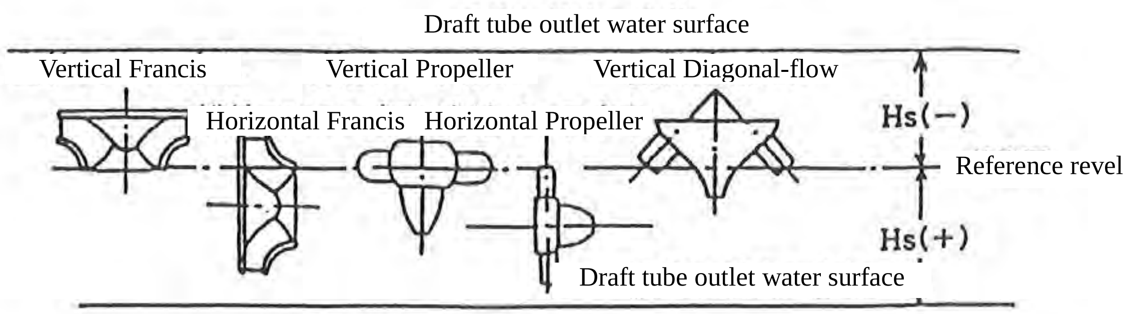
Source: IEEJ standard JEC-4001 Hydro turbine and pump turbine

Figure 12-22 Cavitation Coefficient of Francis Turbine



Source: IEEJ standard JEC-4001 Hydro turbine and pump turbine

Figure 12-23 Cavitation Coefficient of Kaplan Turbine



Source: IEEJ standard JEC-4001 Hydro turbine and pump turbine

Figure 12-24 Specific Point of Runner and Static Suction Head

12.1.4 Inlet Valve

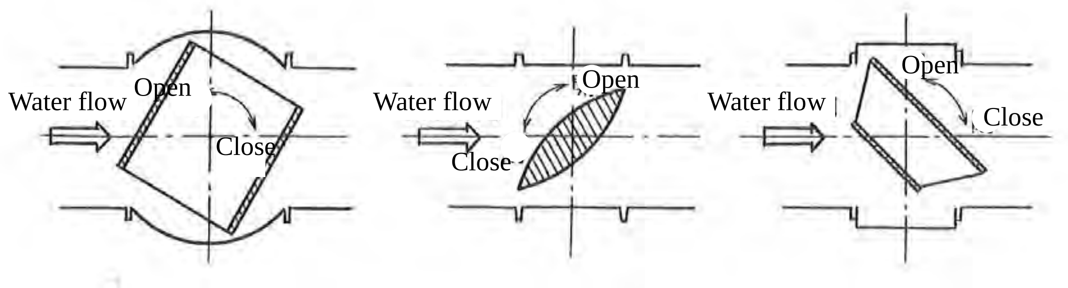
The inlet valve is located between the turbine casing and the end of the penstock. It is a water stop valve which is opened/closed when the turbine is operated/stopped. Ordinarily the inlet valve is operated when the guide vanes are completely closed and there are no flows at all. In an emergency, however, such as when the guide vanes are not closed, the inlet valve is sometimes designed to shut off the entire turbine inflow. Three types of inlet valves are presently employed, which are a spherical valve, a butterfly valve and a through valve. Figure 12-25 shows the structure of these valves.

A spherical valve has a cylindrical valve inside the spherical valve body. When the valve is fully closed, water is stopped by a ring shaped sealing surface installed on the side face of the cylinder. When the valve is fully opened, the hollow cylinder connects the penstock and the turbine casing to be one single continuous pipe.

A butterfly valve has a lens-shaped valve inside the cylinder body. When the valve is fully closed, the erect valve cuts off the passage and stops the flow. When the valve is fully opened, the lens-shaped valve remains flat in the flow passage to connect the penstock and the turbine casing.

A through valve contains within a cylindrical body a water stop disc and a reinforcing plate connected by several ribs. The flow is cut off by the disc. When the valve is fully opened, the valve itself remains in the passage. The disk, plate, and ribs stand against the water flow, but water can pass through the valve.

Generally, spherical valves are used when the head is over 250m. Butterfly valves are used when the head is less than 200m and through valves less than 350m. In the case of the low head less than around 70m, it is possible to omit an inlet valve when the upstream waterway is short and there are not two or more units or other water utilization. In that case, however, the intake gate is designed to function as emergency flow interruption.



Spherical valve Butterfly valve Biplane valve

Figure 12-25 Outline Structure of Inlet Valves

12.1.5 Turbine Auxiliary Equipment

(1) Speed governor

The speed governor automatically adjusts the guide vanes opening according to the change of rotating speed to control the turbine speed and output. The speed governor adjusts the rotating speed to synchronize at the time of turbine start up, and then controls the output connected to the power system during synchronous operation. During single operation with a limited load, the output is adjusted according to the variations of the load to control the frequency. When operation is disconnected due to a fault or breakdown of the distribution line, the turbine is immediately shut down by closing the guide vanes to prevent an abnormal speed rising of the turbine and generator.

Water level regulators which regulate the turbine discharge depending on the waterway inflow are installed in run-of-river hydropower schemes. In this case a speed governor responds to changes in the head tank water level and adjusts the guide vanes opening. In recent years, a digital type of the speed governor has become the mainstream. To give an example at a small scale hydropower station, PLC (Process Logic Controller) is applied as a package control panel integrating not only the start stop sequence control but also the excitation control and speed control of the generator and is controlled by software.

(2) Oil pressure supply system

This system provides the oil pressure needed to operate the turbine's guide vanes, runner blades, deflectors, needles, brakes and inlet valve servo motors. The oil pressure supply system consists of an oil pressure tank, a sump tank, an oil pressure pump, unloaders, etc. There are two types of oil pressure supply system, a one-on-one type with one oil pressure supply system for one turbine, and a centralized type with 1 or 2 oil pressure supply systems for two or more turbines.

In recent years, there have been power stations where electrical servo motors are adopted instead of oil pressure servo motors in order to exclude the risk that leakage oil from a hydropower station affects the neighboring environment. It depends on a water turbine type and is applicable to the turbine with turbine output less than about 30MW. In this case the designer must remember to secure, in the design of direct current power equipment, drive energy of the electrical servo motor to enable the control of turbine when power fails. There is an example using a counter weight or water pressure as a backup at the time of a blackout.

(3) Lubricating oil feeding system

1) Lubricating oil feeding system for bearings

This system supplies lubricating oil to oil re-circulating type turbine bearings. The system consists of an oil pump, an oil tank, cooling device, an oil flow relay, etc. This system is often adapted to bulb turbine. For a small scale hydropower plant, a self-storing system is applied to most turbine bearings and a lubricating oil feeding system is not used. In the case of propeller turbine etc., turbine bearings exist inside the flow channel and a water circulating system is popular nowadays adopting resin bearings.

2) Grease feeding system

This system automatically feeds a certain amount of grease at a certain intervals to the gate operating mechanisms such as guide vanes, needles, etc., and to the sliding part of the inlet valve bearings, etc. There are two types, a parallel type and a series type (progressive type). In the parallel type, the distribution valves work together to feed the grease. In the series type, the distributing valves work and feed in succession. Oil-less bearings with graphite or fluororesin fixed lubricant embedded in the metal base have recently been developed and put into service to prevent a grease spill to the river.

(4) Cooling water supply system

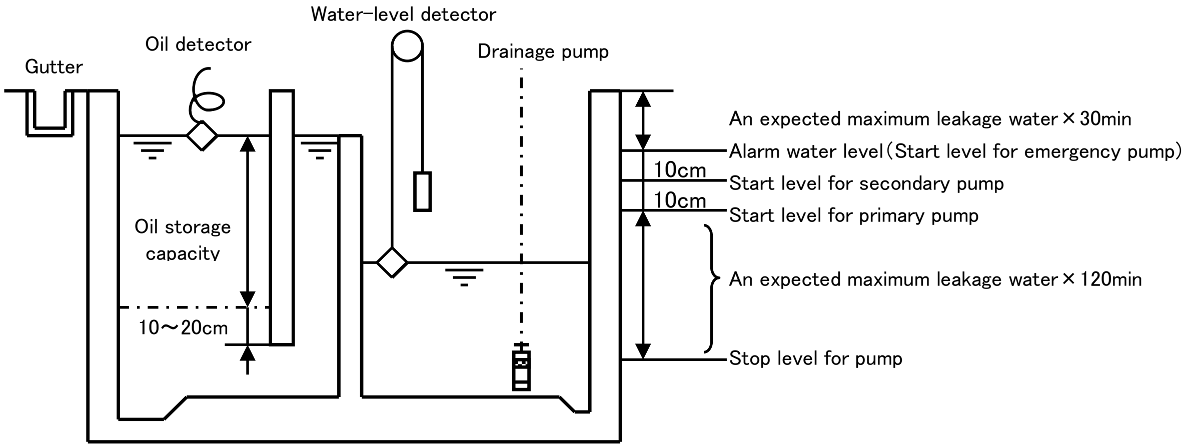
This system supplies cooling water to the turbine and generator bearings, and generator itself as well as firewater. Generally, in power plants where the head is 30 to150m, the water is supplied from a penstock by an automatic valve via a pressure reducer. In power plants where the head is less than 30m and where there is insufficient feed water pressure, and in power plants where the head is over 150m and where it is uneconomical to use high pressure water, the water is fed from the tailrace by a feed water pump. When feeding water from the penstock, a feed water tank is generally installed at an appropriate height to reduce the pressure, and settle the sediment. The tank capacity is of a size to supply water for about five minutes at a water failure. The feeding water passes through an automatic strainer and then is fed to each cooler. Sand separators and rapid filters are used where purified water is needed as main bearings seal water, etc.

For a turbine and a generator for a small scale hydropower generation around a few hundred kW, the cooling water supply system is omitted by adopting air-cooling for generator with air duct, plastic packing for main shaft sealing, air cooling with the heat radiation fin, heat piping of the main bearings, etc. It aims at simplification of the equipment and future maintenance works.

(5) Water drainage system

Coming into a power plant are leakage from building walls and foundations, leakage from turbine seals, water remaining in the penstock, casing, and piping during works, and general service water. All water is collected in a drainage pit provided at the lowest level and the drainage pump automatically operates depending on the level of collected water to discharge it to outside the power plant. Jet pumps and small turbine pumps are used as countermeasures against a station blackout. However, jet pumps cannot be used in power plants where the effective head is less than 40m. A capacity of the drainage pump is estimated from the volume of the building and quantity of leakage water. A capacity of the drainage pit is estimated from how long it takes for a member of maintenance to arrive at the power plant, and carryout effective measures when power fails, etc.

Even if lubricating oil or turbine oil leaks from equipment or piping in a power station, an oily water separator pit is installed in the drainage pit not to directly release it in the river. When it is a Pelton turbine and the installed height of water turbine is higher than river flow surface, leakage water can naturally flow down from the power station, but it is necessary even in this case to install a drainage pit and an oily water separator pit so that leakage oil does not flow out into the river outside. A design example of a drainage pit and an oily water separator pit is shown in Figure 12-26.



Source: Denkikyodoukennkyu Vol42-2 Design guide for water turbine auxiliary equipment

Figure 12-26 Example of a Drainage Pit

(6) Compressed air supply system

In conventional hydropower plant, compressed air is used to refill oil pressure tanks and as a generator brake control. In late years, at the small scale hydropower station, there are many designs that do not use compressed air by the adoption of the electric servo motors for needle vanes and guide vanes, the adoption of the electromagnetic brake for generator, and adoption of the electromagnetic and spring operation mechanism for the switch gears.

12.2 Generator

12.2.1 Generator Types

The water turbine generators mainly used are salient-pole rotating-field type 3-phase alternators. Induction generators are sometimes used in small scale hydropower plants in consideration of cost reduction. Recently, new type generators which combined an inverter (a power conditioner) with a permanent magnet alternator or a direct current alternator have been developed. It is necessary to choose the most suitable generator type in consideration of the scale of the power station, relations with the demand, and its construction cost and future maintenance cost. Conventional synchronous generators may be classified as follows depending on the shaft axis and cooling system.

(1) Classification by shaft axis

Turbine generators may be classified into two types by shaft orientation, a horizontal shaft type and a vertical one. A horizontal shaft type is generally applied to high speed generators and a vertical type is suited to low speed generators. Shaft axis is selected considering the turbine type and speed, topography, an amount of excavation required, building design, and maintenance, etc.

(2) Classification by generator cooling system

Generator Cooling systems are classified, depending on the combination of the following three types.

- Used coolant (air, water, hydrogen).
- Coolant passage and heat dissipation method (free circulation type, inlet and outlet pipe ventilated type, heat exchanger type)
- Coolant feed method (self-cooled, unaided, aided)

In generators for water turbines, generally, air is a primary coolant to directly cool a stator and a rotor, with water as a secondary coolant. These are roughly categorized into the following types. There is a method to directory cool by water the inside of a rotor and a stator in a large-capacity generator as an extremely rare example.

- Open type (free circulation type): Used for small capacity units. In general, ordinary induction motors are classified in this.

- Full enclosed, air duct ventilation type (pipe cooling type): Used for small to medium-capacity units. Most are outlet pipe cooling types. Generally it used up to 20MW.
- Full enclosed, air duct circulation type (water cooling heat exchanger type): Circulation types need an air cooler inside the duct and a cooling water system. Because it provides a high cooling capability, it is used for mid-and large-capacity generators.

(3) Classification by bearing arrangement

1) Horizontal shaft generator

For a horizontal shaft generator, it is classified into a bracket type and a pedestal type by the placement of the bearings. The bracket type has the structure that integrates a bearing with the main body of a generator as shown in Figure 12-27, and the pedestal type puts away guide and thrust bearings in each case placed in the front and behind the main body of a generator as shown in Figure 12-28. There are a lot of adoption examples of the bracket type for small capacity generators, and its installation and adjustment works are easy.

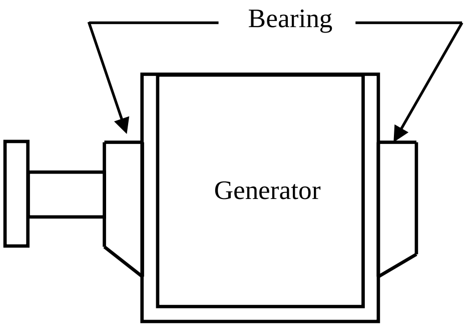


Figure 12-27 Bracket Type

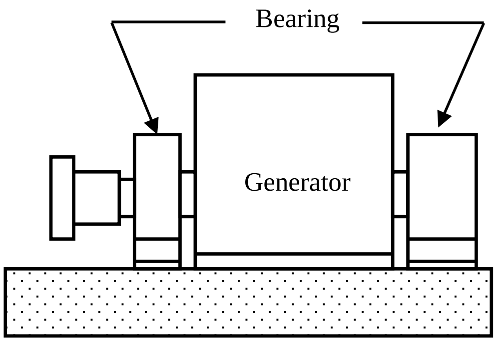


Figure 12-28 Pedestal Type

2) Vertical shaft generator

Vertical generators are classified by their bearings arrangement. A suspended type has the thrust bearings above the rotor and is provided with two guide bearings, one above the rotor and the other below it. An umbrella type has the thrust bearings below the rotor and one guide bearing also below the rotor. A semi-umbrella type, which is also called as a modified umbrella type, has an additional guide bearing above the rotor. For very large capacity generators or for generators to be applied for a low head and large discharge hydropower generation scheme, the thrust bearings may be supported from the turbine head cover to achieve more compact arrangement. Figure 12-29 shows the bearing type arrangement.

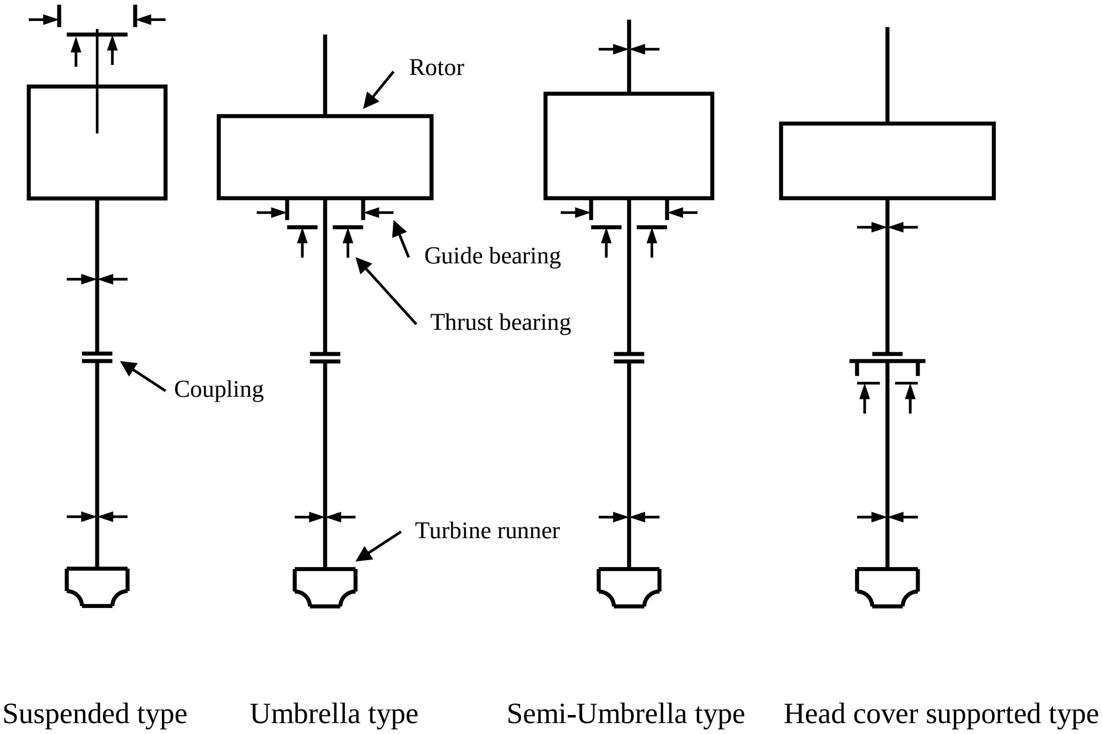


Figure 12-29   Bearing Arrangement Type of Vertical Shaft Generator

12.2.2 Designing the Generator

The design works for a generator is carried out as follows.

(1) Capacity and power factor

It is common that the rated capacity of a generator assumes water turbine maximum output, and generator output at a rating power factor, and the rated capacity is calculated using the generator loss curve shown in Figure 12-31 by the following flow.

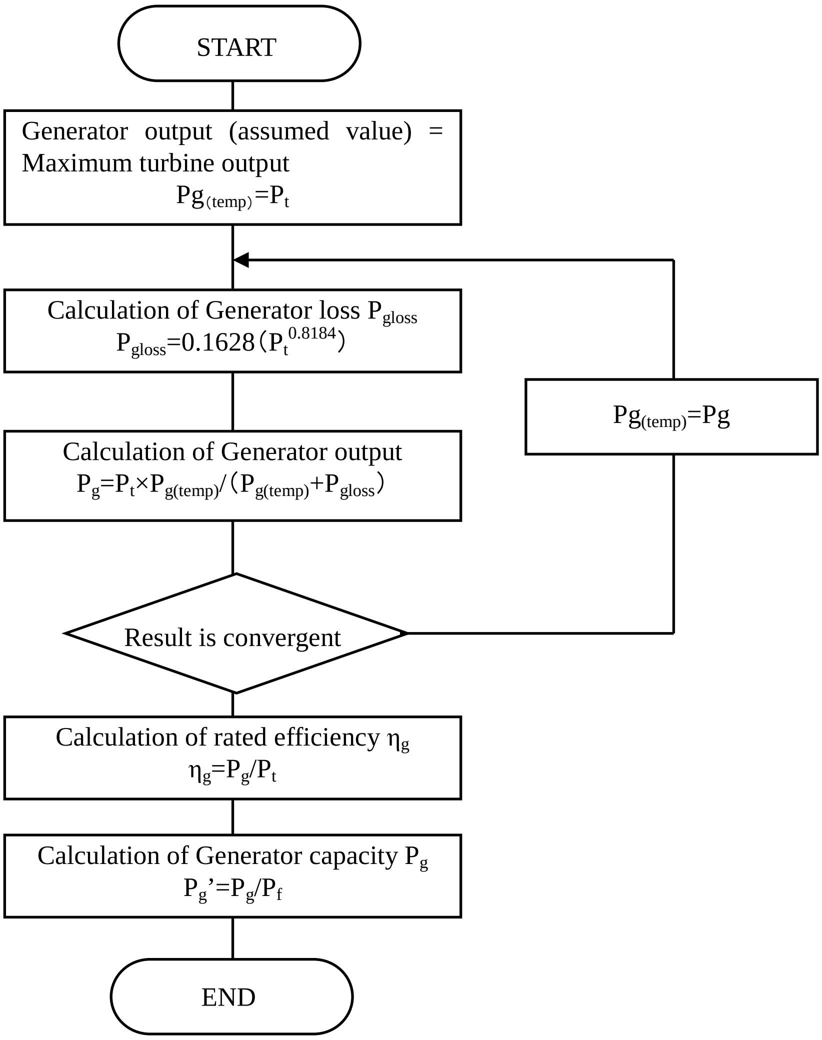


Figure 12-30 Design flow of a generator

where,

P_t	: Turbine maximum output (kW)
$P_{g(temp)}$	: Generator output (assumed value) (kW)
P_{gloss}	: Generator loss (kW)
P_g	: Generator rated output (kW)
η_g	: Generator rated efficiency
P_g'	: Generator rated capacity (kVA)
P_f	: Generator rated power factor

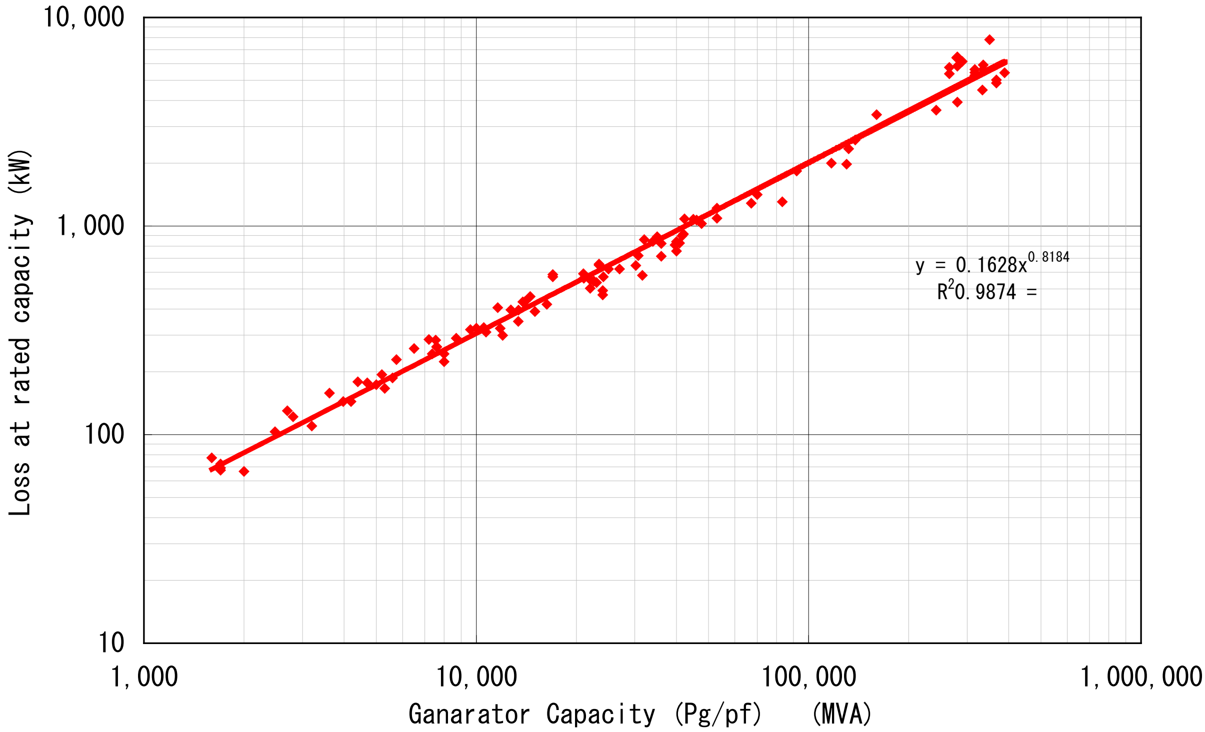


Figure 12-31 Generator Loss at the Rated Capacity (P_{gloss})

A calculation example is shown bellow.

First calculation

If turbine rated output P_t= 2,410kW, generator output is assumed to be $P_{g\,(temp)}$= 2,410 (kW)

Generator loss P_{gloss}= 0.1628 ($P_{g\,(temp)}{}^{0.8184}$) = 95 (kW)

Generator output P_g= P_t×$P_{g\,(temp)}$/($P_{g\,(temp)}$+P_{gloss}) = 2,410×2,410/(2,410＋95)= 2,318 (kW)

Second calculation

Generator output (assumed value) $P_{g\,(temp)}$= 2,318 (kW)

Generator loss P_{gloss}= 0.1628 ($P_{gkW(temp)}{}^{0.8184}$) = 92 (kW)

Generator output P_g= P_t×$P_{g\,(temp)}$/($P_{g\,(temp)}$+P_{gloss}) = 2,410×2,318/(2,318＋92)

= 2,318 (kW) (convergent P_g= $P_{g\,(temp)}$)

From estimated generator output P_g= 2,318 (kW)

Generator efficiency η_g= P_g/P_t= 2,318/2,410= 96.2%

Therefore, if the designer adopts P_f= 0.95 (95%) of rating power factors.

Generator rated output is, P_g’= P_g/P_f= 2,318/0.95= 2,440 (kVA)

Generator efficiency is basically determined by the generator output and power factor. The generator power factor is determined in consideration of the characteristics of the load and power system. A rated power factor of approximately 98% to 85% is usually adopted. A rated power factor as near to 100% as possible can reduce generator capacity if the generator is not connected to an independent power system to be operated with an independent load. As a result, the cost reduction of the generator is achieved. If there are not requirements of voltage control of the power system etc., a power factor of around 98% may be selected.

(2) Frequency, rotating speed and poles

Power transmission system frequencies are usually 50 Hz and 60 Hz. The frequency used depends on the area. As is described in a clause of calculation of water turbine revolving speed, alternator revolving speed is also selected in accordance with the frequency and is determined by the following formula, which is not applied to a direct current generator, an induction generator or a combination system of an induction generator and an inverter.

$$n = \frac{120 \times f}{p}$$

where,

n : Revolving speed (min^{-1})

f : Power system frequency (Hz)

p : Number of the poles

Revolving speed is determined by the maximum specific speed which depends on the turbine type. Generators become smaller as the revolving speed increases, which serves to reduce the manufacturing cost. However, the draft head of the turbine becomes higher and it requires more excavation, which in turn serves to reduce the economic merits. The revolving speed is, therefore, determined considering the overall plant cost. Table 12-3 shows the standard revolving speed of turbine generator.

Table 12-3 The Standard Revolving Speed of a Generator（JEC-4001）

Pole	50Hz	60Hz	Pole	50Hz	60Hz	Pole	50Hz	60Hz
4	1,500	1,800	28	214	257	60	100	120
6	1,000	1,200	30	200	240	64	94	113
8	750	900	32	188	225	70	86	103
10	600	720	36	167	200	72	83	100
12	500	600	40	150	180	80	75	90
14	429	514	42	143	171	84	71	86
16	375	450	48	125	150	88	68	82
18	333	400	50	120	144	90	67	80
20	300	360	54	111	133	96	63	75
24	250	300	56	107	129	100	60	72

(3) Voltage

The higher the voltage is the thicker and heavier the generator coil insulation becomes at a lowered occupation ratio of the conductor. Low voltage is more advantageous in this aspect. However, selecting low voltage has a disadvantage of a large current rating which requires a larger capacity of the cables and connecting conductors as well as breaking devices. It may reduce the overall economy when considering the main bus leading to the transformer, the circuit breakers and other switching equipment.

Considering these aspects, the rated voltage is to be selected. Generally, however, the following rated voltage is used with generator capacity as shown in Figure 12-32.

Generator capacity	Less than 3MVA	: 400 (V)
	3 - 10MVA	: 6.6 (kV)
	10 - 50MVA	: 11 (kV)
	50 - 100MVA	: 13.2 (kV)
	100 - 300MVA	: 16.5 (kV)
	Over 300MVA	: 18 (kV)

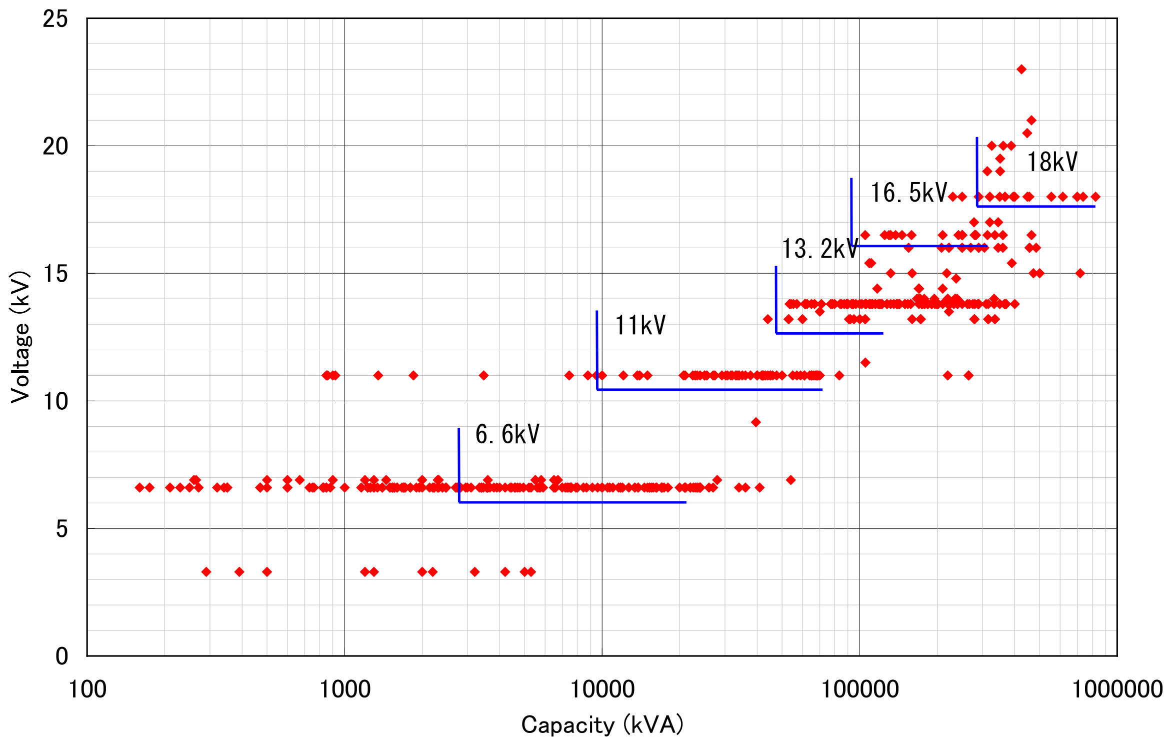


Figure 12-32 Relation between Generator Capacity and Voltage

(4) Current

The generator current is important in determining the specification of the generator itself, and the circuits and switch gears connected to the transformer. The generator rated current is calculated by the following formula.

$$Ig = \frac{Pg'}{\sqrt{3}E}$$

where,

Ig	: Generator current (A)
Pg'	: Generator rated capacity (kVA)
E	: Generator rated voltage (kV)

(5) Excitation system

The excitation circuit supplies a field current for the rotor of generator, and adjusts its output and voltage, which consists of a thyristor, an exciter power transformer, a slip ring, and an automatic voltage regulator. In recent years, for a small scale hydropower station, the integrated control panels have been operated by software which incorporated the excitation control of the generator in PLC (Process Logic Controller).

1) Excitation method

The following methods are now mainly used.

- DC excitation system
  It is a method for supplying a field current by installing a direct current dynamo. Nowadays there becomes rare adoption, and the existing DC excitation systems are often replaced with other methods because a direct current alternator is expensive and it needs the maintenance of a rectifier.
- Thyristor excitation system
  The output of an exciter transformer or an AC generator is converted to DC by a thyristor rectifier, and a field current is then supplied being regulated by thyristor phase control. This method is now used in most cases because of easier maintenance without a rectifier and quicker control speed.
- Brushless excitation system
  The output current of a revolving armature type AC generator directly connected to the rotor of the main shaft is converted to DC by a rectifier attached to the same revolving shaft, and a field current is directly supplied without using a slip ring. Brush maintenance is not needed in this method which is applied to a small scale hydropower plant.

2) Exciter capacity and voltage

Exciter capacity is determined as follows: the exciting power required to operate the generator at the rated output/rated power factor, plus a 10% margin. In most instances, the exciter voltages are 110V, 220V, and 440V.

3) Automatic voltage regulator (AVR)

An automatic voltage regulator (AVR) regulates the field current automatically to keep the generator voltage constant. To set the generator voltage, firstly constant voltage power is regulated by a voltage-adjustment resistor. This being the reference voltage, secondly it is compared with the generator terminal voltage, and the field current is regulated according to the deviation signal.

4) Automatic power factor regulator (APFR)

When a comparatively small capacity generator is connected in synchronism to a large capacity power system, the reactive power required to stabilize the generator voltage may cause the generator over-current. In this case, it is better to operate the generator, with a fixed power factor, at the voltage corresponding with the varying voltage of the power system. To do this, an automatic power factor regulator (APFR) is used. It is general for a small scale hydropower station to carry out the constant power factor operation with this APFR except independent operation of the generator.

12.3 Transformer

Hydropower plant transformers are classified into three types, a main transformer used to step up generator voltage to line voltage, a station service transformer to lower the generator voltage to house voltage, and a station low-voltage transformer to lower the house voltage to equipment voltage.

12.3.1 Main Transformer

Outdoor three-phase transformers are normally used in hydro-electric power plants. Regarding the cooling method, a self-cooling type is used in small capacity units. An air-cooling type, a feed oil air cooling type, and a feed oil water cooling type are used as the capacity increases. When the transformer is to be installed indoors, in many cases a feed oil water cooling type is used considering the limited installation space and room ventilation. When installing it outdoors, a feed oil air cooling type is normally used due to the less construction cost and easier maintenance than the water cooling system.

To maintain the transformer performance acquired at shop assembly, on-site transformer reassembly should be avoided whenever possible and on-shop assembled transformers should be transported to the site. This, however, may not be possible depending on the power plant location, and the size and weight of the transformers. For this reason, a “special three-phase transformer” and a “special three-phase six division transformer” are often used. A “special three phase transformer”, for example, is a transformer each phase of which is independently built with its own magnetic circuit and is structurally a single-phase transformer. This is transported as a single-phase unit and assembled on the site as a three-phase transformer and installed. A “special three-phase six division transformer” has two sets of coils and an independent magnetic circuit for each phase.

The rated capacity of the main transformer is set at the rated output (kVA) of the generator. Primary voltage is normally set at about 5% lower than the rated voltage of the generator. However, as a system becomes ultra-high voltage, the power factor comes near to 1. In this case, primary voltage is set at the rated generator voltage.

In addition, in a small scale hydropower station, if transmission line voltage and the generator voltage are chosen at the same voltage, an insulation transformer of the same voltage is installed as a main transformer to protect the generator from an external attack such as a thunder surge from a transmission line and to reduce the short circuit capacity in the power plant.

12.3.2 Station service transformer and house transformer

Transformer capacity is determined after a full study of the required power for plant equipment such as a water supply pump, a water drainage pump, lighting for the plant and a crane for the installation and maintenance works. Generally, a dry transformer is used for a small capacity unit installed indoors to avoid the danger of fire. It is installed in the same place parallel to the metal enclosed switchgears in consideration of the safety. The impedance of a station service transformer and a main transformer influences the short circuit current of the station service circuit. Therefore, at the time of the purchase of these transformers, it is necessary to study so that both impedance and short circuit electric current may become appropriate values.

12.4 Main Circuit Connection and Electrical Equipments

12.4.1 Main Circuit Connection

The following are considered when selecting the main circuit connection system: the number and capacity of generators, the number of transmission lines and the connection method, restrictions on power plant space, a station power receiving method and the existence or non-existence of distribution lines, the construction cost and transportation conditions of the transformers and switchgears, the range of power failure caused by in-station accidents, and the safety and ease of repair and maintenance. These aspects are considered in perspective from the viewpoint of reliability of the power plant as well as economy and technology. Typical main circuit connections are shown below.

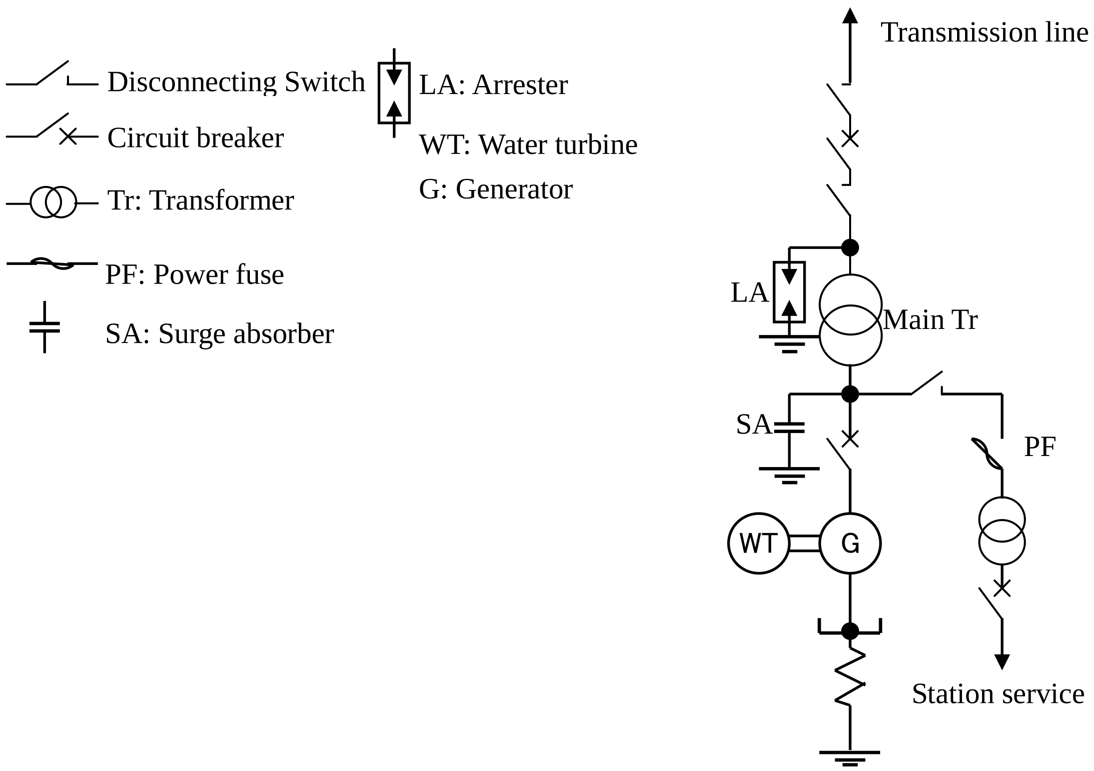


Figure 12-33 1 Unit of Turbine Generator, Single Transmission Line

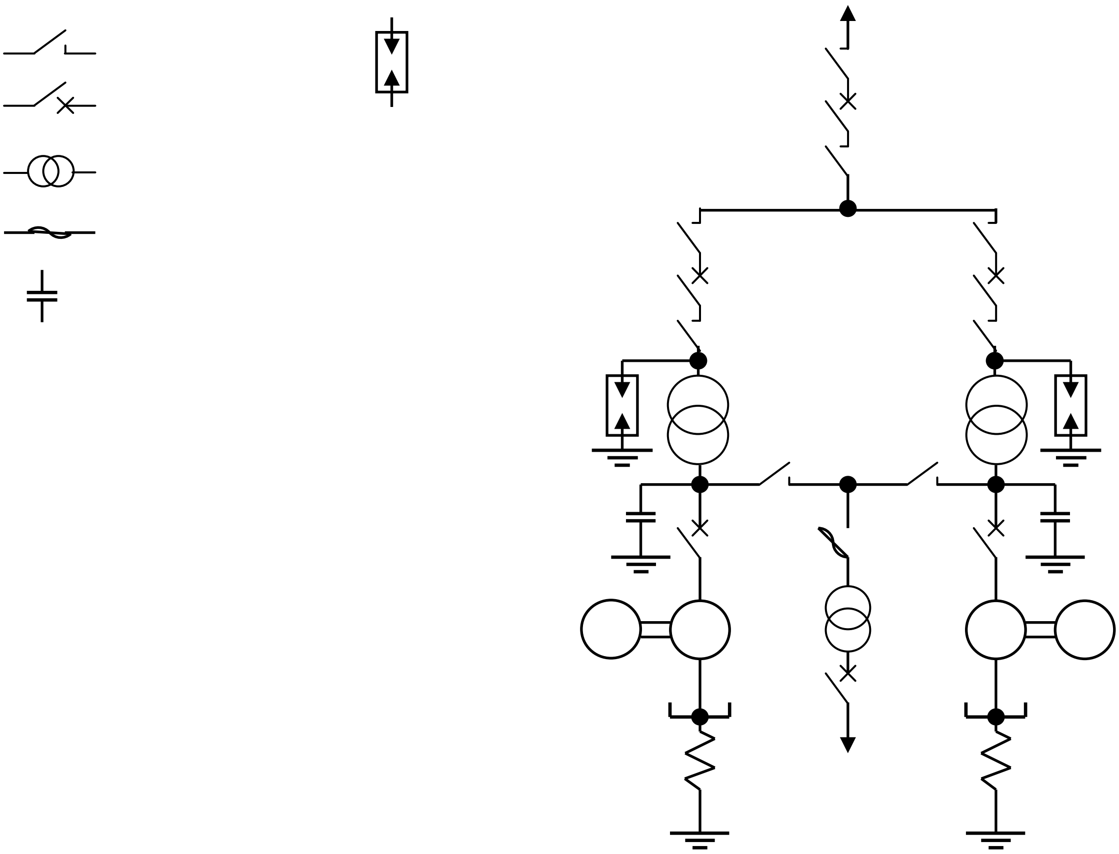


Figure 12-34 2 Unit of Turbine Generator, Single Transmission Line

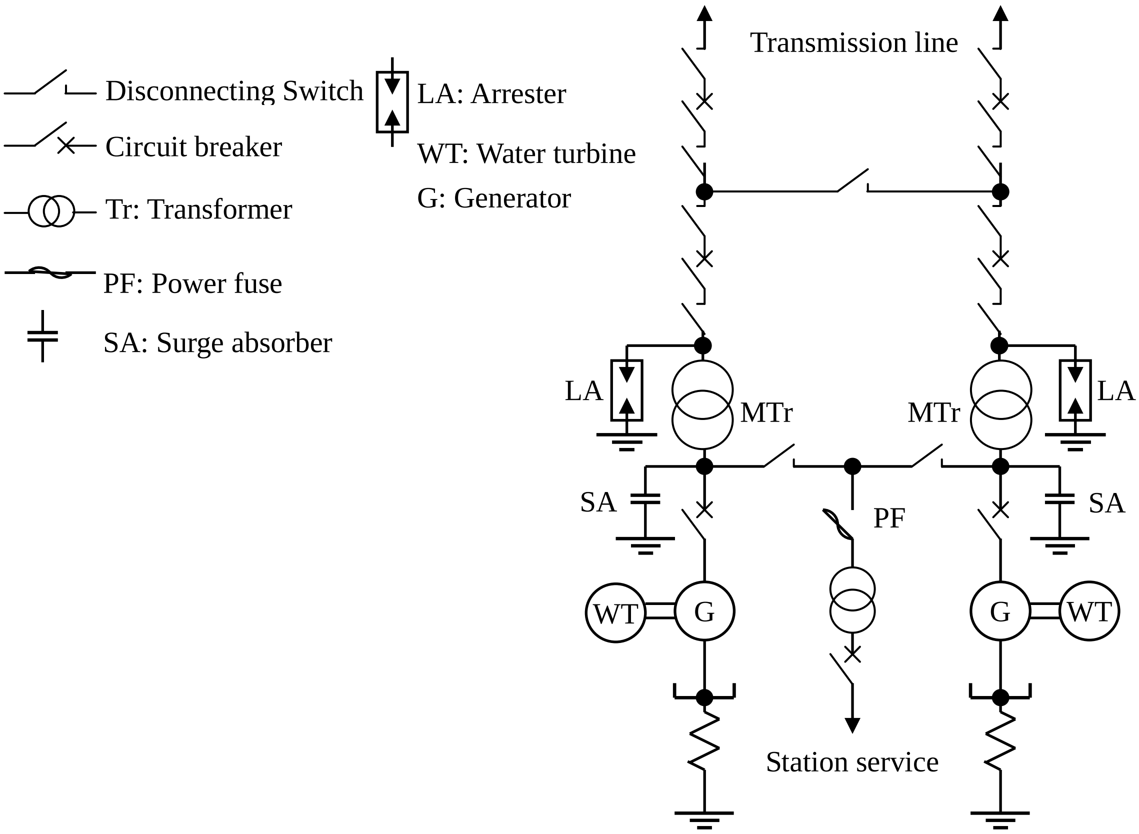


Figure 12-35 2 Unit of Turbine Generator, 2 Circuits of Transmission Line

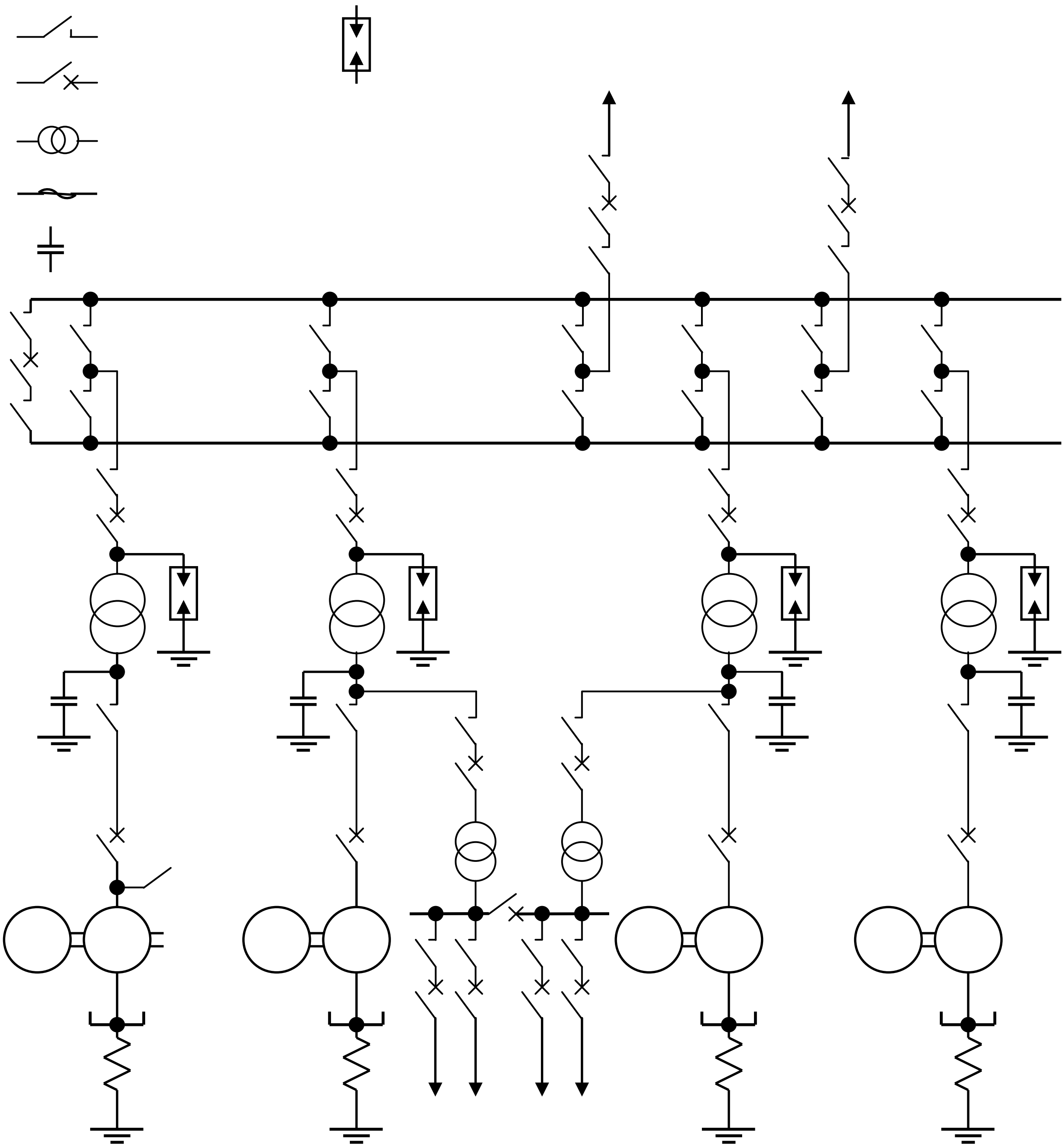


Figure 12-36   4 Unit of Turbine Generator, 2 Circuits of Transmission Line

12.4.2   Circuit Breaker

Circuit breakers switch the load currents and also interrupt short circuit fault and ground fault currents. The types are shown in Table 10-4. Circuit breakers are operated by air, oil or an electric motor and a spring. At small scale hydropower stations, there are many designs that do not use compressed air, and there is much adoption of oil pressure operation or electric motor spring operation. The circuit breakers used depend on the rated voltage as shown in Table 10-5, which is referred to when selecting the circuit breakers. The following should also be considered when making a selection.

(1) Rated voltage

The maximum voltage of the circuit used should not exceed the rated voltage of circuit breaker. In Japan, normally used are circuit breakers with a rated voltage of 1.1 to 1.2 times nominal power system voltage. For example, 72kV circuit breakers are used for 66kV nominal power system voltage.

(2) Rated current

The maximum value of the load current which passes the circuit breaker should not exceed the rated current. A slightly overloaded current may pass it without problems, which should be limited for a short time only.

(3) Rated breaking current

The rated value of the breaking capacity is expressed by the breaking current value. Calculate the maximum fault current of the circuit used and then circuit breakers with a rated breaking current exceeding the value are to be used. When calculating, the impedance of the rotating alternator will be the sub-transient value. Normally, it is enough to calculate the three-phase ground current as the maximum, but for circuits where the zero-phase impedance is smaller than the positive/negative phase impedance, one-line-to-ground current becomes the maximum and it is necessary to use the maximum fault current.

Table 12-4 Circuit Breaker Types Classified by Arc Control Principle

Type	Symbol	Arc Quenching method
Oil breaker	OCB	The current is interrupted by the blast of the gas generated by decomposition of insulation oil in the arc quenching chamber.
Air circuit breaker	ACB	This is in air and cooled in the arc quenching.
Magnetic circuit	MBB	The arc in air is induced into the arc quenching chamber by magnetic force, and extended and cooled for quenching.
Air blast circuit breaker	ABB	Compressed air is blasted to the arc for quenching.
Vacuum circuit	VCB	This is quenched by quick diffusion of electrons in vacuum.
Gas circuit breaker	GCB	Special gas (SF6) having high insulation is blasted for quenching

Table 12-5 Circuit Breaker Types Classified by Rated Voltage

Type	Rated Voltage (kV)							
	7.2	12	24	36	84	168	300	500
MBB	○							
ACB		○						
ACB	○	○	○	○	○			
GCB			○	○	○	○	○	○
VCB	○	○	○	○	○			

12.4.3 Disconnecting Switch

A disconnecting switch is not used to switch the load current on or off. It is used to disconnect electrical machinery and apparatus from circuit. A disconnecting switch is also used to switch the circuit which is charged at the rated voltage such as the bus and transmission line charging current and the transformer exciting current.

Disconnecting switch types and applicable voltages are shown in Table 10-6. Bus construction and installation methods and other aspects must be considered as a whole when making a selection. The rated voltage and rated current are determined in approximately the same way as the circuit breaker.

Table 12-6 Disconnecting Switch Types and Applicable Voltage

Method of operation	Type	Applicable voltage class
Manual	Vertical, 1 contact	Less than 7.2kV
Motor drive of Air driven	Horizontal, 2 contacts	7.2kV - 300kV
	Horizontal, 1 contact	7.2kV - 500kV
	Pantograph type	168kV - 500kV

12.4.4 Instrument Transformer

Instrument transformers can be classified into potential transformers (PT or PD: Potential Device) and current transformers (CT). They are used to measure high voltage large current electric circuits. They supply voltage/current, in proportion to the circuit voltage and current, to instruments, relay and watt-hour meter based on the principle of the power transformer.

(1) Potential transformer (PT, PD)

Potential transformers can be classified into indoor and outdoor types depending on the installation site, and into a dry type and an oil immersed type by insulation construction and into a winding type and a capacitive type by principle. The winding type is structured based on the same principle as the power transformer and is generally called a PT. The capacitor voltage transformer is generally called a PD and uses a capacitor's potential divide principle. The dry type is used when the voltage is less than 22kV. When the voltage is over 66kV, the capacitive oil immersed sealed type which provides high insulation reliability is used. PD and PT of a gas insulation type combined with gas insulated switchgears (GIS) are developed.

(2) Current transformer (CT)

Classification by used location and insulation construction is about the same as the potential transformer. When classified by coil type, there are a winding type, a through type and a bushing type. The winding type is used for the generator main circuit or for measuring the house circuit bus. The oil or gas immersed type is used for higher voltages.

12.4.5 Arrester

Lightening protective devices are the general term for devices which protect the insulation of electric facilities and equipment from over-voltage. Lightening protective devices include arresters, protective gaps and protective condensers. The arrester is a typical lightening protective device. When over-voltage (impulse over voltage or switching surge voltage) with a high crest value caused by lightening or operation of circuit breakers and other switching devices, an accompanying electric current is discharged to limit over-voltage and protect electric facility insulation. The arrester also cuts dynamic current in a short time and automatically recovers system voltage. There have been various types of arresters. Today, however, a zinc oxide type (gapless type) arrester which enables easier serviceability is used.

12.4.6 Metal enclosed Switchgear

Put away in steel boxes in consideration of safety and security are circuit breakers, disconnecting switches and instrument transformers of each power supply (400V, 200V, 100V and a 6.6kV circuit of the power station), which are called metal enclosed switchgears in a lump. The metal enclosed switchgears bear electricity supply functions for the auxiliary equipment, ventilating devices, and a DC power supply system, etc. These should be divided properly into a charge and a no- charge part at the time of maintenance work to carry out the safe work efficiently.

12.4.7 Control Panel

Operation, control and protection of turbine, generator, main circuit, main transformer, transmission line, short circuit etc. are carried out through control panels. The control panel is designed according to number of equipment and transmission lines, control method, number of operators, and the scale and importance of the power plant.

At the important power stations such as large-scale hydroelectric power stations and pumped storage power stations, plant control system is generally designed with a fail-safe. Also, SCADA system, a network consisting of PLC for sequence control, digital type governor, digital type exciter, PLC for common equipment control, and computers for monitoring and control, shown in the Figure 12-37 is commonly adopted.

In recent years, for a small-scale hydroelectric power station, the integrated control panel is used to be used for cost reduction. The panel is equipped with PLC (Process Logic Controller) which integrates functions of not only start stop sequence control but also excitation control and speed control of the generator by software. An example of an integrated control panel is shown in Figure 12-38.

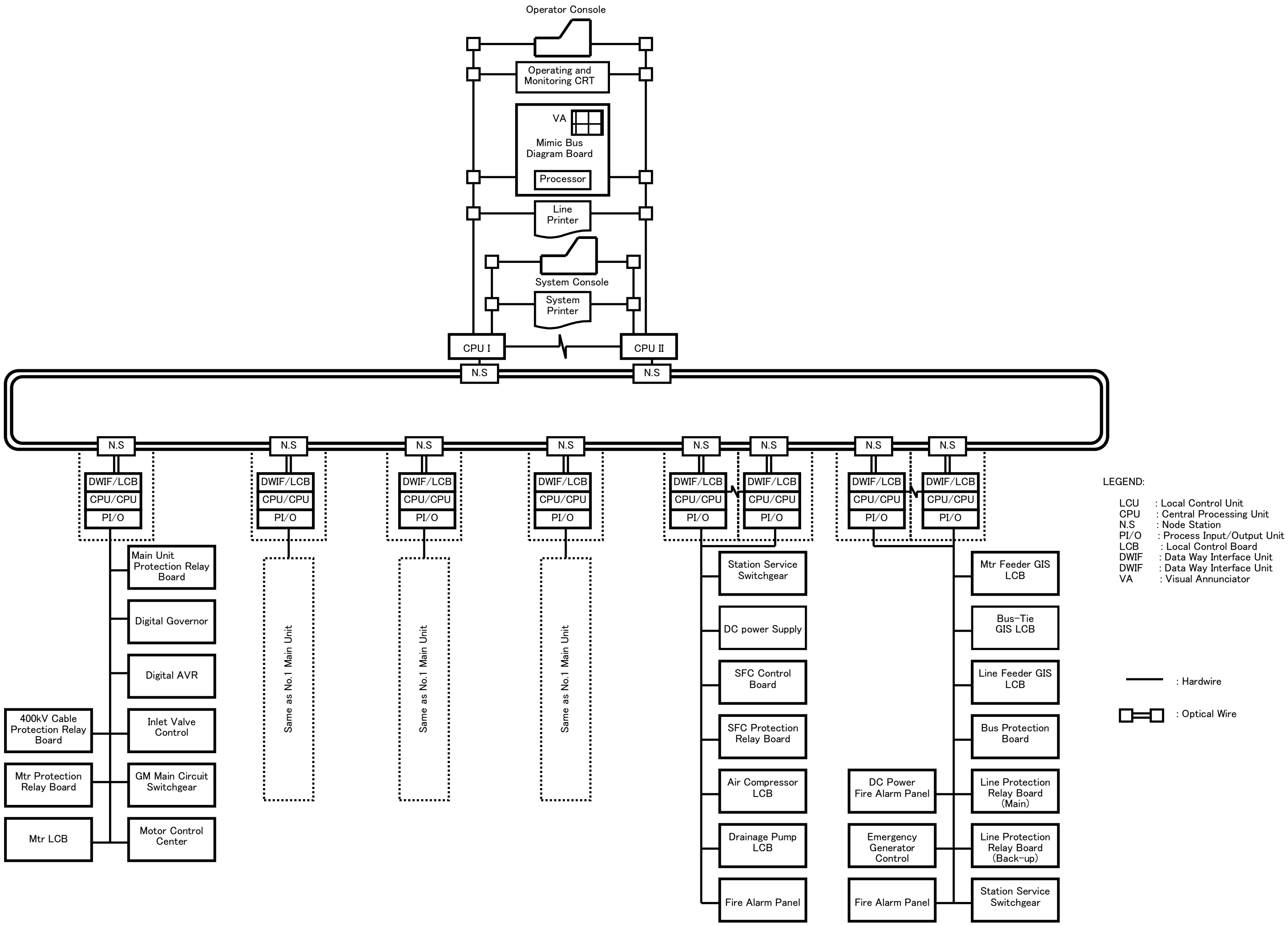


Figure 12-37 Control and Monitoring System for a Large Scale Hydropower Station and a Pumped Storage Power Station

Control and protection

1 Human interface (LCD touch panel)
2 Automatic synchronizer
3 Alarm and fault indicator
4 Lockout relay and alarm reset button
5 Emergency stop button
6 Control switch for circuit breaker
7 WH Meter

Control and protection (internal alignment)

1 Power supply unit
2 CPU unit
3 GOV interface unit
4 AVR interface unit
5 Protective relay (over current)
6 Protective relay (grand short over voltage)
7 Power switch and Remote switch

Auxiliary relay (internal alignment)

1 Auxiliary relay and timer
2 Speed detector
3 Lock out relay
4 Terminals for control cables

Source: Integrated control panel Catalog Toshiba Corporation

Figure 12-38 An Example of Integrated Control Panel

12.4.8 Protective Relay System

Protective devices are installed to limit damage caused by faults and to keep the following at a minimum.

- Cost of damage and repair
- Expansion of damage caused by fault
- Outage period
- Loss to electric utilities and customers

For effective protection, protective systems in hydropower plants are divided into turbine generator protection, transformer protection, bus protection and transmission line protection. A list of protective items of turbine, generator, transformer, etc. is provided in Table 12-7.

Table 12-7 Turbine Generator Protection Items

Protection category	Failure item
Emergency stop	Generator internal short circuit, Generator internal ground, Main circuit short circuit, Generator circuit ground, Generator over voltage, Generator over current Lost of excitation, Main transformer internal short circuit, Main transformer internal ground fault, Excitation system failure
Quick stop	Excess bearing temperature (failure level), Governor failure, Excess speed, Actuator low oil pressure, Low pressure of governor (failure level), Low pressure of oil pressure tank (failure level), Low oil level of sump tank (failure level)
Normal stop in alarm mode	Excess bearing temperature (alarm level), Low oil level of bearing oil tank, Low pressure of oil pressure tank (alarm level), Low oil level of pressure oil tank, Cooling water supply failure, Excitation system malfunction, Stator coil high temperature,
Alarm	High oil level of bearing tank, Air cooler outlet high temperature, Excitation circuit grounding, Cooling water cut off, Start-up impeded, Air tank low pressure, Auxiliary machine malfunction (or breakdown), Drainage pit high water level, Main transformer high temperature

1) Emergency stop

An electrical fault occurs on the generator. The synchronizing circuit breaker is opened instantly in order to prevent the adverse effect on power system, and at the same time, the excitation power is cut off to stop the fault current from generator and prevent aggravation of the fault.

2) Quick stop

A mechanical fault occurs on the turbine or generator. Although a prompt shutdown is required, the mechanical failure could be aggravated if the load is suddenly shut off by a synchronizing circuit breaker similarly to the case of an emergency stop because this would result in a turbine speed rise and a water pressure rise which may have a severe effect on the turbine generator and structures. In this case, the guide vanes are completely closed soon after the failure is detected, then the synchronizing circuit breaker is opened to stop the turbine and generator under the condition of no loads.

Recently at small scale hydropower stations, there have been examples that a quick stop mode is omitted and integrated into an emergency stop mode in order to simplify the control equipment.

3) Normal stop in alarm mode

Although there is no need to urgently shut down the machines as in the case of an emergency stop or a quick stop, there is a risk of aggravating the breakdown, failure, etc. if operation is continued. The machines are shut down automatically according to the normal stop sequence.

4) Alarm

An operator or an inspector confirms the equipment conditions and various measurement data, and exercise a judgment on continuous operation. They either reduce the generator output or shut it down according to the circumstances.

12.4.9 DC Power Supply System

A DC power supply system is supplying a direct current for controlling the control panels, switchboards, protective system; it is comprised of a battery charger, a battery and an inverter. At a small power station less than 100kW, a DC power supply system is occasionally omitted and then AC power is applied for the plant controlling, as long as the water turbine and generator are specified that they should not operate at the outage of the power plant and that they should absorb its run-away speed. In addition, as simplified DC power supply facilities, general-purpose uninterruptible power supply equipment such as UPS, and CVCF (UPS: Uninterruptible Power Supply, CVCF: Constant Voltage Constant Frequency) is adopted.

12.4.10 Operation Control System

An operation control system is selected from the following systems considering the scale of the power plant, its duty within the power system, geographical conditions, the number of operators, etc. Figure 12-39 shows: an example of a generator start control system.

(1) One-man control system

This is an automatic operation system. Turbine and generator operation, load/voltage regulation, gauging and monitoring can be done by one operator using the control panel. In the event of a failure it automatically shuts down or issues a warning signal, depending on the nature of the fault.

(2) Remote supervisory control system

Operation of the turbine and generator, load/voltage regulation, monitoring, etc., are done from a remote control center. In the event of a fault, it automatically shuts down or issues a warning signal depending on the nature of the fault. In many cases, this system is being aggressively adopted due to the advancements in communication and computer technologies as well as operator savings. There are cases where more than ten power plants are collectively controlled by one control station.

(3) Fully automatic control system

This is a fully automatic control system. When the pre-set starting conditions are ready, the turbine and generator start automatically. The load is regulated automatically and operation is continued. The generator is automatically shut down when the set shutdown conditions are formed or in the event of a fault. Conditions of full automatic operation are mainly the time or load set in advance, and water level in a pond or a head tank at a run-of- river hydropower station.

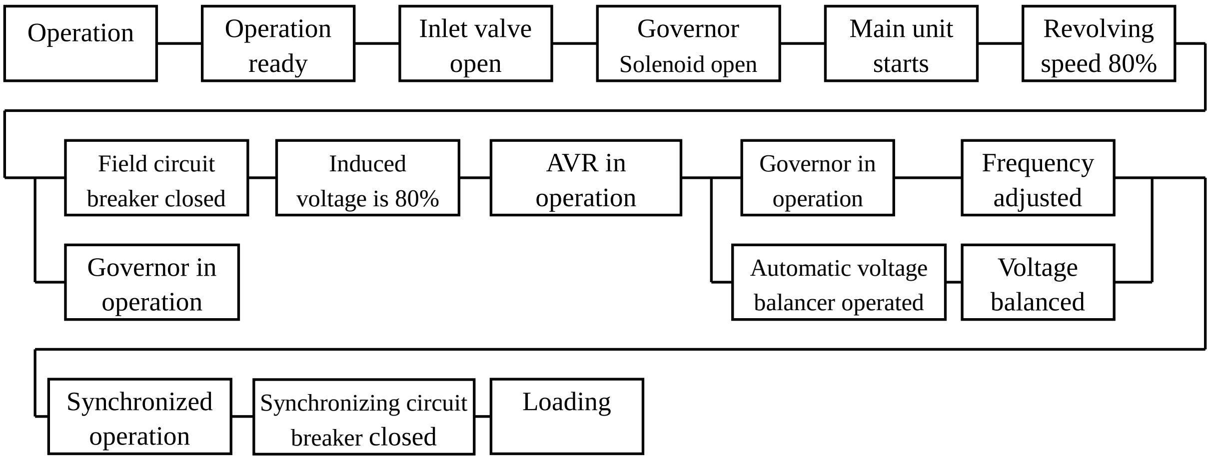


Figure 12-39 Example of Water Turbine Generator Start Sequence

12.5 Other Equipment

12.5.1 Crane

An overhead traveling crane or a gantry crane is installed for installation works and maintenance works of the water turbine and generator in a hydropower station. In this case there are many examples that the maximum hoisting load of the crane is designed to be the hoisting load of the generator rotor. In addition, a crane for carrying-in works of the apparatus and materials may be installed separately from an assembling crane when a power station is installed underground or in a deep vertical shaft. At small scale hydropower schemes, there are examples that a truck crane, a small electric hoist, a hand-operated hoist and a chain block are applied as a substitute for an assembling crane.

12.5.2 Grounding Wire

At the time of a ground fault of a transmission line or a generator, an electric current flows to a ground fault point from the power station. Therefore, a grounding mesh is installed in and around the power station area in order to lower the ground resistance at or less than the specific value, and it is necessary for a safety problem not to occur by the step voltage or the touch voltage. It is indispensable to connect all the electrical equipment such as a water turbine, a generator, auxiliary equipment, metal enclosed switchgears, control panels, etc to a grounding mesh surely.

The designer measures or estimates resistivity of the ground such as local bedrock and designs the area that meets the necessary ground resistance. It is important that grounding wires should be installed like a mesh after excavation works down to the bedrock foundation, and before concrete placement works of the foundations, walls and floors in the power station. And, it is necessary to make ready the ground wire taps to be connected to all the required equipment. The cooperation with the civil constructor is important to adjust the complicated works of reinforcement bar and concrete. Similarly, it is necessary for the grounding wires to be embedded after foundation excavation works in the outdoor switchyards. The ground resistance is measured after the installation of all grounding wires to be confirmed. An additional grounding wire is necessary when the ground resistance does not meet the specific design value.

T Target values of the ground resistance are specified in the electrical preservation standard of the country, and the following values are referred to as well.

A power station at the directory grounded neutral system
(Neutral of the main transformer directly grounded) : Less than 1Ω
A power station at the non directory grounded system (Neutral of the main transformer
non grounding, resistance grounding, reactor grounding, or etc.) : Less than 10Ω

12.5.3 Emergency Power System

The important hydropower station on the power system operation may perform the initial charge operation at the time of the system blackout. Therefore, emergency power system is equipped to supplying electricity for the station service circuit to start a turbine and generator even at the time of a blackout. Or emergency power system for securing of preservation load in the power station is installed for the lost of the oil pressure, the air pressure, DC control power and prevention of the submergence by a blackout of the long time.

For this emergency power system, a diesel engine generator, a gas turbine generator, a small water turbine generator is applied. The designer estimates the facilities capacity of a necessary apparatus at the time of a blackout at the generation capacity study of emergency power system. It is necessary to consider arrangement of station survive circuit and switchgears at the time of a design beforehand so that it is easily changed to emergency power system at the time of a blackout.

Reference of Chapter 12

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[2] Guide manual for Development Aid Programs and Studies of Hydro Electric Power Project, New Energy Foundation, 1996

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[5] TURGOIMPULSE TURBINE Catalog, Gilkes Co.ltd.

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[8] Integrated control panel Catalog, Toshiba Corporation

Chapter 13

Design of Transmission and Transformation Facilities

Chapter 13 Design of Transmission and Transformation Facilities

13.1 System Planning

13.1.1 Purposes of System Planning

Transmission of generated electric power to consumers usually requires connection to already operating electric power systems. Pursuit of cost effectiveness is an extremely important task in this process, since the construction of an electric power system requires great funds, labor, and time. It is also necessary to seek reasonable features based on long-term perspectives, because such facilities are used for a long time once they are completed. Meanwhile, the required reliability differs among different facilities. It is necessary to construct systems while keeping a good balance between economic efficiency and reliability so that one facility would not degrade the reliability of the whole.

Yet, the step involved with actual works is not as simple as “system planning” and “transmission planning.” Starting with rough system planning that is created based on already operating systems and future perspectives, the precision needs to be gradually improved while alternately repeating facility designs and system analyses.

13.1.2 System Categories

Based on functional aspects, electric power transportation facilities can be roughly categorized into a trunk transmission system, a transmission system for regional supply, and a distribution system. The trunk transmission system can be further categorized into a power source transmission system and interconnected system, each of which has the following roles and features. Figure 13-1 shows the basic configuration of an electric power system.

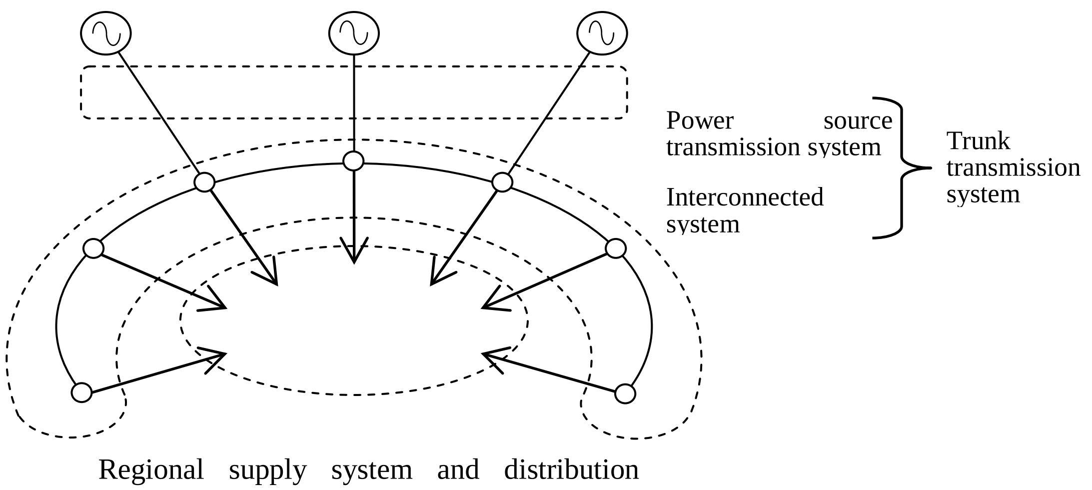


Figure 13-1 Basic Configuration of an Electric Power System

(1) Trunk transmission system

The role of a trunk transmission system is to connect the transmission and systems of a main power source into an integrated system. Power source transmission lines from multiple directions are connected to an interconnected system integrated with trunk transmission lines. The grid power from the power sources are then pooled and distributed to base substations of different regions. A trunk transmission system usually consists of the highest voltage of the system since it has to perform long-distance, large-capacity electricity transmission as the center of an entire system. The highest reliability is also required in a trunk transmission system because there is a risk of affecting the entire system in case of a failure.

(2) Transmission system for regional supply and distribution system

The electricity pooled in a trunk transmission system goes through a power transmission substation installed near an area of consumption and then is supplied to consumers and power distribution substations while gradually being reduced to lower voltages. A distribution system consists of power distribution substations that are at the end of a transmission system as well as distribution lines, transformers, and circuit breakers that connect power distribution substations with consumers. While distribution facilities involve smaller facilities than transmission and transformation facilities, an enormous amount of distribution facilities are required due to the net-shaped horizontal configuration.

13.1.3 Voltage Class

Cost effectiveness of voltage varies depending on load density, transmission distance, and grid power. The following aspects shall be carefully considered when selecting a voltage.

- The voltage shall allow rational connection with already operating systems.
- Site conditions in the vicinity shall be taken into account.
- The voltage shall be selected based on long-term perspectives (such as the expansion of the system).

Yet, when each system individually selects its optimal voltage, devices with different ratings must be used at different locations. It also requires more transforming facilities that connect systems and results in increased cost of transformation constructions and more failures in transformers. Therefore, it is not necessarily an economical option in general. This is why a system voltage needs to be selected to allow simple system configuration whenever possible, and a reasonable option is to select a voltage from previously specified standard voltages.

Table 13-1 shows a part of a voltage standard (IEC60038) that is an international standard specified by IEC. Common electric power equipment available in the market usually complies with this standard voltage; thus, using standard voltages of major standards like IEC enables the use of general-purpose equipment.

In addition, voltages of systems that might be connected with neighboring countries must be determined by taking account of the conditions in the other countries.

Table 13-1 Standard Voltage (High Voltage)

Highest voltage for equipment [kV]	Nominal system voltage [kV]	
(52)	(45)	–
72.5	66	69
123	110	115
145	132	138
(170)	(150)	–
245	220	230

Source: IEC60038, July 2002

Note: The values are voltages between phases. Voltages within ( ) should not be used in new systems.

13.1.4 System Configuration

Trunk transmission systems that process large-capacity transmissions are associated with great effects in case of a failure, and small-capacity distribution systems with limited effects in case of a failure require many facilities. The frequency and duration of an electric outage are used as indicators of the reliability of electric power supplies, which is called “service reliability” in general. Since the continuity and quality of electric power supplies are the opposite of the price of electricity, a certain limit (reliability criteria) should be set as an imminent task. The criteria differs depending on the importance of facilities; yet, the overall concept should be unified (consistency should be ensured) by taking cost effectiveness into consideration. The concept of reliability criteria and its examples of measures to be taken in a system configuration are described below.

(1) Trunk Transmission System

- A trunk transmission system shall not cause outages or limit electric power generation of major power sources in case of a single-facility accident.
- A trunk transmission system shall prevent wide-area power source fallout or a disconnection of system in case of a double facility accident. Even when a disconnection occurs, individual systems shall not cause outages after a disconnection whenever possible, and stable system operations shall be maintained.
- The above reliability features shall be maintained even during a facility shutdown for repairs or maintenance.
- Example of measures: Install two circuits of transmission lines or make it a loop, or make a dual transforming facility (transformers and bus).

(2) Transmission systems for regional supplies

- A system shall be capable of quickly recovering electric power supplies in case of a single-facility accident. (The level of a recovery shall be individually designed depending on working conditions.)
- A system shall prevent outages during a single accident as much as possible while processing especially important loads.
- The above reliability features shall be maintained even during a facility shutdown for repairs or maintenance.
- Examples of measures include making a transmission system a loop, or installing a circuit to connect to emergency equipment.

In addition, each piece of equipment has circuit breakers with different abilities to shut down fault currents. Fault currents increase as more power sources are connected parallel to a system; thus, it is necessary to be cautious so that the fault currents would not exceed the capacity of circuit breakers. Reinforcement (replacement) of equipment becomes necessary in case a fault current exceeds the capacity (when a circuit breaker for fault clearing cannot shut down a fault current). Thus, consider dividing a system as needed when reinforcing a power source.

Meanwhile, a system becomes more cost effective when a substation is located near the area of consumption. Yet, an area without restrictions on the connection with transmission and distribution lines is favorable for a substation, and required conditions include that spaces are available for inspection and repairing of installed equipment. Positions of installation need to be determined while keeping a good balance of the entire area, since changing the position of a substation affects the construction cost of transmission and distribution lines.

13.1.5 Quality of Electric Power

Besides service reliability, the following criteria are generally used to measure the quality of electric power.

- The absolute value of a frequency and its variation range
- The absolute value of a voltage and its variation range

Low quality of electric power causes various problems. For example, unstable voltage and frequency cause defects in factories. Also, long and frequent electric outages might drive consumers to choose private generators, which make it difficult to recover capital investment due to the lack of expected demand. This is the reason that it is necessary to maintain a certain quality in electric power.

To maintain a certain quality in electric power and eliminate these risks, a target value (allowable variation range) for a voltage and frequency needs to be specified, and a system configuration needs to be designed to ensure the target value. Table 13-2 and Table 13-3 show examples of target values of voltages and frequencies.

Table 13-2 Target Voltages of Electric Power Systems

[Unit: %]

	Power station (terminal voltage)	Substation (secondary bus)		
		Trunk transmission system	Major system in a region	A system with common loads
Under heavy load	100 - 102	100 - 105	100 - 102	100 - 105
Under light load	95 - 98	95 - 105	95 - 100	95 - 100

Source: *Power transmission and Distribution*. The Institute of Electrical Engineers of Japan, August 2001

Table 13-3 Target Frequencies of Electric Power Systems

North America (NERC)	Annual standard deviation of one-minute average	
	East	0.018 Hz
	West	0.0228 Hz
	ERCOT (Texas)	0.020 Hz
	Hydro-Québec	0.0212Hz
	Annual standard deviation of ten-minute average	
	East	0.0057 Hz
	West	0.0073 Hz
	ERCOT (Texas)	0.0073 Hz
	Hydro-Québec	0.0125 Hz
Europe (UCTE)	Occupancy ratio within	50±0.04Hz, 90% or more
	Occupancy ratio within	50±0.06Hz, 99% or more

Source: *Technical Report of the Institute of Electrical Engineers of Japan*. No.869, The Institute of Electrical Engineers of Japan, March 2002

13.1.6 System Stability

The stability of an electric power system is roughly categorized into phase angle stability, which discusses the propriety of synchronizing generators, and voltage availability, which discusses the presence of voltage collapse.

(1) Phase angle stability

Phase angle stability is criteria that indicate whether a system can continue parallel operation without causing step-out synchronization in some or all of the generators in the system when some disturbance occurs while generators connected to the system are rotating at the same speed (synchronous speed) during AC transmission. It is generally expressed based on the degree of stability power limit. The stability power limit increases in approximate proportion to the square of voltage, and it decreases in approximate inverse proportion to a transmission distance under the same voltage.

A rational concept of phase angle stability is to divide it into the following three concepts depending on time domain.

(a) Transient zone

- Stability of initial time domain (about one to two seconds) in which the phase angle between rotors is disturbed after a disturbance such as short circuit or ground fault affected a system.
- While the effects of external disturbance such as conditions or duration of an accident are great, effects of the responses of generator control systems are often relatively small.

(b) Intermediate zone

- Stability during several seconds to around 15 seconds of electricity power disturbance after the onset of disturbance following a transient zone
- Greatly affected by responses and load characteristics of generator control systems with nonlinear characteristics

(c) Static zone

- More than about 15 seconds of stability, which is longer than the intermediate zone
- Greatly affected by system-specific unstable modes, which are determined by the synchronizing power between generators, the inertia of generators, and responsiveness of control systems
- The stability against a relatively mild disturbance such changes in generator outputs is considered to be the stability in this zone.

(2) Voltage stability

Voltage stability indicates the margin up to a stability limit at which a voltage collapse is triggered. A voltage collapse occurs when a supply cannot keep up with a rapid load increase or an increase in reactive power loss caused by a shutdown of transmission and transformation facilities, which cause the voltage to drop and trigger a wide-area load drop.

A feature (P-V curve) that indicates the relationship between the voltage and the active voltage of a load is used to determine voltage stability. Figure 13-2 shows the simplified model of a system (the pair of one device and one load).

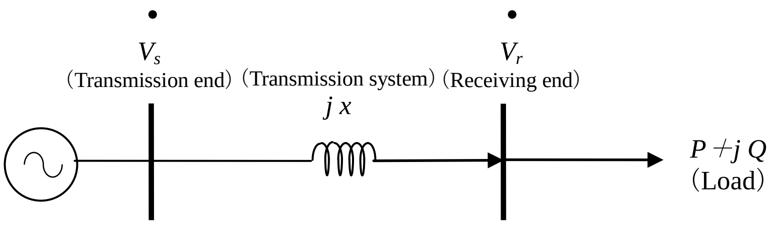


Figure 13-2 A Model of the Pair of One Device and One Load

In this model, the relationship between the receiving power and receiving voltage can be expressed in the equation below.

$$V_r = \frac{V_s^2 - 2Qx}{2} \pm \sqrt{\left(\frac{V_s^2 - 2Qx}{2}\right)^2 - x^2\left(P^2 + Q^2\right)}$$

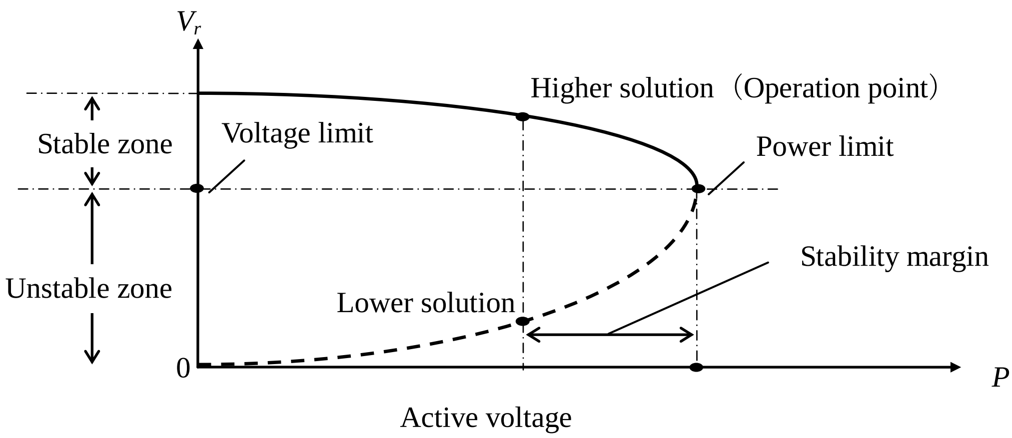


Figure 13-3 P-V Curve

Figure 13-3 shows the relationship in a diagram. The value P, which becomes the largest in this characteristic, is called "power limit" and its voltage "voltage limit." There are two solutions (*V*) for the same power P, and the upper solution is called "higher solution" and the lower solution "lower solution." When there is a slight decrease in a load, the load increases with the increase of voltage and returns to the original operation point in the higher solution part, whereas the voltage further decreases as the load decreases in the lower solution part, which makes recovery difficult. Thus, the part below the voltage limit (the parts indicated with a broken line) becomes the unstable zone, and the operation point becomes located on the higher solution. The condition in which the voltage stability is low means there is a small margin of active power (the operation point and limit value are close).

Stability analysis becomes extremely difficult as an electric power system becomes larger. Thus, the computer-based analysis is common today. Different software requires different data, but the following data is usually required. Standard values are used when data is unavailable or unknown, but actual data needs to be entered whenever possible to ensure accuracy. Various factors of old devices are difficult to obtain; thus, factors should be made available when installing devices.

- Expected electricity demand (during the period for which system analysis is to be conducted)
- Line constant associated with impedances such as transmission lines, distribution lines, and transformers
- Transmission lines and distribution lines: Number of circuits, positive-phase resistance, positive-phase reactance, positive-phase capacitance
- Transformers: Positive-phase reactance, tap ratio
- Constants associated with generators, etc.
- Rated capacity of a generator, rated output, inertia constant, synchronized reactance (*Xd*, *Xq*), transient reactance (*Xd'*, *Xq'*), sub-transient reactance (*Xd''*, *Xq''*), armature leakage reactance (*Xl*), armature constant (*Ta*), zero-phase reactance (*X0*), negative-phase reactance (*X2*), automatic voltage regulator data, governor data
- Concept of the order of generator operation priority and output range
- Availability of phase modifiers (type, capacity (conditions of operation))

13.1.7 Technical Problem Examination Flow

Figure 13-4 shows the examination flow of technical problems in the system planning of power source development plans.

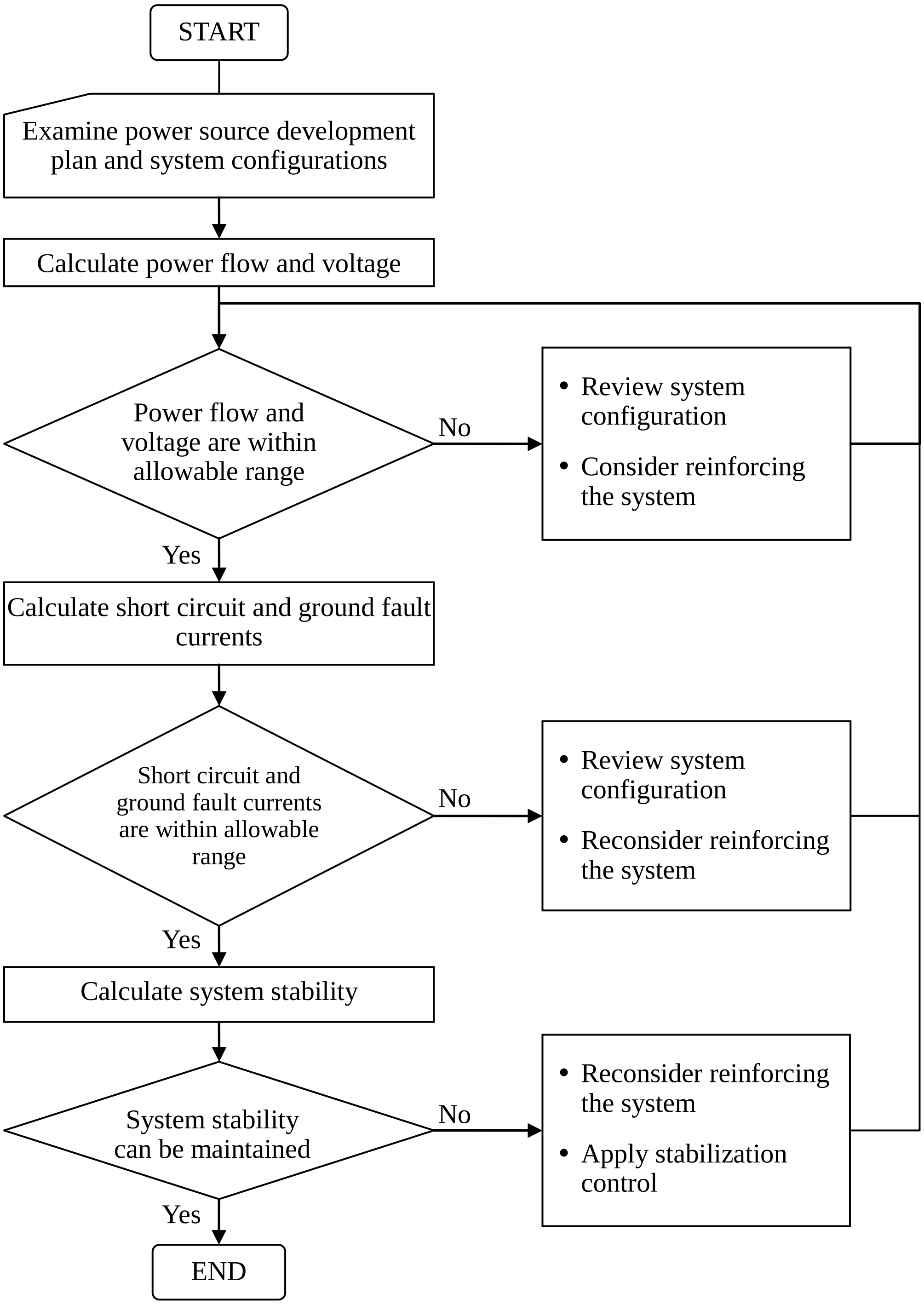


Figure 13-4 Example of Technical Problem Examination Flow

13.1.8 Example of an Improvement of Technical Problems

The following sections describe representative examples of improving a situation when problems are found in the examination flow in Figure 13-4.

(1) Actions concerning power flow

- Change system configurations.
- Increase the number of conductors.
- Increase the size of conductors.

(2) Actions concerning fault current

- Change the system configuration and reduce the short circuit capacity in the area with problems.
- Use a high-impedance transformer and reduce the short circuit capacity.
- Install a current limiting reactor and inhibit fault current.

(3) Actions concerning system stability

Consider implementing the actions in Table 13-4 for corresponding areas with problems.

Table 13-4 Actions to Improve Stability

Actions	Transient zone	Intermediate zone	Static zone	Voltage
Use higher transmission voltage	○	○	○	○
Transmission line and transformer reinforcement	○	○	○	○
Reduce the reactance of generators, transformers, and other equipment	○	○	○	○
Install a serial capacitor	○	○	○	○
Install a static var compensator	○	○	○	○
Install intermediate switching stations on transmission lines	○	○	○	○
DC interconnection	○	○	○	–
Adopt a high-speed excitation system	○	–	–	–
Adopt a power system stabilizer	–	○	○	–
Install a dumping resistor	○	–	–	–
Adopt a turbine high-speed valve control	○	–	–	–
Adopt high-speed protective relay method or high-speed breakers	○	–	–	–

13.2 Transmission Planning

13.2.1 Procedure of Designing the Rroutes of Transmission Lines

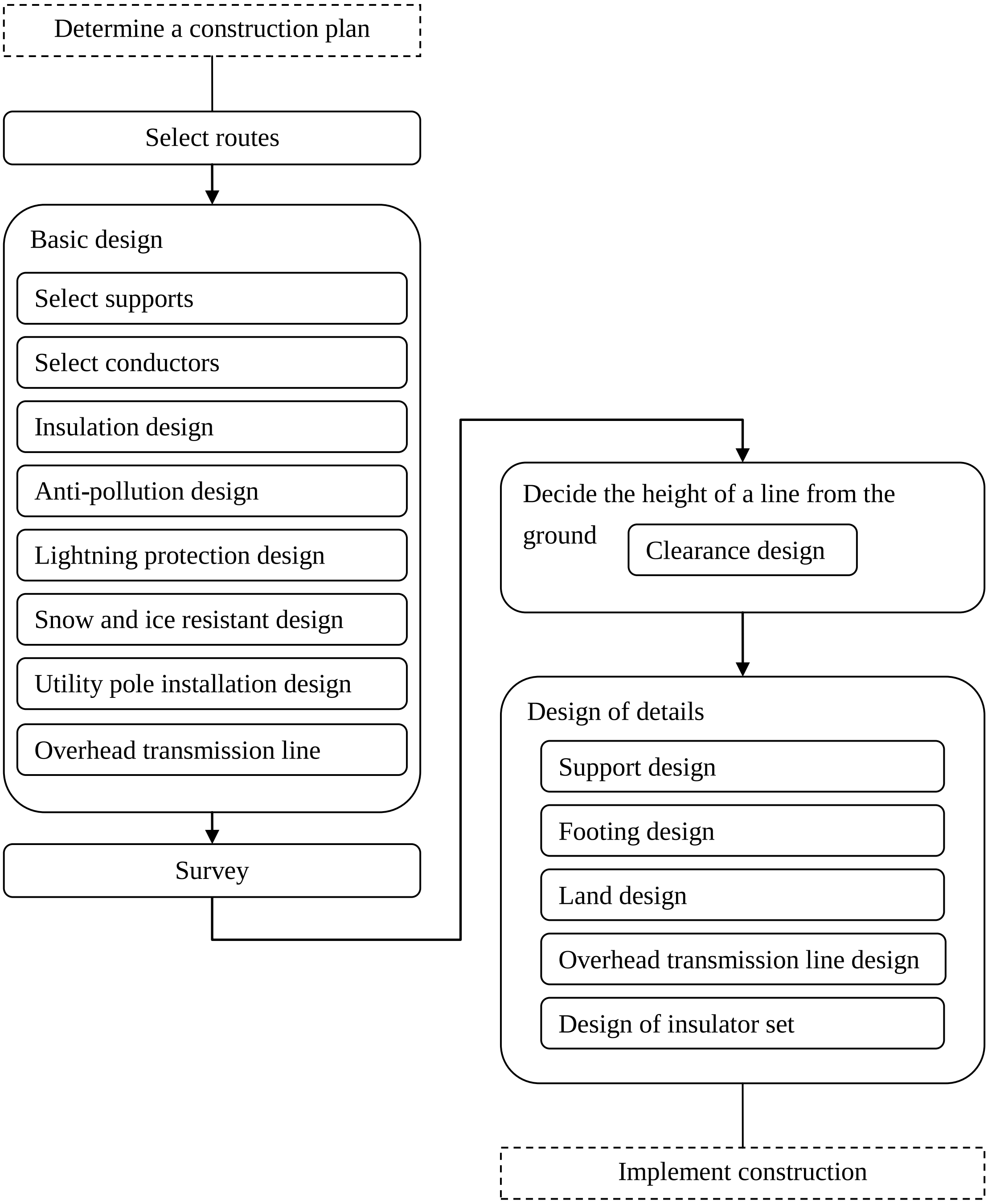


Figure 13-5 Operation Flow of Designing the Route of Transmission Lines

13.2.2 Select Routes

(1) Procedure for selecting routes

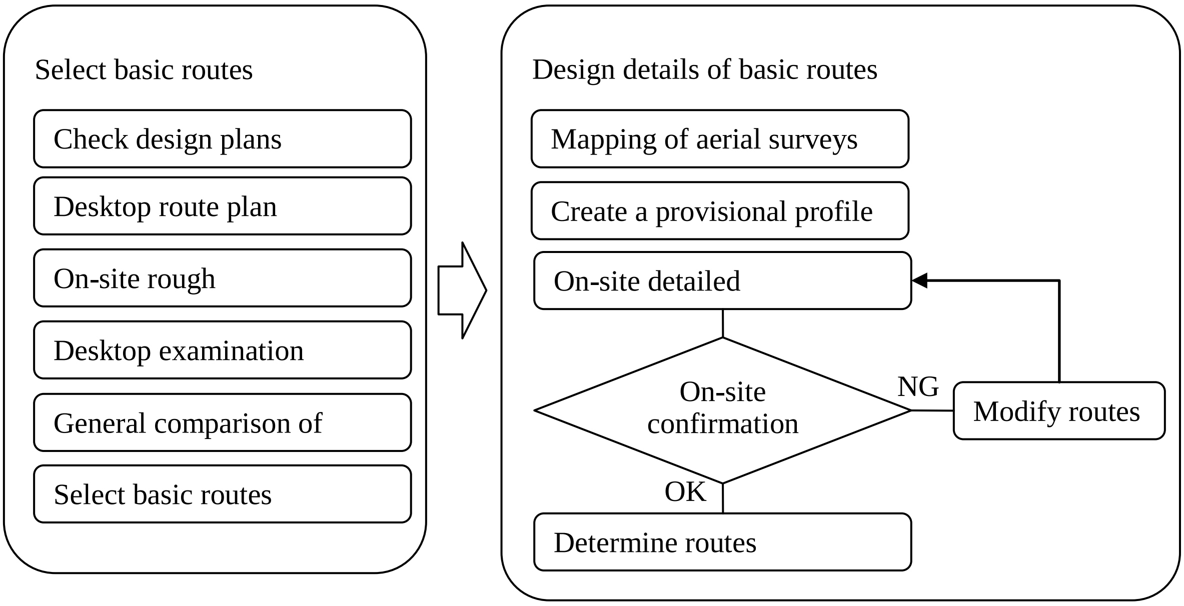


Figure 13-6 Procedure for Selecting Routes

(2) Route selection criteria

Route selection of transmission lines greatly affects the reliability and cost effectiveness of the facilities, and it is one of the most important operations in the construction of transmission lines. Thus, passing routes and positions of individual supporting structures must be selected to ensure optimal performance based on sufficient investigation and examination of the following conditions.

1) The facilities should not be greatly affected by various disasters.

Damage by lightning, contamination, ice and snow, landslide, ground subsidence, avalanche, etc.

2) The general construction cost should be cost effective.

Cost of land acquisition, footing construction cost, transportation cost, cost of environmental features, overhead line construction cost, etc.

3) Make proper adjustments with legal restrictions on land uses and various development plans.

4) Be cautious not to damage natural environment, social environment, cultural heritages, etc.

Meteorological data is extremely important information required in various designs. Outdoor temperatures, amount of snowfall, and wind speed affect power transmission capacity and the tension of power lines (strengths of supporting structures); thus, reliable meteorological data based on long-term observations should be used in this process. When long-term meteorological data is not available, estimates must be made based on short-term meteorological data. IEC60826 (Design criteria of overhead transmission lines) provides a detailed description of the method of making estimates. Since it greatly affects the reliability of the facility, however, observations should be started as early as possible once a clear plan is made.

(3) Comparison of routes

When selecting routes, compare multiple route proposals (about three proposals) based on selection criteria and then select one optimal proposal.

13.2.3 Basic Design

(1) Select an optimal basic design based on sufficient investigation and examination of various conditions for selecting supporting structures.

1) Tower

Towers include angled steel towers, which mostly use equal angle steels as structural members, and steel pipe towers, which mostly use hollow steel pipes as structural members. Towers are better in terms of construction and maintenance because of their large span lengths. Towers also allow joint uses by multiple circuits and have longer service life. One of the advantages of towers is that they allow designs that withstand various loads such as in areas with frequent icing or heavy snow, mountainous regions with large differences in elevation, areas that require long span lengths and supporting structures for high towers such as when lines cross straits or rivers.

2) Steel-reinforced concrete pillars

Steel-reinforced concrete pillars are highly durable and fire-resistant semi-permanent supporting structures without the risk of decaying. They are used in locations to which construction materials can be easily transported, such as in flat areas, cities, suburbs, and along roads. The construction cost becomes higher in mountainous areas with steep geographical features because transportation and on-site pillar installation are difficult in such areas.

3) Iron pole

Iron poles include square iron poles, steel plate built-up columns, and steel pipe build-up columns. The construction materials and designs are mostly the same as towers. They are used in areas with small loads, however, because they are less strong and have narrower footings than towers. Iron poles are also used as the alternative of steel-reinforced concrete pillars when it is difficult to use them in areas such as mountainous regions.

(2) Selection of conductors

Types of power lines should be selected based on comprehensive examinations including the size of current as well as environmental conditions, installation conditions, ease of maintenance, and cost effectiveness. The thickness of power lines should be selected based on long-term perspectives rather than simply based on proximal power flow. When selecting overhead earth wires, ones with sufficient strength should be selected as well. The strength of power lines is determined by types and thickness. Still, be careful that safety factors are often specified in technical standards or other regulations, because it is necessary to take age-related decrease in strength into account.

Transmission capacities of power lines are determined based on both the specifications of the power lines and the environment, which makes meteorological data an important aspect. CIGRE and IEC provide suggestions on calculation methods, but different methods are used depending on environmental factors to be considered. Table 13-5 shows examples of the calculation of safe currents (continuous allowable current). Calculation conditions are as follows.

- Conductor temperature: 90 °C (ACSR), 150 °C (TACSR)
- Ambient temperature: 40 °C
- Frequency: 60 Hz
- Solar radiation: 0.1 W/cm^2
- Absorptivity (emissivity): 0.9
- Altitude: 0 m
- Wind direction: Right angle to the power line axis
- Wind speed: 0.5 m/sec

Table 13-5 Comparison with Current Capacity (Safe Current)

Type of power line	CIGRE	IEEE	IEC	The Insitute of Electrical Engineers of Japan
ACSR 160mm^2	503	490	487	454
TACSR 410mm^2	1,424	1,398	1,364	1,323
TACSR 810mm^2	2,135	2,106	2,065	1,998

Source: *Technical Report of the Institute of Electrical Engineers of Japan*. No.660,
The Institute of Electrical Engineers of Japan, December 1997)

(3) Insulation design

Determine an insulation strength for internal abnormal voltage (switching surge, short-time overcurrent) to prevent flashover, and coordinate the insulation with the entire transmission lines and the substation that draws in the transmission lines.

Implement measures against lightning strike using lightning protection design, because different measures are required on lightning strike depending on locations and supporting structures.

(4) Anti-pollution design

Estimate the amount of salt adhesion caused by seasonal winds in applicable areas, and determine the type and quantity of insulators to be connected so that they will withstand normal ground voltage with the salt adhesion. Also, identify conditions in areas with special risks of taint damage in regions such as industrial areas and reflect the findings in designs whenever possible.

(5) Lightning protection design

Installation of overhead earth wires is especially important for protecting power lines from direct lighting strikes, reduction of the induction voltage due to induced lightning, prevention of tower voltage from rising due to lighting strike, and reduction of electromagnetic induction voltage. The resistance can be sufficiently reduced basically when overhead earth wires are installed on all supporting structures. Ensure the effects by installing multi-layers of overhead earth wires on especially important lines and substation inlets.

(6) Snow and ice resistant design

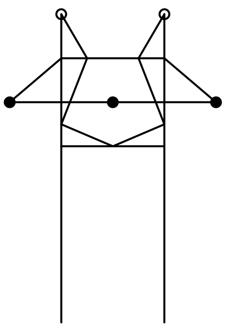
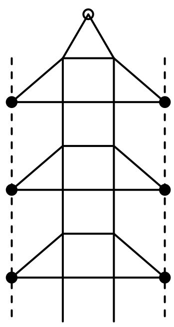
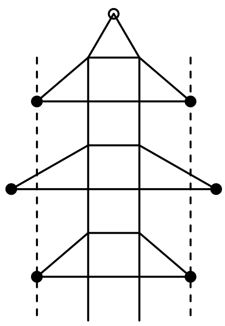
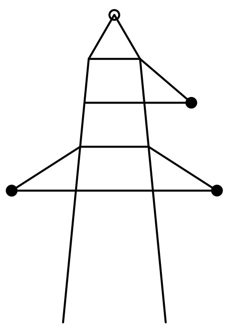
Adhesion of ice and snow on transmission lines not only makes the lines heavy but also enlarges areas exposed for wind, which increases the tension on supporting structures. Thus, transmission lines that pass through areas with the risk of ice and snow adhesion should be designed to ensure electric and mechanical resistance against the expected ice and snow adhesion.

(7) Utility pole installation design

Utility pole installation means to determine power line arrangement and positions and lengths of crossarms based on intervals between power lines and ground interval of power lines.

Figure 13-6 shows examples of basic utility pole installation. The application is determined by creating and checking a clearance diagram (a diagram used to verify that minimum spaces between supporting structures and utility poles can be maintained even when power lines and insulators are swinging sideways).

Table 13-6 Examples of Utility Pole Installation

Name	Horizontal arrangement	Vertical arrangement	Hexagonal arrangement	Triangle arrangement
Shape				
Remark	Used to secure spaces between required power lines when ice and snow are falling.	Used when spaces between lines are narrow.	Used to secure offset to secure spaces between required power lines when ice and snow are falling.	Used with one-line utility pole installation.

(8) Overhead transmission line design

Transmission lines expand and contract under the ambient temperature and the temperature caused by load current, and these changes are expressed as changes in sag. In general, transmission lines begin to show notable expansion when heavy load and extremely hot temperature overlap, and the sag becomes the largest. Meanwhile, the sag also may become the largest in winter in areas where ice and snow adhere on transmission lines because of the effects of the weight of the adhered ice and snow and the increased areas exposed for wind. In any cases, it is necessary to design transmission lines so that separation distances will not become smaller than the expected values, or that excessive tension will not be applied on supporting structures.

13.2.4 Clearance Design

Values specified by relevant laws must be complied for the separation distances and ground height of transmission lines and adjacent structures. Determine these features based on the examination of various situations such as side swings caused by winds, collapsed trees, and effects of fire.

Aspects to be considered differ depending on areas to pass through. For example, ground heights must be determined by taking heights of structures and meltdown during fire into account in areas densely populated with buildings. Tree growths based on tree-trimming intervals must be taken into account in mountainous areas.

13.2.5 Design of Details

As discussed in “Purposes of system planning,” facility design must be implemented along with system planning. Details are designed based on basic designs and survey outcomes, but the basic designs must be checked to make sure that they are problem-free before the design of details begins. When problems are found, either reevaluate the basic designs or create a design to avoid the problems with system configurations.

(1) Design of supporting structures

Design details of supporting structures selected in basic designs. Examples of aspects that need to be examined when using towers are as follows.

1) Decide the shape of the tower
2) Calculate expected loads (high temperature season, low temperature season, with snow adhesion)
3) Calculate member stress
4) Decide framework members and bolts
5) Calculate tower weight and basic stress
6) Consider and design reinforcement

(a) Consider reinforcing the curves of the main materials of angle steel tower crossarms
(b) Consider secondary stress on crossarms
(c) Consider the load of line drawing work
(d) Consider the load of line stringing work
(e) Consider using flange bolts at bend points of main pole materials
(f) Consider using relevant materials of ground wire installation
(g) Consider using auxiliary facilities

- Consider using flat ladders and ladders on main pole materials
- Consider installing aeronautical ground lights
- Consider installing cable racks

7) Special considerations

(h) Consider running spatial analyses
(i) Examine flexibility

(2) Footing design

Footing is categorized into vertical load footing and moment load footing based on dominant load application. Each of them is further divided into four types of footings based on methods in which they transmit loads to the ground: spread foundation, pile foundation, pier foundation, and anchor foundation.

Commonly used square towers distribute vertical loads and horizontal loads applied on a tower to four legs. Since each leg works with compressive force and lifting force, however, the integration of these aspects needs to be taken into account. Figure 13-7 shows the components of loads applied on the foundation of towers.

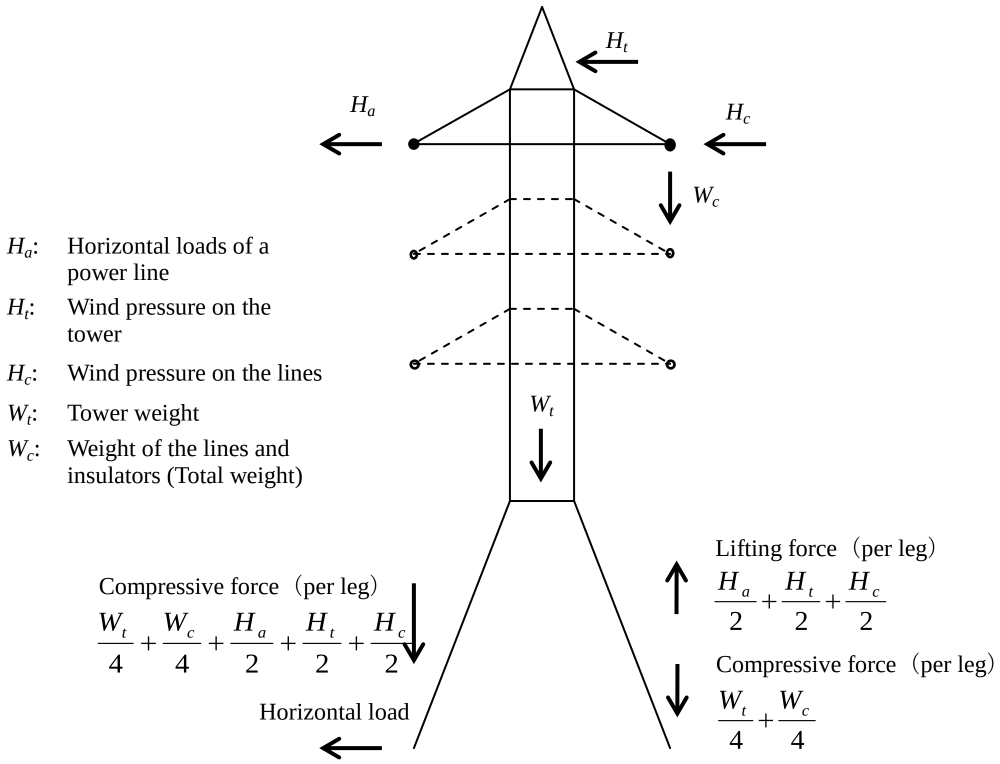


Figure 13-7 Load Components of Tower Foundation

(3) Land design

When constructing a tower on slopes such as in mountainous areas, sometimes expected foundation resistance cannot be obtained on the lower side of the tower due to the lack of soil on the base plate. In such cases, individually examine the lowest joints of the tower and its foundation based on survey outcomes of the property and implement necessary measures.

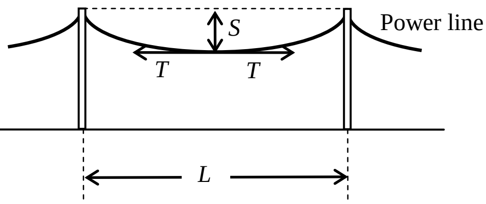
(4) Overhead transmission line design

Leaving power lines tight with small sags during summer causes the lines to constrict and increase the tension in winter when the temperature drops and increases loads on power line supporting structures. If snow and ice adhere on such lines, the weight and the area exposed for wind increase, and the increased tension on power lines may result in major accidents such as the collapse of the supporting structures.

On the contrary, keeping a large sag in winter lowers the power line tension, which is a safe side of measures on power line supporting structures. The sag further increases when the power line extends in summer when the temperature rises, which may cause risks such as that the power line gets closer to or comes in contact with surrounding trees or structures.

Therefore, sags must be determined so that the maximum working tension of power lines stays constant in the worst conditions of the design (high temperature period, low temperature period, and with the weight of snow adhesion). Power lines between two supporting points show a catenary curve, and the sag is usually calculated using the simple equation below.

$$S = \frac{W \times g \times L^2}{8T}$$



S	:	Sag (m)
W	:	Weight of the power line (kg/m)
g	:	Gravity acceleration (N/m)
T	:	Horizontal tension (N)
L	:	Span length (m)

Figure 13-8 Sag

Calculation of sag for actual stringing expects a case in which the combined weight of the line and the weights of adhered snow and wind pressure are applied on the power line. The sag is determined so that the line shows the maximum tension under the combined weight.

(5) Insulator design

Insulators used on transmission lines must electrically insulate power lines from the ground, and mechanically and constantly support power lines. Therefore, the dielectric strength must be withstand not only normal voltages but also continuous abnormal voltages during a ground fault on one line and internal abnormal voltages such as switching surges that occur during line switching. Mechanically, insulators are required to have sufficient strength against line tensions as well as external loads.

Meanwhile, incorporating external abnormal voltages such as lighting strikes causes excessive designs. Thus, protect the insulator by causing a flashover with the arc-horn installed on the insulator to prevent side flashover. Thus, required dielectric strength on temporary overvoltage is usually determined based on switching surges.

Insulator designs are usually examined and determined based on aspects such as system configurations of lines, meteorological conditions in areas to be passed through, and the reliability required in lines. Types and configurations of insulators must be determined by specifically considering the salinity, dust, and smoke in areas to be passed through, because these accretions significantly degrade insulating performance.

Reference of Chapter 13

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[5] Transmission and Distribution Electrical Engineering Newness, 1999

[6] Design Criteria of Overhead Transmission Lines, IEC60826, 2003

[7] Current Capacity of Overhead Transmission Lines (in Japanese), Technical Report of the Institute of Electrical Engineers of Japan. No. 660, 1997

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Chapter 14

Construction Planning and Construction Cost Estimate

Chapter 14 Construction Planning and Construction Cost Estimate

14.1 General

In the feasibility of a hydropower development project, the construction plan and the construction schedule are formulated and the construction cost is estimated with the following objectives:

i) To formulate the construction plan on the basis of the construction methods that are appropriate for the present engineering level taking into consideration the site conditions of the project;

ii) To estimate the total construction cost of the project to be used as the basis for arranging the necessary project implementation budget by the organization(s) that will execute the project; and

iii) To prepare the basic data for the determination of the annual expenditure of the hydropower development project based on i) and ii) above to be used in the economic and financial analyses of the project.

The factors requiring attention in the construction planning, construction scheduling and construction cost estimation are discussed below.

14.2 Construction Plan and Construction Schedule

14.2.1 Study

In formulating the construction plan and the construction schedule, sufficient advance study and verification of the design, the construction conditions, and the topography, geology, meteorology, hydrology and siting conditions are necessary.

The study includes the following aspects:

- Meteorology: temperature, rainfall, number of rainy days
- Hydrology: flood discharge, occurrence and frequency of flood
- Water quality: temperature, turbidity, pH of river water
- Topography and geology: topographic map (incl. temporary facility site area), geologic map and investigation work data
- Site conditions: Geographic location, social and living environments in the vicinity and applicable environmental laws, regulations
- Construction conditions: procurement of construction equipment, materials and labor
- Transportation: general transportation condition in the vicinity of the construction area (transportation means, routes and capacities)
- Power for construction: availability and capacity of existing power transmission lines near the construction areas

- Construction water: Quantity and quality of river water
- Construction site: Confirmation of procedures relevant to land acquisition

14.2.2 Construction Plan and Construction Schedule

A hydropower development project is comprised of preparatory works; civil works; installation works for the gates, penstocks and other hydraulic equipment; installation works for the turbines, generators and other electrical equipment; and transmission and distribution line works. Simultaneous execution of these different types of works at the same location is a distinct feature of the hydropower development project. Generally, except for the transmission and distribution line works, the construction plan and the construction schedule for civil works influence the installation schedules for the hydraulic and electrical equipment.

The project executing agency must consider the following points in the construction planning for the civil works.

(1) Preparatory works and temporary facility plan

The planning of preparatory works and of the temporary facilities, should take into consideration all the requirements for proper maintenance and administration during the construction period and after the start of operation of the hydropower plant.

(a) Transportation facilities

The implementation of a hydropower development project would necessitate the upgrading and reinforcement of the existing transportation facilities and/or the installation of new ones to facilitate the movement into the project site of materials, equipment, explosives, cement, aggregate, structural steel, heavy construction machinery, hydraulic equipment such as gates and steel conduits, and electrical equipment.

These facilities should be properly planned taking into account the available means of transportation, which include trucks, railroads, and inland and marine transport. In examining their use, cargo types and size, vehicle sizes, road widths and alignments, tunnel cross-sections, bridge structures and load limitations, and harbor facilities should be studied.

(b) Construction roads

Construction roads are roads to be built solely for the purpose of implementing the project.

These are apart from the existing roads. Some sections of such roads will be used for power plant operation and management following the completion of construction.

The selection of routes and road structures such as the width, alignment, gradient and sub-base are based on the type of vehicles, traffic volume, vehicle speed and vehicle load, and in consideration of the topography and geology of the site.

(c) Construction buildings

The construction buildings are listed below. The number, size, type of structure and location of such buildings depend on the location of the work, magnitude of the project, construction period, and topography of the construction site.

i) Site office

ii) Living quarters and annexes

iii) Motor pool for construction machinery

vi) Repair shop

v) Fabricating yard for reinforcement bars

vi) Temporary assembly yard (hydraulic equipment, etc.)

vii) Carpentry shop

viii) Laboratory

ix) Explosives warehouse

x) Other warehouses

xi) Materials storage yard

vii) Other buildings (parking lots, first aid station, etc.)

(d) Power and communication facilities

The power supply facilities for the construction works should have sufficient capacity to provide adequate and dependable power supply to the power-consuming facilities as required during the construction period.

The capacity (kVA) of the construction work power facility should be able to fully meet the demand of the maximum power consumption month, based on the monthly total power consumption calculated from the construction schedule.

The major power consuming facilities are the various plants, air supply equipment, water supply and drainage systems, and lighting equipment. Where construction sites are scattered, power distribution and communication facilities are required to connect such locations.

(e) Water supply facilities

Water for construction works are consumed at plants for cleaning, cooling water of machines, sprinkling and camp. The water intake is to be set at a convenient location in the river or marsh and close to the construction work site. In calculating the capacity of the water supply facilities, the required water quantity, the head and the piping length are taken into consideration. The maximum water consumption volume should provide a 10% to 30% margin to the required water quantity.

(2) Construction planning

(a) Dam

The construction plan of a dam and intake is accompanied with an appropriate river diversion plan. In general, concrete dams are not critically damaged by overtopping of water, but in the case of fill dams overtopping of water would result in construction delays and failure of dam.

Therefore, in river diversion planning, utmost care must be taken in the determination of river flow and in the design of the diversion structures and cofferdam.

The types and required number of construction equipment are determined in consideration of the work volume, construction conditions and construction methods. After the selection of the appropriate construction equipment, the construction plan and construction schedules are determined taking into account the hydrologic and meteorologic conditions, available number of work days during the year and the number of operation hours for each work day.

(b) Headrace

The construction planning and construction scheduling for headraces (open channel, tunnel) are generally prepared in the following sequence;

i) Study of construction method (excavation, lining)

ii) Calculation of mean monthly excavation progress and mean monthly lining progress

iii) Construction facility planning

The construction method of the headraces must be studied with full understanding of the geological conditions. The construction speed should be so calculated as to provide for contingency for possible changes in the geological conditions.

In the case of tunnels, the low working efficiency periods at the tunnel entrance and in the early stage of the works must be taken into consideration in construction planning. When the headrace is long, proper construction plan, such as increasing the number of work adits, must be adopted in order to synchronize the headrace construction period with that of other works.

(c) Penstock

The construction work for the penstocks is preceded by the excavation of the penstock route and the installation of penstock is performed. Since penstock installation speed is influenced by the length of the steel conduit, the optimum unit pipe length is determined in consideration of the overall construction schedule and the transportation conditions.

(d) Powerhouse

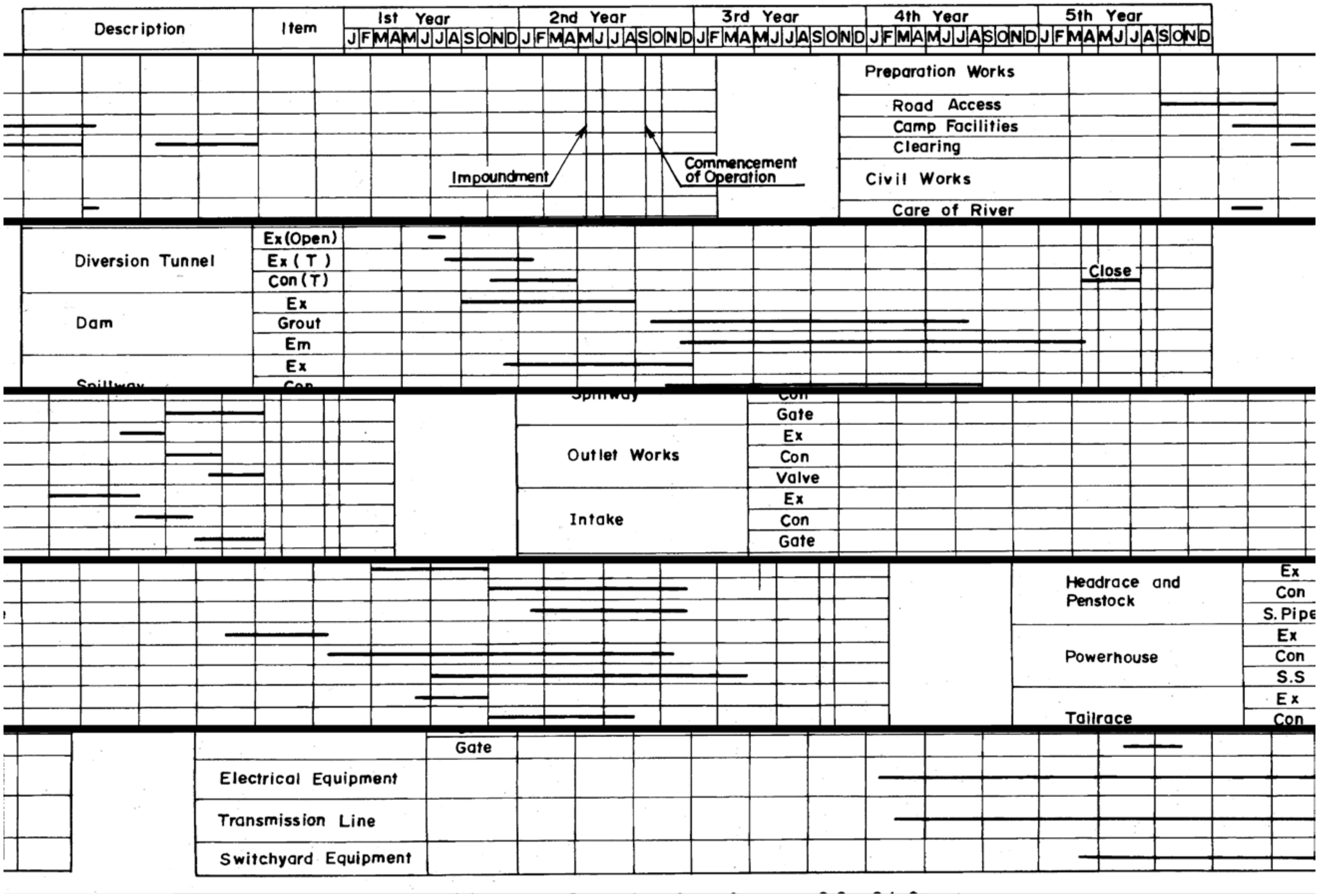
The powerhouse construction works are preceded by the foundation excavation, foundation concrete placing and other civil works. The draft tube installation work, constituting a part of the electric works, is then executed. Thereafter, installation of the electrical equipment and the concrete placing for the civil works of the power plant will proceed in parallel. Therefore, for the installation works of the electrical equipment, the overall schedule up to the start of operation must be established on the basis of the design and fabrication time required by the electrical equipment manufacturers, their transportation time, the installation work time, which are executed in parallel to the civil works, and the test operation time such as dry and wet tests.

All the works relating to the water conveyance components such as the dam, headrace, surge tank/head tank and the penstocks, must be completed in time for the wet test.

(3) Overall construction work schedule

The overall construction work schedule including the preparatory works, civil works, hydraulic equipment installation works, electrical equipment installation works, and power transmission and distribution line works must be prepared in the form of bar chart. The example is shown in Figure 14-1

Table 14-1 Example of Construction Schedule



14.3 Construction Cost

14.3.1 General

The construction cost estimated in the feasibility study as described herein, is the total of the expenses for the following items, and termed the "total construction cost" or "project cost" or "total amount of investment".

i) Preparation works

ii) Land and compensation cost

iii) Environmental mitigation cost

iv) Civil works

v) Hydraulic equipment

vi) Electro-mechanical equipment

vii) Transmission and distribution lines

viii) Administration costs

ix) Engineering service cost

x) Taxes

xi) Physical contingency

xii) Price contingency (contingency for price escalation)

xiii) Interest during construction

Construction costs are generally broken down into local currency and foreign currencies components, and are allocated to the respective years on the basis of the construction schedule to prepare the disbursement schedule. The total of 0 through ヨ, which are free from unknown factors, is sometimes called the base cost.

14.3.2 Basic Conditions for Construction Cost Estimate

(1) Time of cost estimation

In estimating the construction cost, the base year of the estimate must be clearly defined, e.g., 1994, first half of 1994, and August, 1994.

The reason for this is that the currency exchange rate, the price levels for labor, material, machinery and other cost items, interest rates etc., change depending on the time of estimation.

For the construction cost calculated in a feasibility study, the time when the estimate is made must be stated.

(2) Local currency and foreign currencies components of construction cost

The construction costs are generally estimated separately for local currency and for foreign currencies. The costs of labor, material, etc., which locally procurable, are included in the local currency. Items procured from foreign countries are included in foreign currencies. In general, the following divisions are employed.

Table 14-2 Local Currency and Foreign Currency of Construction Costs

		Local currency	Foreign currency
i)	Preparatory works	○	—
ii)	Costs for land and compensation	○	—
iii)	Environmental mitigation cost	○	○
vi)	Civil works	○	○
v)	Hydraulic equipment	○	○
vi)	Electro-mechanical equipment	○	○
vii)	Transmission and distribution lines	○	○
viii)	Administration cost	○	—
ix)	Engineering service cost	○	○
x)	Taxes	○	—
xi)	Physical contingency	○	○
xii)	Price contingency	○	○
xiii)	Interest during construction	○	○

14.3.3 Breakdown of Construction Cost

(1) Preparatory works

The preparatory works generally include the following:

- Rerouting of roads, construction of access roads, etc.
- Repair of existing roads, existing bridges, etc.
- Site offices and accommodations for the owner and consultant
- Various investigation works

(2) Costs for land acquisition and compensation

The land and compensation costs include the following, and are generally included in local currency.

- Costs for lands to be occupied by permanent structures
- Cost or rent for lands used by temporary construction facilities, and the resettlement cost for people living in area to be inundated
- Business compensation cost for the residents in or around the construction work sites
- Compensation costs for fishery during the construction period
- Compensation costs for trees felled within the construction work areas
- Other costs

The above items should be separately estimated. However, these costs may be calculated on the basis of past experience of similar project, that is, by multiplying a certain percentage to the total of preparatory works, civil works and transmission and distribution line works.

(3) Civil works

The cost for the civil works is calculated from the quantity of works of civil structures estimated from design drawings and unit prices. The unit price of each work item is broken down into labor costs, material cost, equipment costs, etc. on the basis of the quantity of works, work conditions, construction plan, construction schedules, etc., on the premise that works are to be awarded by international competitive tenders.

The unit prices of materials and construction equipment which can be procured locally, provided that the quality and quantity are acceptable and available, are calculated in the local currency.

Those materials and construction equipment, which are not locally available and which must be imported, are calculated in foreign currency.

The local currency is the currency of the country where the project is located. Foreign currency may be calculated in various currencies, but in the feasibility study stage, however, these are all generally denominated in one monetary unit (e.g., US $).

(4) Hydraulic equipment, electro-mechanical equipment and transmission and distribution lines

The following are included in the estimation of the construction costs for hydraulic equipment, electrical equipment, and transmission and distribution lines.

i) Hydraulic equipment: gates, screens, penstocks, etc.

ii) Electro-mechanical equipment: turbines, generators, control systems, transformers, switchyard equipment and other auxiliary equipment.

iii) Transmission and distribution lines: transmission and distribution line towers (incl. foundation works), overhead lines, etc.

In estimating the costs for the above, international competitive tendering system is assumed as in the case of the civil works. The costs for the above equipment include design, fabrication, ocean transportation, local transportation and installation work.

The costs are estimated separately for local currency and foreign currency components, as in the case of civil work costs. Included in the local cost component are the locally procurable items such as the material, equipment, insurance, local transportation, and labor cost for installation workers.

In the foreign currency component are the costs of design and fabrication of equipment in foreign countries, ocean transportation costs and the labor costs of foreigners engaged in the installation. The costs of foreign-made equipment are estimated on the basis of past contract records for similar or equivalent equipment, or estimated by specialists such as consultants.

(5) Administration cost

The administration cost includes the expenses that the power authority, the power utilities or other owners implementing the project may incur during the preparatory period before construction, and the expenses of the engineers and the staff of the owner stationed in the site office for the supervision of the works. The administration cost is included in the local currency component.

(6) Engineering service cost

The engineering service cost includes the costs for the definite design work, for the preparation of bidding documents, and for the construction supervision performed by the consultants.

Generally, both foreign and local consultants are employed. Therefore their respective personnel costs, overhead costs, direct costs, etc. are separated into foreign and local currencies.

(7) Taxes

Taxes include import duties and value-added taxes. Import duty is imposed on imported construction equipment, materials, equipment, etc., and is generally included in local currency.

This item of cost need not be included if it is exempted for special reason.

(8) Physical contingency

The physical contingency is for possible increase in quantity of work arising during the execution of the project, against the quantities of work that were estimated on the basis of the topographic map and geologic survey in the feasibility study.

Generally, as a physical contingency, the amounts are calculated for the following items by applying a certain factors based on past records.

- Preparatory works
- Civil works
- Hydraulic equipment
- Electro-mechanical equipment
- Transmission line

(9) Price contingency (Contingency for price escalation)

The contingency for possible price increase is calculated on the basis of the domestic price increase index for local currency items, and of the price increase indices published by the authorities of respective countries for foreign currency items.

(10) Interest during construction

Interest during construction is computed based on the estimated annual fund requirement and a given interest rate. It is the accrued annual interest of actual investment on the project.

Domestic rates are used for the interest on local currency, and the interest rates of international financial institutions for foreign currencies. It should be noted that there can be cases where interest on local currency are exempted.

Interest during the construction is generally calculated at simple interest on the estimated annual disbursement schedule.

14.3.4 Disbursement Schedule

Disbursement schedule of annual expenditure is made on the basis of the construction schedule.

The expenditure is calculated for every work item, and for foreign currency and local currency as explained in 14.3.2.

Reference of Chapter 14

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Chapter 15

Environmental and Social Considerations

Chapter 15 Environmental and Social Considerations

15.1 Environmental Impact Caused by Hydropower Development

An environmental impact caused by hydropower development can be classified into physical, ecological, or social. Moreover, when discussing an environmental impact in Environmental Impact Assessment studies, it is necessary to deal with not only a negative impact but a positive one as well. The following chapters describe in detail such aspects of an environmental impact.

15.1.1 Physical Impact

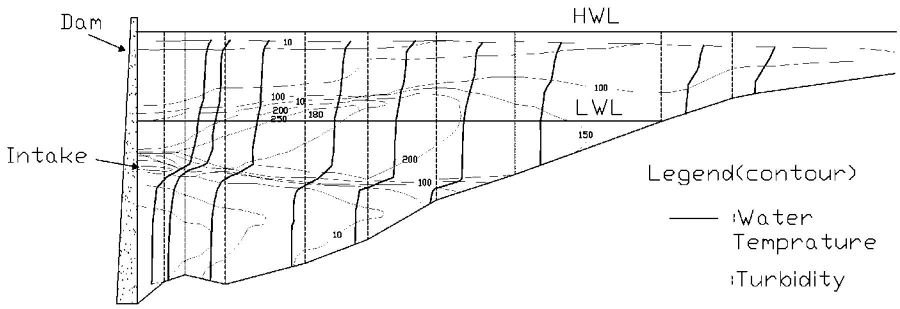
Construction for hydropower development causes changes in topography, hydrology and climate. The following are the characteristics of these changes.

(1) Change in Downstream Water Volume

The reservoir type and pondage type store river flow during rainy season and release supplemental flow during dry season. However, both types change natural water flow downstream of the power stations due to peak power generation. The changing of water flow in a day would be heavy if there were no re-regulating reservoir just below the power station.

(2) Change in Water Temperature and Water Quality

Water temperature and water quality is shown in Figure 15-1. The water temperature of the upper part of the reservoir increases while that of the lower part stays low in the Reservoir type. Then the water temperature from the outlet might differ from the natural one. The bottom water of the reservoir might become hypoxia because of oxygen consumption by decomposition of organic sedimentation. On the other hand, blue-green algae might emerge at the surface water of the reservoir because of overgrowth of phytoplankton.



Source: J-Power

Figure 15-1 Distribution of Water Temperature in the Reservoir

(3) Flow of Turbid Water

Turbidity in the reservoir might remain long in the Reservoir type. The cause of turbidity is the floating of soil particles that hardly decompose. If soil particles are concentrated, most rocks in the downstream of a power plant look as if they are covered by mud.

One of an effective method to remove turbid water is to lead the water to a certain layer (intake level) by utilizing the phenomenon that turbid water flows into the height of a layer of discontinuity of water temperature.

(4) Appearance of Recession Area

As shown in Figure 15-2, the quantity of river flow between the dam site and the outlet decreases because of the water intake for power generation in both the dam and conduit type power generation and the conduit type power generation. This area with decreased water is called a recession area and the discharged water for maintaining the river in the recession area is called a river maintenance flow. The recession rate can be controlled by an effluent from the outlet facility at the dam site.

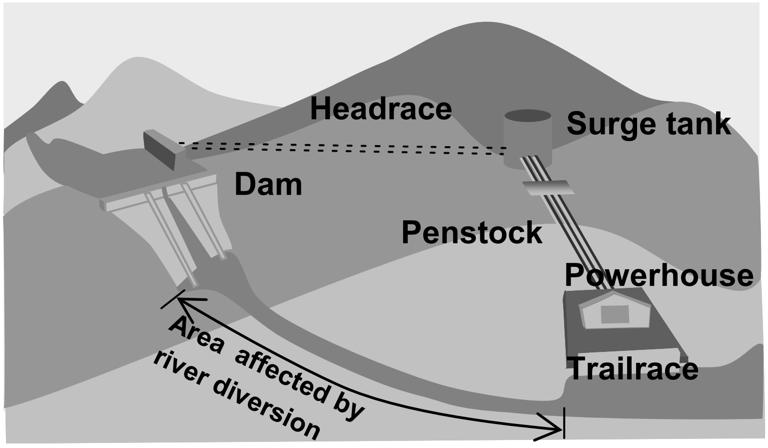


Figure 15-2 Recession Area

(5) Change in Underground Water

The underground water level around reservoirs rises when the water is impounded in the reservoir. The underground water level around a headrace tunnel might decrease because of water discharge from the headrace tunnel if the tunnel goes through an area rich in underground water.

(6) Sedimentation

Sand sedimentation in the reservoir or regulating pond will happen in the Reservoir type or Regulating-Pond type as shown in Figure 15-3. Decreasing storage capacity caused by sedimentation depresses the power control system or electricity output. The mechanism and characteristics of sedimentation is mentioned in 3.3. River bed degradation and coastal erosion might happen because sedimentation prevents sand and soil flow to the downstream. River bed aggradation might happen at the upper part of a reservoir or regulating pond because of lower flow velocity. Flooding might occur if the river bank is not high enough.

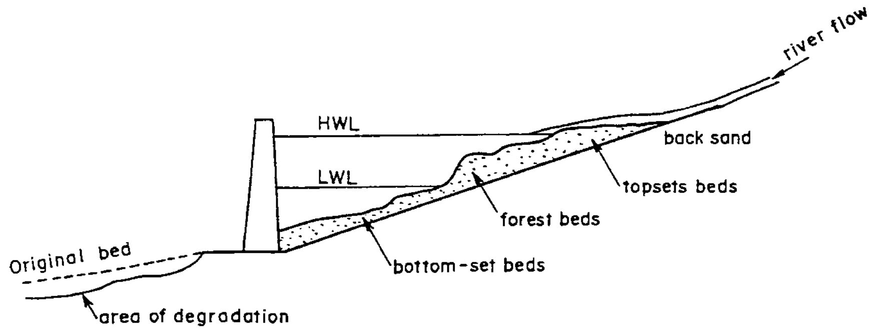


Figure 15-3 Impact by Sedimentation and Sedimentation Style

(7) Downstream River Bed Degradation and Coast Erosion

River bed degradation and river bank erosion might happen in the area downstream of the dam because of stopping soil and sand supply caused by sedimentation (see Figure 15-3). In addition to the river bed degradation, granularity composition is also changed because of the characteristics of the water flow which is likely to transport the same size of soil particle. Decreasing soil and sand supply and changing flow rate might cause coastal erosion or salt water intrusion at the river mouth.

(8) Topographic Change

Mountains and river banks are excavated for collection of aggregate and fill for dam construction. Meanwhile, a damping site is prepared for the excavated material of a tailrace tunnel or power plant. These actions cause topographic changes.

(9) Micro Climate Change

The evaporation source before the construction is a river; the source after the construction might be a huge reservoir. Such a drastic change in the evaporation system might cause micro climate change around the reservoir.

(10) Greenhouse Gas

Greenhouse gas might be generated by reservoirs if organic sedimentation in the reservoir or regulating pond is decomposed. The discharge is likely to be high in a high-temperature tropical region.

15.1.2 Natural Environment

(1) Emergence of New Ecosystem

A stagnation water ecosystem will emerge after the completion of a reservoir or regulating pond. Fish and birds that prefer the stagnation water will increase.

(2) Habitat Loss Caused by Inundation

The terrestrial habitat in the inundation area is submerged. The entire ecosystem will be lost with its wildlife habitat if the area is a wildlife concentration area. The plant community along the river depends on dynamic river topography and hydrology and it supports a unique ecosystem along the river. It is practically impossible to mitigate the loss by transplantation or re-promotion.

(3) Degradation of Aquatic Biodiversity

Aquatic gene biodiversity will decrease because of reduced opportunity of genetic interaction caused by dam as migration barrier. Migratory fishes which move between upstream and downstream areas and sea could suffer particularly serious damage due to blockage on migration and reproduction. The aquatic species which depend on river dynamics including flooding also lose their habitat because of equalization of the flow rate in the downstream area.

(4) Change in Ecosystem That Depends on River Dynamics

A change in downstream hydrology by a dam or barrage may affect the entire downstream ecosystem. Many bank vegetations keep their community by repeated flooding disturbances. Thus decreased flooding and stable hydrology cause the loss of the natural habitat of bank vegetations. Decrease in plant diversity causes degradation of fauna diversity and the river ecosystem. A project which has watershed modification might cause a particularly serious impact on a downstream ecosystem.

(5) Migration Barrier on Ground Cursorial Species

A reservoir and a regulating pond might be a migration barrier for ground cursorial species. Species that cannot fly lose genetic interaction opportunity and their genetic biodiversity may decrease.

(6) Poaching and Illegal Logging

If the hydropower plant site is a deep forest where no one has had access so far, construction makes access to the forest easier. If the usage of a road to the forest is not controlled or poorly controlled, poaching and illegal logging will increase.

(7) Invasion Species

Many vehicles enter the construction site in association with carrying in and taking out materials. Alien plant seeds and insects might be carried into the project area by the vehicles. If the species are those with very high viability, the indigenous ecosystem might be disturbed.

15.1.3 Social Environment

(1) Positive Impact by Projects

- Reduction of flood damage: A reservoir that has flood control could reduce the flooding damage in the area downstream of the dam.

- Expansion of available land use area: The available land use area will expand because a dam will decrease a possible flooding area.
- Increase in job opportunities: The local economy will improve temporally because of the job opportunities through dam construction. Employment will also remain for a while because the dam facilities need maintenance.
- Emergence of a new industry: The reservoir or regulating pond might become a new fishery area or tourism site.
- Improvement of living conditions: If the project area had no electricity and limited public infrastructure, the hydroelectricity project will improve living conditions including road, water supply, electricity, health facilities, community center, and schools as a part of compensation for resettlement.

(2) Resettlement

People who live in the planned reservoir or regulating pond area have to move to the outside of the project area. The number of resettlements would be high if the planned reservoir or regulating pond covered an area with many houses.

(3) Loss of Agricultural Land

Farmland will be submerged if the planned reservoir or regulating pond comprises farmland. Then the total agricultural area will decrease.

(4) Infectious Disease

If there are any indigenous people who have very limited contact with urban inhabitants in or around the project area, they might contract infectious diseases because of lack of immunity. Even if their habitat is not affected, mere contact with urban people might make them ill. HIV/AIDS might also spread because it is usually brought by workers from distant areas.

(5) Impact on Fisheries

Fisheries upstream and downstream might be affected by projects. Construction of dams changes not only river shape but also flow rate, flow velocity, water temperature, water quality, bottom material and river side vegetation. As the changes are big, so is the impact.

(6) Impact on Tourism

Tourism around the project area might be affected. Possible tourist attractions and activities include scenic site, nature observation, mountain climbing, rafting, and sports fishing.

(7) Impact on Transport

The local transport system might be affected if the project crossed or cut the existing road. Risks of traffic jam and accidents might increase if the existing road were used for the transport route of construction vehicles.

(8) Submergence and Degradation of Cultural Assets or Fossil Remains

If the reservoir or regulating pond is huge, unknown cultural assets or fossil remains might be submerged without being recorded. Buried cultural property cannot be excavated after flooding. Degradation of fossil remains might also accelerate after submerge.

(9) Accident of Discharged Water

If the hydropower plant responds to peak electricity demand, a great deal of water is discharged accordingly. If the discharge time is not notified to the downstream area, a water accident might happen by swollen river.

(10) Loss of Intangible Cultural Assets

If an entire village is submerged by a reservoir or regulating pond, intangible cultural assets such as traditional techniques, culture and language might be lost.

15.2 Basic Concept of Handling Environmental and Social Issues[1]

There are various ways for cope with environmental and social potential problems. But if the project team misses adequate timing for an adequate survey and fails to reflect the survey result in the project design, mitigation measures will be ineffective and inefficient. To mitigate the impact effectively and efficiently, it is necessary to understand and implement the following three basic concepts: (1) Sequencing; (2) No Net Loss; and (3) Tiered Approach.

15.2.1 Sequencing

Mitigation measures are classified into Avoid, Minimize and Compensate. The most ideal mitigation is Avoidance, the second best is Minimization, and the last resort is Compensation. When it comes to cost effectiveness, Avoidance is the best, Minimization is the second best, and Compensation is the worst choice that is highly unlikely to produce high effectiveness. Avoidance can be implemented only if it is considered before site selection. If the project team misses the chance and be late in starting environmental consideration, it will have no choice but Compensation.

15.2.2 No Net Loss

No Net Loss and/or Environmental Offset are a newly introduced concept for sustainable development. No Net Loss means that one can keep development work going without losing the natural environment if the same value as the environmentally loss caused by the project is created by the project. For example, if a 100-ha forest will be lost by the project, the project has to create the forest of the same size and same value outside of the area. The newly created forest is called an

[1] Environmental and Social Consideration: Environmental and Social Consideration includes consideration on not only for the physical and ecological environment but also social matters such as resettlement and human rights of indigenous people.

Environmental Offset. It is not appropriate to call the newly created forest an Offset when the quality of the forest is not the same as the destroyed one. It is also not proper to call it an Offset when the forest creation destroys another ecosystem. One can call it an Offset when the environmental value of the compensation area is increased. For example, recovering work at abandoned mining or abandoned farmland can be called an Offset. Recently many projects embrace the No Net Loss concept and consider adapting an Environmental Offset.

15.2.3 Tiered Approach

When it comes to a project life cycle, environmental and social consideration should not be temporary or a one-time matter. Continuous consideration throughout the project cycle is the most efficient and effective means. The project cycle can be divided into the Conception stage, Master Plan stage (M/P), Feasibility Study stage (F/S), Detail Design stage (D/D), Construction stage, and Operation stage as shown in Figure 15-4. Strategic Environmental Assessment (SEA) is applied to the Conception stage and M/P. Environmental Impact Assessment (EIA) is applied to F/S. Resettlement Action Plan (RAP) and Environmental Management Plan (EMP) are applied to D/D. Mitigation and Environmental Management are applied to the Construction stage and the Operation stage. This tiered approach minimizes the environmental cost and maximizes the cost performance.

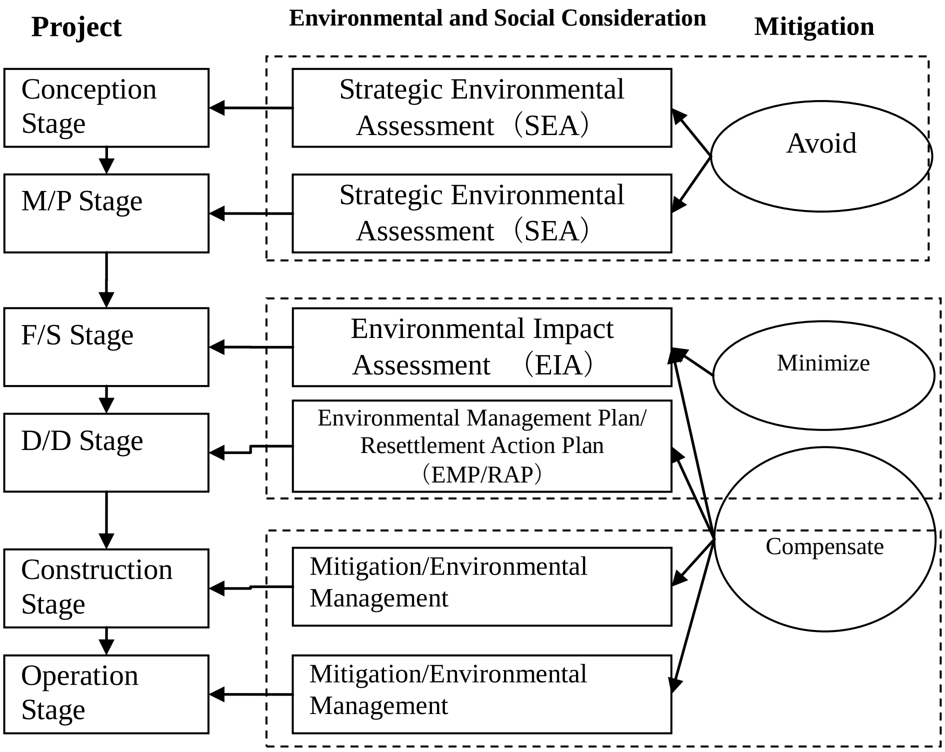


Figure 15-4 Project Cycle and Applied Environmental Consideration

15.3 How to Address Environmental and Social Problems

To minimize the possible negative impact of projects, developers have to consider the environment not only at the F/S stage but also at the all the stages of the project such as the M/P stage, D/D stage, Construction stage and Operation stage. The following are the recommended measures for each stage.

15.3.1 M/P and Prior to M/P Stage

All the stages from the Conception to the M/P stage cover a long period from the stage of project type identification (including WASP calculation) to just before the F/S stage. There have been few experiences of environmental and social consideration in this stage. However, EIAs alone in the F/S stage have not achieved enough minimization of environmental impact so far. These experiences raised awareness on the importance of environmental consideration in an early stage. This early stage environmental consideration is called Strategic Environmental Assessment (SEA).

(1) Basic Concept of SEA

Here is a definition of SEA: “The formalized, systematic and comprehensive process of evaluating the environmental impacts of a policy, plan or program and its alternatives, including the preparation of a written report on the findings of that evaluation, and using the findings in publicly accountable decision-making”[2]. SEA aims to achieve the effective environmental consideration by evaluation of alternatives before concrete design and reflection of the evaluation result in decision making. Different from EIA which evaluates impacts on project passively, SEA is an active planning procedure cooperating with the planners. To make SEA effective, not only the environmental specialist but also planners and decision makers must understand the basic concept of SEA.

(2) SEA Procedure

SEA does not have a fixed procedure, because many countries do not have SEA regulations and the targets cover a wide range from policy to program. The minimum procedures of SEA should include alternatives, scoping, survey, and impact assessment. In the M/P stage or pre-M/P stage of hydroelectric development, the procedure shown in Figure 15-5 would be desirable.

[2] Thérivel et al., 1992.

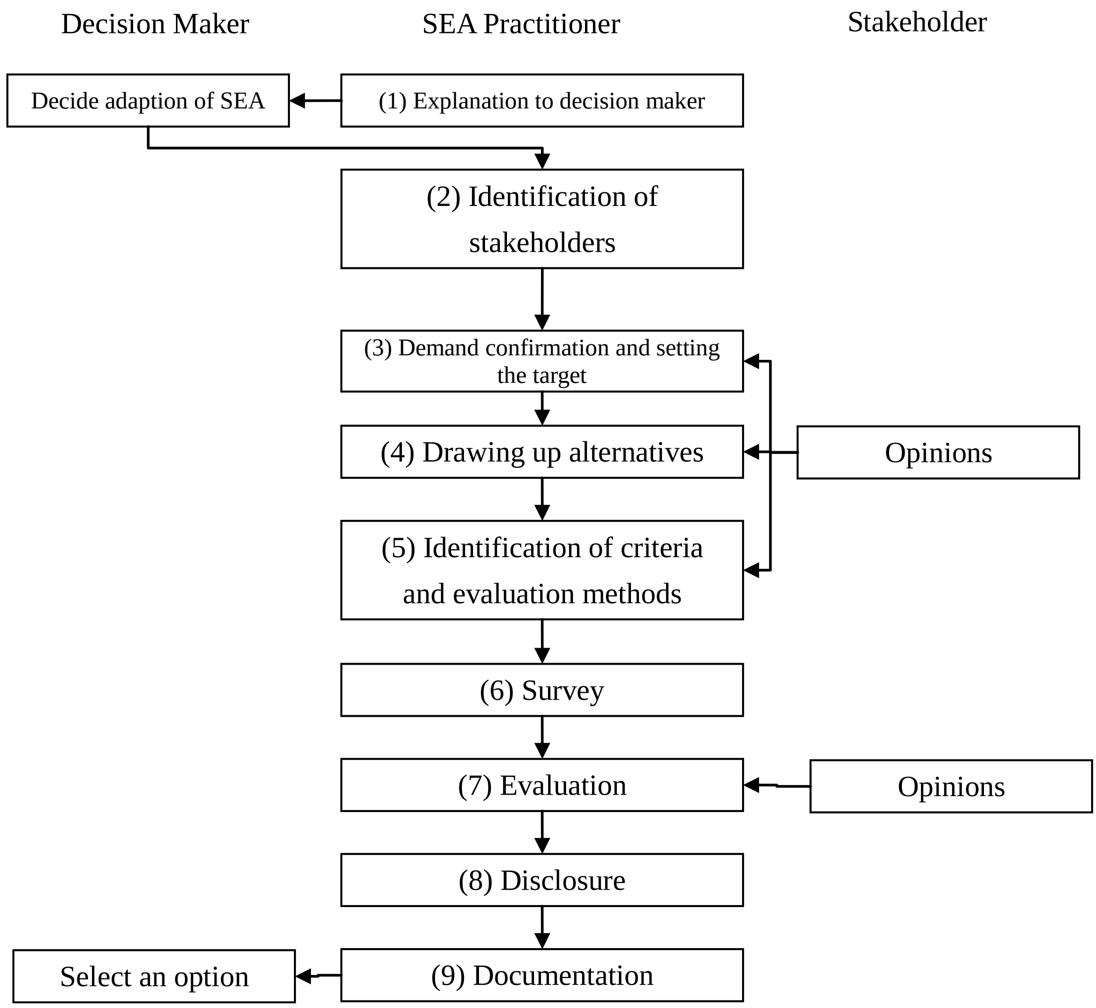


Figure 15-5 Example of SEA Procedure

1) Explanation to decision maker

To use SEA effectively, all the stakeholders including decision makers must understand the concept of SEA. All the stakeholders must share and understand the following: for what SEA has to be done; what the specific contents of SEA are; who the stakeholders are; and what can be gained from SEA. SEA could also be a useful tool for proponents. If the proponent can avoid a serious environmental impact and obtain a consensus on the project, the project will proceed smoothly and the environmental cost will be reduced.

2) Identification of stakeholders

The next step in SEA is identification of stakeholders. Here are its details.

- List all the possible relevant parties such as representatives of the possibly affected community, authorities that give permission, academic experts, related NGOs, and donor agencies.
- Ask the parties above if they can take part in SEA meetings. If possible, delegates of minorities that may be negatively affected by the project and advocates for wildlife should

be included.

3) Demand confirmation and setting the target

The next step in SEA is demand confirmation and setting the target. The demand should be realistic. If the demand is excessive, the necessity for the project will be called into question. The targets should include not only those regarding electricity supply, such as how much electricity is needed, where it is needed, and until when it is needed, but also minimum requirements of ecological and social environment. For example, the conditions might be “No impact on natural protected area”, “Never let endangered species go extinct”, “No resettlement over 10,000”, or “No reduction of tourism resources”. If possible, a consensus on the demand and target should be reached through discussions among stakeholders.

4) Drawing up alternatives

More than two alternatives should be examined in SEA. Alternatives differ depending on which stage the project is at. Discussion for drawing up alternatives among the stakeholders is desirable. One of the alternatives might be no action, which shows the necessity of the project.

5) Identification of evaluation criteria and evaluation method

Unlike EIA, SEA identifies a new set of evaluation criteria for each project. Evaluation criteria must contain all the items to be considered during comparison of alternatives. It is recommended that the ratios of the criteria of economic, environmental and social items be even. The possible criteria should be evaluated from measurability, independence, and relations to the decision making points of view, and should be narrowed down 6 to 20. If possible, these items should be discussed in the stakeholder meeting and made public.

6) Survey

Conduct the survey necessary for evaluation items identified in the preceding step. Generally, a SEA survey is not as detailed as an EIA one. A site survey for SEA would be simpler and cover a broader area than EIA. Sometimes only a short-term literature survey is done for SEA. One of the merits of SEA is that a survey can be done easily and in a short time. If site survey data will be a key issue for discussion, it is perfectly acceptable to conduct a detailed and long-term survey as in EIA. If the survey result helps to change the design and avoid a serious impact, it might be concluded in EIA screening that either no EIA is necessary or Initial Environmental Examination (IEE) is necessary.

7) Evaluation

Based on the survey result, all criteria are evaluated for all alternatives. The evaluation method selected in the scoping step will be used. Some methods evaluate quantity and others evaluate quality. Evaluation items include those in the social and technical aspect (e.g., maximum power, construction cost, construction duration, and funding progress), ecological aspect (e.g., impact on protected area, and impact on wildlife), and social aspect (e.g., number of resettlements, and impact on cultural assets).

8) Public participation and information disclosure

SEA should not be discussed behind closed doors. Public participation and information disclosure are required. They make it easier to have the public understand the necessity of the project and project site. In addition, incorporating the public opinion in the project design makes it easier to have the people accept the project. If possible, two stakeholder meetings should be held at the stages of both scoping and evaluation. The discussion results and the decision made should be made public along with the SEA report.

9) Documentation and decision making

The result of SEA should be documented in an easy-to-understand manner for decision makers, incorporating opinions of stakeholders. The report should contain details of the alternatives, reason for the selection of the criteria, and evidence for the evaluation.

(3) JICA's Conditions on SEA

JAPAN INTERNATIONAL COOPERATION AGENCY GUIDELINES FOR ENVIRONMENTAL AND SOCIAL CONSIDERATIONS (JICA 2010) (hereafter the "JICA Guideline") requires environmental and social consideration from the M/P stage. Below are the categorization and procedure of the JICA Guideline.

1) Categorization of M/P

M/P studies conducted by JICA are classified into four categories: A, B, C, and FI. Table 15-1 shows the definitions of the categories. JICA will categorize projects based on the screening format (Appendix A-15-1) filled by project proponents.

Table 15-1 Project Categorization of JICA[3]

Category A: Proposed projects are classified as Category A if they are likely to have significant adverse impacts on the environment and society. Projects with complicated or unprecedented impacts that are difficult to assess, or projects with a wide range of impacts or irreversible impacts, are also classified as Category A. These impacts may affect an area broader than the sites or facilities subject to physical construction. Category A, in principle, includes projects in sensitive sectors, projects that have characteristics that are liable to cause adverse environmental impacts, and projects located in or near sensitive areas.
Category B: Proposed projects are classified as Category B if their potential adverse impacts on the environment and society are less adverse than those of Category A projects. Generally, they are site-specific; few if any are irreversible; and in most cases, normal mitigation measures can be designed more readily.
Category C: Proposed projects are classified as Category C if they are likely to have minimal or little adverse impact on the environment and society.
Category FI: Proposed projects are classified as Category FI if they satisfy all of the following requirements: JICA's funding of projects is provided to a financial intermediary or executing agency; the selection and appraisal of the sub-projects is substantially undertaken by such an institution only after JICA's approval of the funding, so that the sub-projects cannot be specified prior to JICA's approval of funding (or project appraisal); and those sub-projects are expected to have a potential impact on the environment.

2) Required procedure of M/P

The JICA Guideline requires different procedures by study type and by category. A Category A Preparatory Survey has to disclose TOR/SEA and the SEA report, but a Category B Preparatory Survey can do so if necessary (see Table 15-1). Table 15-2 shows the needed procedure of M/P by category.

Table 15-2 JICA-Required Procedure of M/P Stage

Survey Type	Preparatory Survey			Technical Cooperation for Development Planning		
Category	A	B	C	A	B	C
Type of environmental considerations	SEA	SEA	No need	SEA	SEA	No need
Preparation of draft TOR/EIA	Needed	Needed	No need	Needed	Needed	No need
Information disclosure of draft TOR/EIA and SHM	Needed	If necessary	No need	Needed	If necessary	No need
Survey	IEE level	IEE level	No need	IEE level	IEE level	No need
Information disclosure and SHM* when considering the rough outline	If necessary	If necessary	No need	If necessary	If necessary	No need
Information disclosure of draft report and SHM	Needed	If necessary	No need	Needed	If necessary	No need

*SHM: Stakeholder Meeting

[3] JICA GUIDELINES FOR ENVIRONMENTAL AND SOCIAL CONSIDERATIONS (http://www.jica.go.jp/environment/guideline/ref.html)

(4) Specific Environmental Measures at M/P Stage and Prior to M/P Stage

1) Demand-side management option

The demand-side management option can be considered at the M/P stage and prior to the M/P stage. Demand-side management seeks the possibility of not implementing the project by lowering the demand in planning the alternatives. Demand-side management includes reduction of demand by the user's effort, reduction of new demand by recycling, reduction of supply loss by technology, and accelerating the incentive for reducing demand by consolidating the legal system.

2) Greenhouse gas

Effective measures for greenhouse gas can be considered only before selection of generation type. For example, comparison of the total amount of greenhouse gas emission for future decades on different electric power development scenarios can be considered. The comparison of CO_2 emission from reservoirs is also possible, but it cannot be a key factor because of many other important items.

3) Site selection for impact avoidance

"Avoiding a place with high impact possibilities for the project site" can be taken only before site selection. If this measure is taken, it will not be necessary to be concerned about environmental problems in the area afterward. In terms of sedimentation, the project site should not be a place with frequent landslides in the upstream area, a large deforestation area, or an area with sedimentation-prone soil texture. The cost for the sedimentation survey is the lowest among mitigation measures for sedimentation.

4) Ecological environment measures

The best practice of ecological mitigation at the M/P stage and before is Avoidance such as "Avoidance of locating the project in an ecological important area." If avoidance measures are fully implemented, subsequent ecological environment measures will be very smooth. The problem at this stage is whether or not the project proponent can obtain the correct information of the most ecological important areas. Known ecologically important areas include such conservation areas as national park, Ramsar site, Important Bird Area (IBA), habitat of Red List species identified by the International Union for Conservation of Nature and Natural Resources (IUCN). Some information can be obtained on the Internet (see the table below) but that is far from sufficient. It is particularly difficult to fully grasp the distribution of IUCN Red List species. That is one of the reasons that projects without SEA may have to be terminated by discovery of an unknown habitat of important species at the F/S stage. To avoid such a situation, it is advisable to conduct an ecological site survey at the M/P stage and avoid a potentially problematic area for the project. The cost of an ecological site survey at the M/P stage is much smaller than the mitigation cost after EIA. It is sensible to conduct an ecological site survey at the M/P stage if sufficient ecological information cannot be obtained.

Table 15-3 Ecological Information Sources Available on the Web

Type	Area	Information site
Conservation Area	National park, etc.	The World Database on Protected Areas (http://www.wdpa.org/)
	Ramsar site	The Ramsar Convention on Wetlands (http://www.ramsar.org)
	Important Bird Area	Bird Life International (http://www.birdlife.org/)
Habitat of the IUCN Red List of Threatened Species		The IUCN Red List of Threatened Species (http://www.iucnredlist.org/)

5) Social environment measures

The best social environment mitigation measure in M/P and before is Avoidance, i.e., "Avoid an area which might receive a serious impact from the project." For example, the project might cause a major resettlement, submergence of a cultural asset or an entire tribal village, destruction of an important tourism source or an important base of agriculture or fishery. To avoid overlooking an important impact, it is advisable to check the actual impact of similar projects and projects in neighboring areas should be checked, because the possible impacts vary depending on the conditions or characteristics of the site.

15.3.2 F/S and D/D Stages

Normally, EIA in the F/S stage is to be conducted in line with the EIA regulations in the country. If the project proponent is supported by donors, the proponent must follow the donors' guidelines as well. EMP and RAP are prepared during D/D after F/S. If the project is relatively small, EIA and EMP might be prepared at the same time. The following are the specific environmental and social considerations in the F/S and D/D stages.

(1) Basic Concept of Project EIA

The basic concept of EIA at the F/S stage is "Assessing possible environmental impacts correctly and proposing adequate mitigation". If EIA is conducted correctly, the project design is changed adequately according to the survey result and prediction of EIA, and an environmentally sound project will become a reality. However, EIA regulations show only procedures. Then, if proponents do not understand the basic concept and purpose of EIA, EIA cannot fulfill its role and becomes just a letter of explanation. The following are characteristics of an ideal EIA according to the International Association for Impact Assessment and the UK Institute of Environmental Assessment (now the Institute of Environmental Management and Assessment).

Table 15-4 What EIA Should Be

Purposive	The process should inform decision-making and result in appropriate levels of environmental protection and community well-being.
Rigorous	The process should apply 'best practicable' science, employing methodologies and techniques appropriate to address the problems being investigated.
Practical	The process should result in information and outputs which assist with problem solving and are acceptable to and able to be implemented by proponents.
Cost-effective	The process should achieve the objectives of EIA within the limits of available information, theme, resources and methodology.
Efficient	The process should impose the minimum cost burdens in terms of time and finance on proponents and participants consistent with accepted requirements and objectives of EIA.
Focused	The process should concentrate on significant environmental effects and key issues, i.e., the matters that need to be taken into account in making decisions.
Adaptive	The process should be adjusted to the realities, issues and circumstances of the proposals under review without compromising the integrity of the process, and be iterative incorporating lessons learned throughout the proposal's cycle.
Participative	The process should provide appropriate opportunities to inform and involve the interested and affected citizens, and their inputs and concerns should be addressed explicitly in the documentation and decision-making.
Interdisciplinary	The process should ensure that the appropriate techniques and experts in the relevant biophysical and socioeconomic disciplines are employed, including use of relevant traditional knowledge.
Credible	The process should be carried out with professionalism, rigor, fairness, objectivity, impartiality and balance, and be subject to independent checks and verification.
Integrated	The process should address the interrelationships of social, economic and biophysical aspects.
Transparent	The process should have clear, easily understood requirements for EIA content; ensure public access to information; identify the factors that are to be taken into account in decision making; and acknowledge limitations and difficulties.
Systematic	The process should result in full consideration of all relevant information on the affected environment, of proposed alternative and their impacts, and the measures necessary to monitor and investigate residual effects.

(2) EIA Procedure

Required EIA procedures vary depending on the country and the organization. Normally, the procedures start with screening, followed by scoping and survey, and ends in reviewing. The main procedures of EIA are explained as follows.

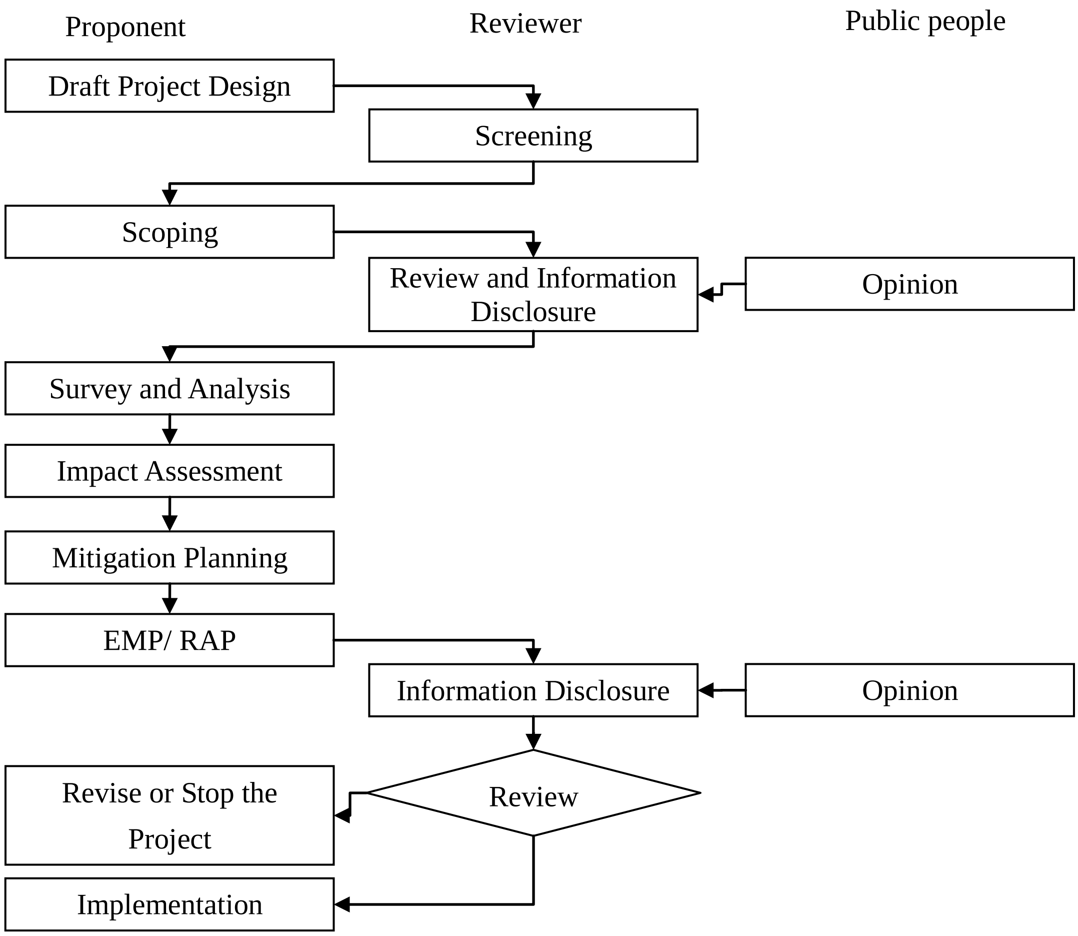


Figure 15-6 Standard Implementation Process of EIA

1) Screening

Screening, the first step in EIA, is the procedure to decide whether the proposed project should conduct EIA. Detailed rules of screening vary depending on the country and the organization. Some rules give only the project types and sizes that need EIA. Other rules have no standard on project type or size and let reviewers decide such matters. The latter usually require project proponents to submit a project brief for screening. Screening results would be “EIA needed”, “no need for EIA”, or “IEE needed”. If the procedure is unclear, contact the agency or ministry responsible for EIA in the country.

2) Scoping

When the project is decided as “EIA needed” or “IEE needed”, the next step is scoping. Scoping identifies relatively large-impact items and decides survey methods, survey time, survey area, and prediction methods. Many EIA regulations require disclosure of the scoping result and holding stakeholder meetings in order to collect their views. The opinions obtained in the meetings should be included in the survey and assessment to the maximum possible extent.

3) Survey and analysis

After scoping comes survey and analysis. Although entrusted survey practitioners will conduct the survey, project proponents also have to check whether the actual survey methods, time, and area are as planned or not, and look into whether or not important conservation species are found. An EIA based on an inadequate survey causes not only an inadequate assessment but also damages the credibility of the proponent. If the data analysis reveals unknown important things, the finding should be reflected in designing.

4) Impact Assessment

After data collection is impact assessment. When describing the assessed impact, to whom, where, when, to what extent, and reversibility must be shown. The impact area should be shown in a map and the impact extent should be presented quantitatively. If it is difficult to forecast an impact, monitoring results from a similar, nearby project could be used as a reference for prediction.

5) Mitigation

Mitigation is the planning of alleviation measures for a negative impact. Ideally, the entire negative values should be mitigated based on the No-Net-Loss concept. To plan implementable mitigation, the experts of all fields, designers, and project proponents should discuss what can be implemented within limited time and fund. Then who, when, how, and where should be identified for the mitigation.

6) Environmental Management Plan

After mitigation comes Environmental Management Plan (EMP), which includes monitoring, evaluation, mitigation, and information disclosure. Implementation structure, responsible person, needed fund should be made clear in EMP.

7) Information disclosure and public participation

EIA regulations usually stipulate the opportunity of information disclosure and public participation. Information disclosure is to be done twice: once for TOR/EIA at scoping, and once more for the draft EIA report after impact assessment. A briefing session for the public might be held at the same time as information disclosure. In information disclosure, the local language should be used in principle, and the contents should be easy to understand.

(3) JICA's Requirements on EIA

The JICA Guidelines (2010)[4] show the procedure and rules in F/S stage. The following are the explanation of the categorization and procedures.

1) Categorization of F/S

F/S studies by JICA are classified into four categories: A, B, C, and FI. The definitions of the categories are the same as M/P (See, Table 1).

[4] JICA website(http://www.jica.go.jp/english/operations/social_environmental/guideline/pdf/guideline100326.pdf)

2) Procedures needed for F/S

The JICA Guidelines require different procedures by study type and category. The Category A projects of Preparatory Survey require information disclosure and SHM at the time of the completion of the draft TOR/EIA and the draft EIA report. The following table shows JICA's requirements on the F/S stage.

Table 15-5 JICA-Required Procedure of F/S Stage

Survey Type	Preparatory Survey			Technical Cooperation for Development Planning		
Category	**A**	**B**	**C**	**A**	**B**	**C**
Type of environmental considerations	EIA	EIA	No need	EIA	EIA	No need
Preparation of draft TOR/EIA	Needed	Needed	No need	Needed	Needed	No need
Information disclosure of draft TOR/EIA and SHM	Needed	If necessary	No need	Needed	If necessary	No need
Survey	EIA level	IEE level	No need	EIA level	IEE level	No need
Information disclosure and SHM* when considering the rough outline	If necessary	If necessary	No need	If necessary	If necessary	No need
Information disclosure of draft report and SHM	Needed	If necessary	No need	Needed	If necessary	No need

*SHM: Stakeholder Meeting

3) Environmental Review before the conclusion of agreement documents

JICA conducts an Environmental Review before the Japanese government decides whether to implement loan aid, grant aid (excluding projects executed through international organizations) and technical cooperation projects. The needed documents for Environmental Review are different by category.

Table 15-6 Needed Documents and Procedure for Environmental Review

Category	A	B	C	FI
EIA reports	Must be submitted to JICA	When an EIA procedure has been conducted, the EIA report may be referred to if needed.	No need	If sub-projects under the cooperation projects are categorized in A, needed documents are same as category A.
Environmental permit certificate	Must be submitted to JICA	When an EIA procedure has been conducted, environmental permit may be referred to if needed.	No need	Same as above
Resettlement Action Plan (RAP)	Needed for projects that will result in large-scale involuntary resettlement	When an RAP has been prepared, the RAP may be referred to if needed.	No need	Same as above
Indigenous People Plan (IPP)	Needed for projects that will require the measures for indigenous people	When an IPP has been prepared, the IPP may be referred to If needed	No need	Same as above
Information disclosure	JICA discloses documents above 120 days prior to concluding agreements.	JICA discloses documents above, if obtained.	Omitted	Same as above
Environmental Review	JICA conducts review using checklist (Appendix A-15-2).	JICA conducts review using checklist (Appendix A-15-2).	Omitted	Same as above

(4) Mitigation in F/S Stage

Environmental measures in the F/S stage incorporate mitigation measures into the project design, after identification of the type, boundary, and extent of impact. The formulation of environmental measures must be conducted together with designers and environmental specialists, because the work is to be done in parallel with designing the project.

1) Mitigation for sedimentation

If sedimentation is predicted in EIA, countermeasures must be incorporated in the plan. Mitigation measures include spilling out facilities, sedimentation dam, and flood bypass tunnel. The sedimentation problem may be alleviated if these measures are implemented together with other dams in the same watershed. The following are the countermeasures.

➢ Spilling out facilities

A spilling out facility flushes out sedimentation in the dam using natural water power. The spilling out gate is usually closed. The gate is open during flood and sedimentation is flushed out (See Figure 11-17, Chapter 11).

➢ Sedimentation dam

A sedimentation dam is a dam constructed in the river upstream of the main dam in order to prevent inflow of soil and sand into the main dam. Sedimentation in the dam is excavated before the dam ceases to function.

- Flood bypass tunnel

A flood bypass tunnel takes sand and muddy water from the river upstream of the dam and flows them downstream of the dam during flooding. The tunnel prevents sedimentation and turbid water in a reservoir and recovers the natural movement of sand and soil (See Figure 11-16, Chapter 11).

2) Water for river maintenance

When water recession is predicted, environmental flow which is water for river maintenance should be examined. The volume of environmental flow cannot be uniformly defined. Some countries and rivers set a rate of environmental flow. If there are no rules for environmental flow, water use, living conditions of the protected species and natural flow should be considered to determine the value. Ten items for water for river maintenance are applied in Japan .

3) Water temperature and turbid water

Selective water intake is one of the countermeasures for the problem of water temperature and turbid water. Selective water intake can take water from any depth of the reservoir. Then it could flow non-turbid and natural temperature water.

The other countermeasures are flood bypass tunnel. It also works for turbid water because it can use the non-turbid water taken upstream.

4) Reduction of generation of greenhouse gas

The main cause of generation of greenhouse gas from a reservoir is organic matter in the water. Then the mitigation of greenhouse gas in the F/S stage is to reduce organic matter in the water. Causative agents of organic matter include plants and soil on the surface of an inundation area, fallen leaves, and floating grass. If there are dams just upstream of the dam, waste inflow might not be a problem. However, if there is a vast residential area and/or forest area, household waste and natural waste might pose problems. Practical mitigation measures would include the following:

- Removal of plants and soil in the inundation area before submerging
- Installation of float garbage collecting facilities near intake
- Installation of garbage capturing facilities upstream of the inflow

5) Mitigation for ecosystem

Impact Assessment for ecosystem should identify current distribution and home range of the affected population at first, and then predict the impact of earthwork and flow fluctuation. When the division of a wildlife habitat is unavoidable, a connecting passage in the form of an animal bridge or animal tunnel should be planned. Design of the passage should match the moving characteristics and sizes of the target animals. A connecting passage cannot compensate the negative impact perfectly and a residual impact will remain.

Note: Residual impact: Real impact that remains after mitigation value is deducted from negative impact caused by the project.

6) Fish pass

A fish pass could alleviate the impact on fish migration. Experts on fish should work together on designing one, because the designing needs consideration on the target species and conditions of the area.

7) Impact Assessment and mitigation for wetland

Impact on wetland depends on the relative location between the dam site and wetland and the extent of water fluctuation. A part of wetland ecosystem can be restored by creation of artificial wetland similar to natural wetland.

Figure 15-7 Example of Fish Pass and Biotope

8) Countermeasures on poaching and illegal logging

The projects might trigger poaching and illegal logging because of easy access to the prime forest. When this might happen, prevention measures must be taken. The measures are classified into physical measures and regulation measures. Physical measures include a road system that prevents free access from the existing roads and installation of a high accuracy camera. Regulation ones include traffic regulation which allows only the registered cars to pass. Preventive measures must take into account the target species and characteristics of the area in cooperation with the wildlife protection authority.

9) Alien invasive species

The species for planting must be carefully selected during the F/S stage in order to prevent alien invasive species from entering the area. Even if the area needs greening fast for preventing erosion, easy selection of foreign species might cause ecosystem disturbance. Domestic species are preferable for planting.

10) Resettlement and revenue compensation plan

If resettlement or land acquisition happens, a compensation plan should be made. Ethnic minorities and non-formal residents should not be excluded in the compensation plan. The plan must be fair. The resettlement of an entire village might require resettlement of such facilities as public hall, clinics, schools and markets.

11) Resettlement program

Resettlement program might include various types of technical training for the resettled people, e.g., agricultural technique training, fishing training, timber process training, and metalworking training. If the resettlement program aims at poverty reduction, microfinance[5] might be included.

12) Precaution for infectious diseases

If the project site is in an area where indigenous people rarely contact outsiders, they might not have any resistance for normal infectious diseases. In such a case, the opportunity of contact between indigenous people and construction people should be restricted. If the expansion of HIV/AIDS is a concern, a HIV/AIDS program must be implemented. Mitigation measures for infectious diseases must include precaution, monitoring, and countermeasures, and cover not only workers but also local residents.

13) Compensation for fisheries

Compensation for fisheries is needed when an impact on them is unavoidable. The compensation cost could be calculated based on balance amount between benefit with the project and without the project. If fisheries will be totally damaged, vocational training or entrepreneurial training could be done in addition to the monetary compensation.

14) Compensation for tourism

If a negative impact on tourism is unavoidable, compensation for tourism is needed. The compensation cost could be calculated based on estimated amount of sales without the project. If there is a possibility of shift to new tourism, business support for the shifting could be provided.

15) Prevention of traffic accident

If accidents by construction cars might happen, truck routes and the current traffic condition should be surveyed and high risk accident areas must be identified. After identification of high risk areas, measures such as bypass route or walkway should be considered.

16) Protection of cultural asset

When some cultural asset will be affected by the project, mitigation measures such as resettlement and protection must be taken. The needed time and fund depend on the characteristics and value of the asset. The protection plan should be designed in cooperation with specialists.

17) Precaution of watering accident

[5] Microfinance: A petty loan for the poor.

Watering accidents may occur when electricity generation starts or a gate opens for flooding. After identification of the high risk time, area, and condition, precaution measures such as counter regulation pond and open gate alarm system must be considered.

18) Cultural compensation program

If traditional language, festival, clothes, or handicraft might be lost, record the cultural matters and prepare the environment for continuing the traditional culture. For example, for the resettlement of an entire village, preservation of cultural places and festival tools, and mitigation of cultural areas for handicraft or needlecraft might be needed.

19) Environmental offset

When a residual impact is large and the project cannot fulfill the principle of No-Net-Loss, the project proponent should seek the possibility of environmental offset. Environmental offset includes supporting environmental recovery works on degraded areas such as an abandoned mine outside of the project, expansion of a conservation area, and upgrading a buffer zone or corridor zone. Planning environmental offset should be done in cooperation with environmental experts, after adequate evaluation of the residual impact. If mitigation banking exists, the proponent can make use of it.

Note: Buffer zone: A green belt around the environmental conservation area to alleviate the impact from a neighboring development area.

Note: Corridor zone: A green belt that connects divided wildlife habitats for alleviating the degradation of biodiversity.

Note: Mitigation banking: A kind of mitigation system which recovers a broad degraded area as mitigation of many projects. The burden shared by each project will be decided based on the No-Net-Loss principle.

15.4 Construction and Operation Stages

Monitoring and follow-up should be done at the Construction and Operation stages. The monitoring points of view are whether or not the environmental impact is within the expected value in the EIA report, and whether or not the mitigation measures are effective. If an unexpected impact is found, the cause of the impact must be identified and new mitigation measures are needed. This manual will not explain monitoring and follow-up in detail, because the target period of the manual is the F/S stage.

15.4.1 Basic Concept of Environmental Management

The basic concept of environmental management is minimizing the actual environmental impact thorough checking and treating the unexpected impact at EIA. Environmental management includes risk management and risk communication. Environmental management plays an important role to prevent a serious impact when the project design is changed or an unexpected accident happens. Environmental management includes monitoring, evaluation of monitoring results, planning and implementing the countermeasures, and reporting and information disclosure.

15.4.2 Procedures of Environmental Management

The components of environmental management are mentioned above. Environmental management is carried out based on the EMP made at the F/S stage.

Chapter 16

Economic and Financial Analyses

Chapter 16 Economic and Financial Analyses

16.1 General

Hydroelectric power development projects require large financial investment. In most cases, the development fund is acquired through Official Development Assistance (ODA) loans from bilateral or governmental and/or multilateral agencies and institutions. The common criteria by which such international agencies and institutions evaluate applications for ODA loans, are that "the subject project meets the policy objectives of the government of the borrowing country, and is positioned correctly within the national economy with no adverse affects upon the nation' s sound growth while producing the satisfactory results expected", and “the project, when completed, will so benefit the enterprise that will run the business in a manner that the enterprise will not default repayment of the loan”.

From this standpoint, the evaluation of the project, for which ODA is being sought, is through economic analysis and financial analysis. Economic analysis involves the evaluation of the socio-economic benefits derived from implementing the project against its cost for its realization on the standpoint of the state and/or the region where the project is located. Financial analysis is an evaluation of the project's economics on the standpoint of the project being a business enterprise with its revenue sources from electric power and water supply, etc. Financial analysis is, therefore, essential for the international financing organizations involved to ensure repayment of the loan.

16.2 Economic Analysis

16.2.1 Outline of Economic Analysis

The outline of economic analysis is presented below:

(1) Indices of economic analysis

The economic analysis of a development project is normally aimed at determining the impact of the project on the overall socio-economic structure of the country, in comparison with the situation where the project is not available in that country.

Economic analysis is measured on the basis of indicators including the net present value (NPV, B−C) which is based on the quantification of the benefit derived from the project and the cost to build or implement the project, the benefit-cost ratio (B/C) and the internal rate of return (IRR).

(2) Economic cost

In the economic analysis, the concept of economic cost is used to evaluate both the benefits and the cost of the project

The market price is generally distorted by various factors based on political interferences including taxes, subsidies, import restrictions, import duties, public utility tariff, minimum wages, and excessive valuation of local currency. It is also distorted by the monopoly systems existing in the market. For this reason, a market price, as it is, should to be converted into an “economic cost” to reflect its true value. For this calculation, the concept of “border price” may be applied. By this method, the goods and services required for the project will be represented by an international price known as border price which is expressed in the basic currency for international trade to separate it from any value-distorting domestic pricing practice. Specifically, to calculate the economic cost of the imported goods for the project and to reflect their real value, domestic transportation cost to the job site and distribution expense are added to the individual CIF price (excluding custom duties and other taxes and charges). The FOB price is used for those goods that can be exported abroad.

The term, CIF, means the import price of goods and services which include insurance and freight charges, whereas FOB means the export price of goods and services that include the cost of inland transportation as far as to the point of shipment (i.e. the border, seaport or airport).

(3) Process of economic analysis

Economic analysis, as generally practiced, goes through the process described below and illustrated in the flow chart in Figure 16-1. In many cases, the study is conducted during Phase-3. The study of Phase-4 depends on the situation of the subject country, and quality of the data made available for the study, etc.

Phase-1: Deduct from the market price the income transferred to the government Transferred items are those regarded as expenses from the project administration’s point of view, but may not be expenses from the standpoint of national income, such as taxes, compensation cost for land acquisition, interest during construction, etc.

Phase-2: Convert the costs determined in Phase-1 into those based on economic cost by specific accounts including trade goods, non-trade goods and labor costs.

Phase-3: Determine the internal rate of return (IRR) based on economic cost and compare this IRR with the opportunity cost of capital of the subject country.

Phase-4: Make a further step to perform a socio-economic assessment, taking into consideration the national saving behavior and income distribution of the subject country.

(4) Economic analysis of an electric power project

If real benefit (power consumption expressed in real value) can be accounted, a method of comparison could be rightly established by calculating all benefits and costs attributed to the project, based on the long-term marginal cost method and the electricity tariff.

In many cases, however, it is difficult to calculate the value of power consumption. Therefore, the evaluation methodology usually adopted is to compare the cost of a hydropower plant with the cost of an alternative thermal power plant which can provide an equal service to that of the hydropower plant. This is on the assumption that the subject project, or any other power project having the same capacity, is needed as part of the socio-economic development program of the country and that, if the subject project is for any reason abandoned, another project to match the subject project would immediately be called for to replace it.

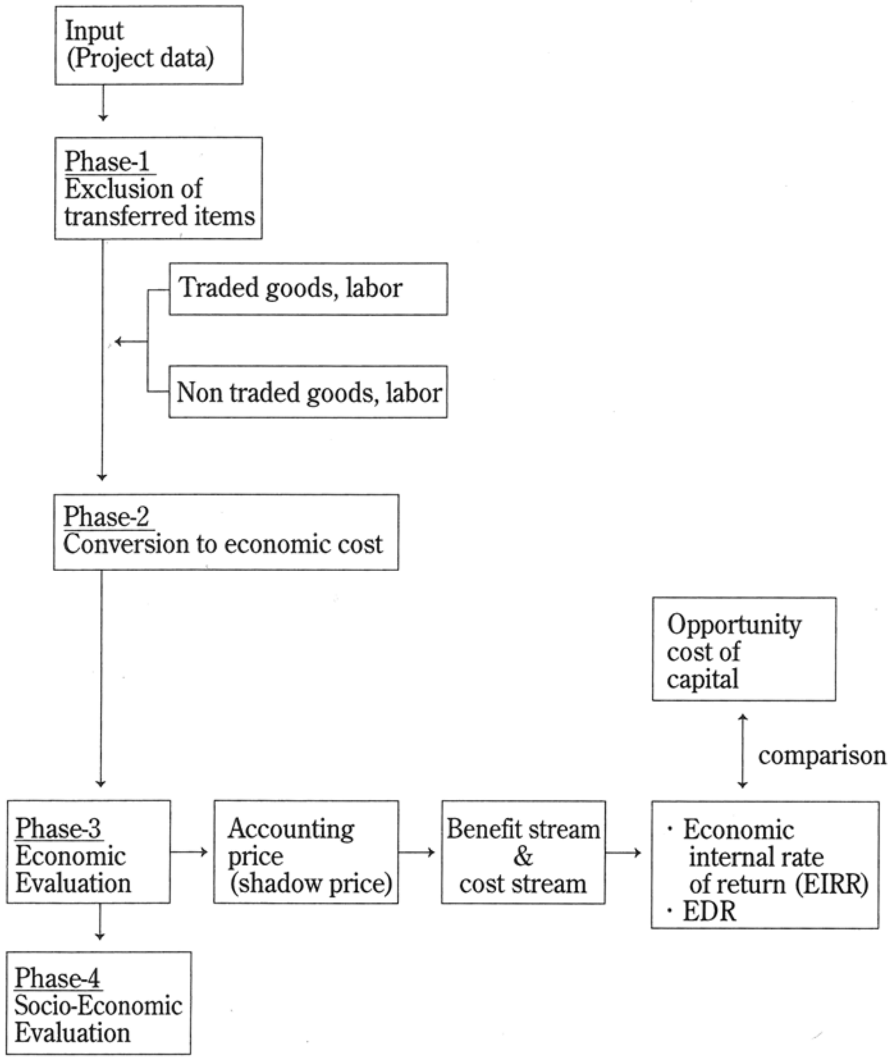


Figure 16-1　Flow Chart of Economic Analysis

16.2.2　Method of Economic Analysis

(1)　Steps of economic analysis

Economic analysis performed in the feasibility study takes the following steps based on the "discounted cash flow" method.

i)　Calculate the costs of the hydropower project (construction cost, running cost, and replacement cost), and prepare an annual investment schedule accordingly (cost stream).

ii)　Calculate the benefit of the hydropower project (cost to build an alternative thermal power plant, running cost, and replacement cost), and prepare an annual investment schedule accordingly (benefit stream).

iii) Convert the figures in the cost stream and benefit stream to their present values.

iv) With the cost and benefit streams expressed in their present values, calculate the net present value (NPV or the present value of B−C), the benefit-cost ratio (B/C) and the internal rate of return (IRR). These three values serve as the economic indicators in the evaluation of the economic feasibility of the project.

(2) Benefit and costs in present value

Cost stream

Figure 16-2 shows the cost streams in both hydropower and thermal power projects, where the construction period of the hydropower project is 4 years and that of the alternative thermal power project is 3 years.

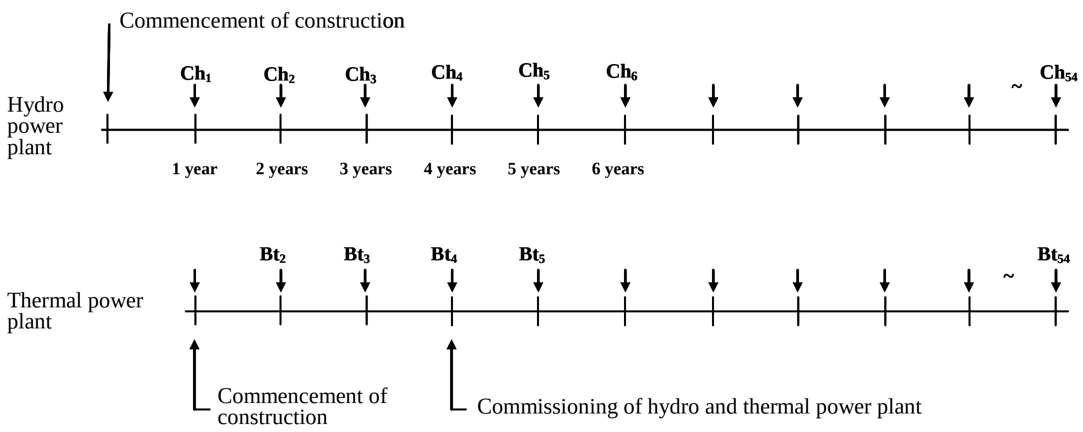


Figure 16-2 Cost Stream

where,

Ch_1- Ch_4 : Construction cost of the hydropower plant

Ch_5- Ch_{54} : Running cost of the hydropower plant (including replacement cost of turbines and generators)

Bt_2- Bt_4 : Construction cost of an alternative thermal power plant

Bt_5- Bt_{54} : Running cost of an alternative thermal power plant (including replacement cost of the thermal power plant)

If the discount rate is i, the annual cost of Ch_k and Bt_k at the year k, converted into present value at the start of construction of the hydropower project is:

$$\frac{Ch_k}{(1+i)^k} \quad \text{and} \quad \frac{Bt_k}{(1+i)^k}$$

1) Economic cost

In economic analysis, “economic cost” is adopted, which is calculated from the “construction cost" described in Chapter 14. In the calculation of economic cost, land acquisition or compensation cost, taxes, contingency for price escalation and interest during construction are deducted from the construction cost. Next, on the local currency portion of the construction cost a conversion factor is applied to calculate "economic cost" on the assumption that the value of the domestic portion expressed as local market price may be distorted by the national policy of the subject country. This value-adjustment practice is called shadow pricing. For the portion of foreign currency, economic cost is the CIF price based on international bidding.

2) Period of economic analysis and service life of project

Generally, the period of economic analysis is the construction period plus the service life of the hydropower project Figure 16-2 shows an example for a construction period of 4 years, an overall service life of 50 years and a period of economic analysis of 54 years. If the service life of the project terminates during the period of the economic analysis, an additional investment will be provided to rebuild or replace the project. If the subject country has its own standard of service lives, they may be used.

(3) Net present value (NPV), benefit-cost ratio (B/C), and internal rate of return (IRR)

These indicators are used in economic evaluation and calculated by the following equations.

1) Net present value (NPV)

$$\mathrm{NPV}=\sum_{k=1}^{n}\frac{B_k - C_k}{(1+i)^k}$$

where,

B_k	: Benefit for year k
C_k	: Cost for year k
I	: Discount rate

2) Benefit cost ratio

$$B/C=\frac{\sum_{k=1}^{n}\frac{B_k}{(1+i)^k}}{\sum_{k=1}^{n}\frac{C_k}{(1+i)^k}}$$

3) Internal rate of return (IRR)

The equalizing discount rate (EDR) is defined as the discount rate at which the cost of the hydropower project and that of alternative thermal power project (benefit of hydro) become equal. The economic internal rate of return (EIRE) is the discount rate at which the cost of the hydropower project and benefit obtained from salable energy and electric tariff become equal. In this Manual, the concept of EDR is adopted in place of EIRR for the reason given in 16.2.1 (4), and for the sake of convenience, the term of IRR having the same meaning of EDR is used. Therefore IRR is defined as the discount rate which gives equal cost of hydro and cost of thermal (benefit of hydro) by the following equation in this Manual.

$$\sum_{k=1}^{n} \frac{B_k - C_k}{(1+i)^k} = 0$$

(4) Evaluation of the project

It can be said that a hydropower project is economically better than an alternative thermal power project if it's NPV, B/C and IRR satisfy the following conditions.

NPV>0

B/C>1

IRR>Discount rate reflecting the opportunity cost of capital

The opportunity cost of capital is the cost when the implementation of another project is prevented by capital being totally committed to the subject project. Here, in principle, the first project to be sacrificed is the one that gives the least rate of return. The internal rate of return of this marginal project corresponds to the opportunity cost of capital.

16.2.3 Benefit of Hydropower Project

(1) Concept and selection of the alternative thermal power plant

There are the following two concepts for alternative thermal power plant:

1) Alternative thermal power

A thermal power plant having the equivalent output as the subject hydropower plant is assumed and the construction cost of the alternative thermal power plant is calculated.

In the study of combination of possible alternative thermal power plants "Screening curve method" is adopted, and as the objective of benefit evaluation the alternative thermal power plant that will provide the best power plant performance and least cost is selected. The annualized cost of each thermal power plant comprises the equalized annual fixed cost and variable cost which is mainly fuel cost corresponding to plant factor. It is expressed by the following linear equation;

Y= annual fixed cost + variable cost × X × 8,760

where,

X : Plant factor

Y : Annualized cost

Figure 16-3 shows the linear representation of the annual cost of each type of thermal power plant. The plant factors corresponding to the intersection points on those linear curves can be calculated, which is the break-even point for economic operation of each plant. The alternative thermal power plant corresponding to the plant factor of hydropower projects is selected. As an alternative thermal power plant for comparison with small hydropower plant in the range between about 1,000 and 10,000 kW, a diesel power plant appears to be the most appropriate.

However, the optimum alternative thermal should be selected after knowing the situation of supply and demand in the given area of power supply.

2) Standard thermal

A thermal power plant considered the most typical of those in a power system where a new project is to be implemented is assumed and, the most typical construction cost per kW is calculated from construction cost of thermal power plants. This method is appropriate when the power system is large and in which many thermal power plants are, or will be constructed.

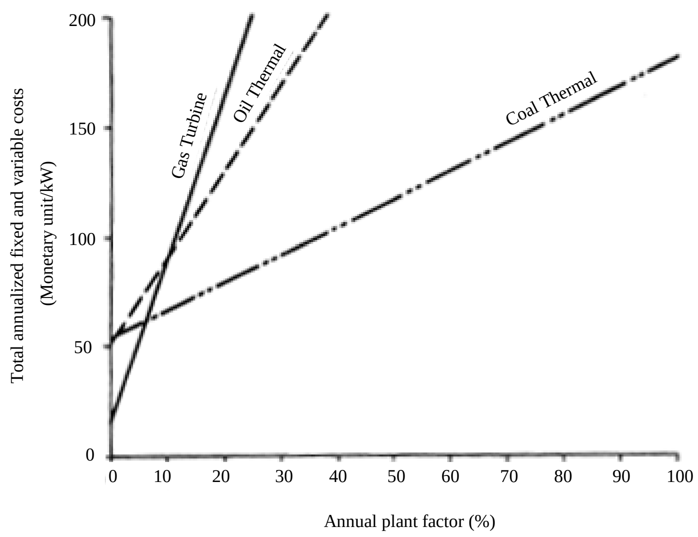


Figure 16-3 Relation between Plant Factor and Annual Cost of Alternative Thermal Power

(2) Salable energy

Energy generation of a hydropower project is classified into primary energy and secondary energy. Salable energy is calculated by deducting the energy consumption for station use and transmission loss from the generated energy of hydropower plant. In case of a run-of-river type, loss should be considered for spilled water due to scheduled outage and forced outage. In case of difference in the value between primary energy and secondary energy, salable energy is calculated separately. In the early stage after commissioning a hydro electric plant, the increment of demand is small and the total energy output of the plant cannot be consumed.

Salable energy is indicated by the quantity which is expected to be consumed.

The salable energy is calculated as follows.

$$E_s = E_H \times (1-H_1) \times (1-H_4)$$

where,

Es : Salable energy (kWh)

E_H : Energy generation (kWh)

H_1 : Rate of station use of hydropower plant

H_4 : Rate of transmission loss

(3) Cost of the alternative thermal power plant

1) Installed capacity of the alternative thermal power plant

Installed capacity is expressed as follows.

$$P_T = P_H \times \text{kW adjustment factor} \times \frac{1 - H_4}{1 - T_4}$$

where,

P_T : Installed capacity of the alternative thermal power plant

P_H : Firm peak output (reservoir type and pondage type) or firm output (run-of-river type) of hydropower plants

H_4 : Rate of transmission loss from the hydropower plant to the load center

T_4 : Rate of transmission loss from the alternative thermal power plant to the load center

The installed capacity of the alternative thermal power plant is determined by taking into account the firm peak output of hydropower plant, and the difference of supply reliability of hydropower plant and thermal power plant. The difference in supply reliability is mainly found in the station use rate, forced outage rate and scheduled outage rate. Compared with hydropower plant, thermal power plant has more instances of forced outage and scheduled outage. Consequently, when a thermal power plant is added to the power system, its installed capacity must be large enough to compensate for its excess outage in order to supply energy with the same reliability as that of a hydropower plant. The factor to calculate the additional required capacity is defined as kW adjustment factor in this Manual.

$$\text{kW adjustment factor} = \frac{(1 - H_1) \times (1 - H_2) \times (1 - H_3)}{(1 - T_1) \times (1 - T_2) \times (1 - T_3)}$$

where,

H_1, T_1 : Rate of station use of hydro or thermal power plant

H_2, T_2 : Forced outage rate of hydro or thermal power plant

H_3, T_3 : Scheduled outage rate of hydro or thermal power plant

2) Construction cost of alternative thermal power plant

The construction cost of the alternative thermal power plant applied in economic analysis should be calculated on the basis of the individual plant. However, its construction cost is not greatly affected by the topographical difference between locations, and in many cases, the calculation is made on the basis of the construction cost per kW (unit construction cost).

Construction cost of thermal power plant (monetary unit) = PT × unit construction cost (monetary unit/kW)

3) Energy generation and fuel cost of the alternative thermal power plant

Annual energy generation of the alternative thermal power plant is arrived by taking into account station service use and transmission loss in relation to saleable energy of hydropower plant. In case of difference in values of primary energy and secondary energy, the amount of energy is calculated separately.

The energy generation of alternative thermal is calculated as follow.

$$E_T = \frac{E_s}{(1 - T_1) \times (1 - T_4)}$$

where,

T_1 : Rate of station service use

T_4 : Rate of transmission loss

E_T : Annual energy generation of the alternative thermal power plant (kWh)

The following is the formula to calculate the fuel cost.

$$\text{Fuel cost (monetary unit)} = E_T \times C_f \times \frac{\text{heat consumption rate (kcal / kWh)}}{\text{Heat rate (lcal / } \ell \text{orkg)}}$$

where

C_f : Unit fuel cost (monetary unit/ℓ or kg)

When performing economic evaluation of primary energy and secondary, fuel saving of existing thermal plants by secondary energy is taken into account to calculate the value of fuel saved.

4) Cost and cost streams of alternative thermal power plant

Cost of the alternative thermal power plant
= Construction cost + replacement cost + operation and maintenance cost + fuel cost

The cost streams of the alternative thermal power plant can be prepared for the cost elements of the above equation for each year throughout the period of analysis. Refer to 16.2.2 (2) for the costs applied in economic analysis.

16.2.4 Cost of Hydropower Project

(1) Construction cost

The construction cost of the hydropower project is estimated in 14.3.

(2) Cost of operation and maintenance (O&M cost)

1) Conventional hydropower plant

This is the cost incurred for operation and maintenance of the power plant after completion of its construction. One of two methods is applied to calculate the O&M cost: the method based on the “ratio to the total construction cost including the dam, reservoir, power plant, and transmission line": or the "method to add each item of cost".

2) Pumped storage power plant

For pumped storage power plant, it is necessary to add to O&M cost of conventional hydro, the cost of energy required for pumping up water. To do this, first select a group of thermal power plants in the power system and allocate the burden of additional work to be shared by each of those power plants to supply energy for pumping and calculate the cost of pumping energy.

In calculating the cost of the pumping energy, it must be considered that the gross efficiency (output energy input energy) is around 70%, that there will be transmission loss during pumping and that the annual operation hours is properly estimated.

(3) Cost and cost stream of hydropower plant

1) Conventional type hydropower plant

Cost of hydropower = Construction cost + replacement cost of equipment + operation and maintenance cost

2) Pumped storage type hydropower plant

Cost of hydropower = Construction cost + replacement cost of equipment + operation and maintenance cost + pumping energy cost

3) Cost streams of hydropower plant

The cost streams are obtained by distributing these costs on an annual basis. The costs used for the economic analysis are described in 16.2.2 (2).

16.2.5 Evaluation of Hydropower Project

The evaluation of the individual project is performed on the basis of the approach and methods of economic analysis described above. Table 16-1 is an example of the annual flow and calculation example of benefits and costs. In this example NPV (B−C) =950 (monetary unit), B/C= 1.16 and IRR= 14.0% are obtained. Judging from the result, it is concluded that the hydropower project is better than alternative thermal power which can provide equivalent service from the viewpoint of cost, and the project is economically sound as long as the discount rate which reflects the opportunity cost of capital does not exceed 14.02%. The benefits and costs used in this evaluation are estimated values, based on a set of assumptions. Thus, in case the assumptions are changed, the economic indicators (NPV, B/C, IRR) can be affected, therefore, sensitivity analysis is conducted to find what effect it will produce. The matter is discussed in 16.4.

Table 16-1 Benefit and Cost Streams and Economic Analysis

(monetary unit)

Serial Number	No. after Completion	Cost				Benefit				
		Investment Cost	O & M Cost	Total	Total (NPV)	Investment Cost	O & M Cost	Fuel Cost	Total	Total (NPV)
1		0.00		0.00	0.00	0.00			0.00	0.00
2		449.50		449.50	401.34	0.00			0.00	0.00
3		1423.20		1423.20	1134.57	0.00			0.00	0.00
4		3327.10		3327.10	2368.16	1351.60			1351.60	962.05
5		3185.80		3185.80	2024.63	1689.51			1689.51	1073.71
6		745.60		745.60	423.07	377.90			377.90	191.73
7	1		120.30	120.30	60.95		101.37	989.41	1090.78	552.62
8	2		120.30	120.30	54.42		101.37	989.41	1090.78	493.41
9	3		120.30	120.30	48.59		101.37	989.41	1090.78	440.55
10	4		120.30	120.30	43.38		101.37	989.41	1090.78	393.35
11	5		120.30	120.30	38.73		101.37	989.41	1090.78	351.20
12	6		120.30	120.30	34.58		101.37	989.41	1090.78	313.57
13	7		120.30	120.30	30.88		101.37	989.41	1090.78	279.98
14	8		120.30	120.30	27.57		101.37	989.41	1090.78	249.98
15	9		120.30	120.30	24.62		101.37	989.41	1090.78	223.20
16	10		120.30	120.30	21.98		101.37	989.41	1090.78	199.28
17	11		120.30	120.30	19.62		101.37	989.41	1090.78	177.93
18	12		120.30	120.30	17.52		101.37	989.41	1090.78	158.87
19	13		120.30	120.30	15.64	1351.60	101.37	989.41	2442.39	317.61
20	14		120.30	120.30	13.97	1689.51	101.37	989.41	2780.29	322.81
21	15		120.30	120.30	12.47	337.90	101.37	989.41	1248.68	148.11
22	16		120.30	120.30	11.13		101.37	989.41	1090.78	100.96
23	17		120.30	120.30	9.94		101.37	989.41	1090.78	90.15
24	18		120.30	120.30	8.88		101.37	989.41	1090.78	80.49
25	19		120.30	120.30	7.93		101.37	989.41	1090.78	71.86
26	20		120.30	120.30	7.08		101.37	989.41	1090.78	64.16
27	21		120.30	120.30	6.32		101.37	989.41	1090.78	57.29
28	22	365.70	120.30	486.00	22.79		101.37	989.41	1090.78	51.15
29	23	1141.03	120.30	1261.33	52.81		101.37	989.41	1090.78	45.67
30	24	1063.75	120.30	1184.05	44.26		101.37	989.41	1090.78	40.78
31	25	327.75	120.30	448.05	14.95		101.37	989.41	1090.78	36.41
32	26		120.30	120.30	3.59		101.37	1480.91	1582.28	47.15
33	27		120.30	120.30	3.20		101.37	1480.91	1582.28	42.10
34	28		120.30	120.30	2.86	1351.60	101.37	1480.91	2933.88	69.70
35	29		120.30	120.30	2.55	1689.51	101.37	1480.91	3271.78	69.40
36	30		120.30	120.30	2.28	337.90	101.37	1480.91	1920.18	36.37
37	31		120.30	120.30	2.03		101.37	1480.91	1582.28	26.76
38	32		120.30	120.30	1.82		101.37	1480.91	1582.28	23.89
39	33		120.30	120.30	1.62		101.37	1480.91	1582.28	21.33
40	34		120.30	120.30	1.45		101.37	1480.91	1582.28	19.04
41	35		120.30	120.30	1.29		101.37	1480.91	1582.28	17.00
42	36		120.30	120.30	1.15		101.37	1480.91	1582.28	15.18
43	37		120.30	120.30	1.03		101.37	1480.91	1582.28	13.56
44	38	452.60	120.30	572.90	4.38		101.37	1480.91	1582.28	12.10
45	39	411.20	120.30	531.50	3.63		101.37	1480.91	1582.28	10.81
46	40	117.20	120.30	273.50	1.45		101.37	1480.91	1582.28	9.65
47	41		120.30	120.30	0.65		101.37	1480.91	1582.28	8.61
48	42		120.30	120.30	0.58		101.37	1480.91	1582.28	7.69
49	43		120.30	120.30	0.52	1351.60	101.37	1480.91	2933.88	12.73
50	44		120.30	120.30	0.47	1689.51	101.37	1480.91	3271.78	12.68
51	45		120.30	120.30	0.42	337.90	101.37	1480.91	1920.18	6.64
52	46		120.30	120.30	0.37		101.37	1480.91	1582.28	4.89
53	47		120.30	120.30	0.33		101.37	1480.91	1582.28	4.36
54	48		120.30	120.30	0.30		101.37	1480.91	1582.28	3.90
55	49		120.30	120.30	0.26		101.37	1480.91	1582.28	3.48
56	50		120.30	120.30	0.24		101.37	1480.91	1582.28	3.11
		13010.43	6015.00	19025.43	7041.27	13516.04	5068.52	61758.01	80342.56	7991.03

NPV(B - C) 949.75718
B/C 1.1348843
IRR(EDR) 0.1401851

16.3 Financial Analysis

16.3.1 Financial Analysis Based on Total Investment

Financial analysis is an analytical evaluation of the total cost to the total revenue of the project from the standpoint of a business entity. The financial internal rate of return (FIRR) is achieved so that the project revenue (electricity sales) converted to present value and total cost converted to present value are equal, and then this FIRR is compared with interest rate of assumed loan. If the value of the FIRR exceeds the interest rate of the loan, the subject project can be considered financially sound.

For computing the FIRR, the discounted cash flow method is applied as in the case of economic analysis. The cost stream is the same as that shown in Figure 16-2, except that the costs Ch and Bt are total investment and operation and maintenance cost, expressed at market prices, which are different from the “economic cost” in the economic analysis.

Since the calculation of total investment made does not take into account financing conditions such as interest rate, repayment of principal, repayment period, etc., the rate of return (earning) of total investment is evaluated irrespective of financing conditions.

A flow chart of calculation of FIRR is shown in Figure 16-4

The price escalation which is deemed to have the same rate as the inflation rate is not usually taken into account. The prices of certain goods or services which are predicted to rise at a rate higher than the inflation rate, and which projections are deemed highly reliable, such as heavy oil price, may have their rises included in the construction cost. In many cases, the influence of price escalation is analyzed by sensitivity analysis.

16.3.2 Financial Analysis Based on Project Equity

A business enterprise usually needs to raise funds before attempting to carry out a project. The following are the indicators generally considered as important by such enterprises when deciding to pursue a project:

(1) Rate of Return

The rate of return is the ratio of the operating profit to the net working fixed assets. If the rate of return is higher than the interest on the procured fund, the interest on the loan can be paid off by the operating profit, and the residual revenue turns to net profit.

(2) Debt Service Ratio

This is ratio of “internally generated fund which is the sum of operating profit and depreciation” to the “payable service which is the sum of redemption and payable interest”. If the debt service ration is larger than 1, the debt service ability of the project is guaranteed, and the project is deemed to be sound. A flow chart of calculation of debt service ration is shown in Figure 16-5. The ration is obtained by taking into account the financial condition of funds, repayment of principal, repayment schedule.

The components of cost in this analysis are the operation and maintenance cost and the depreciation cost. Depreciation cost is calculated on the basis of the construction cost that includes import duties and interest during construction.

16.3.3 Financial Analysis of Hydropower Project

(1) Data for financial analysis

The following data and information are necessary to perform the financial analysis.

1) Financial conditions

(a) Foreign currency portion:

Interest rate, commitment fee, and conditions for repayment of principal and interest

(b) Local currency portion

Conditions of interest rates and repayment of principal and interest

2) Revenue from electricity sale: Average electricity rate

3) Construction cost: Including import duties, contingency and interest during construction

4) Depreciation cost: The method of depreciation (straight line method or declining balance method)

5) Operation and maintenance cost: O&M cost for each facility or component

6) Concept of price escalation

(2) Example of calculating FIRR

An example of FIRR calculation is shown in Table 16-2 which includes cost stream and benefit (revenue) stream by electricity sales. As the FIRR is 13.4% which exceeds the expected interst rate of the loan, this project can be considered financially sound.

Table 16-2 Cost-Benefit Streams and Financial Analysis

(monetary unit)

Serial Number	Year	Cost			Benefit (Revenue)	B-C
		Investment Cost	O & M Cost	Total		
1	1992	0.00		0.00		0.00
2	1993	519.40		519.40		-519.40
3	1994	1810.50		1810.50		-1810.50
4	1995	4502.10		4502.10		-4502.10
5	1996	4470.60		4470.60		-4470.60
6	1997	1095.30		1095.30		-1095.30
7	1998		167.30	167.30	1617.68	1450.38
8	1999		175.00	175.00	1692.09	1517.10
9	2000		183.05	183.05	1769.93	1586.89
10	2001		191.47	191.47	1851.35	1659.88
11	2002		200.27	200.27	1936.51	1736.24
12	2003		209.49	209.49	2025.59	1816.10
13	2004		219.12	219.12	2118.77	1899.64
14	2005		229.20	229.20	2216.23	1987.03
15	2006		239.74	239.74	2318.18	2078.43
16	2007		250.77	250.77	2424.81	2174.04
17	2008		262.31	262.31	2536.35	2274.05
18	2009		274.37	274.37	2653.03	2378.65
19	2010		287.00	287.00	2775.07	2488.07
20	2011		300.20	300.20	2902.72	2602.52
21	2012		314.01	314.01	3036.24	2722.24
22	2013		328.45	328.45	3175.91	2847.46
23	2014		343.56	343.56	3322.00	2978.44
24	2015		359.36	359.36	3474.81	3115.45
25	2016		375.89	375.89	3634.66	3258.76
26	2017		393.19	393.19	3801.85	3408.66
27	2018		411.27	411.27	3976.74	3565.46
28	2019	1624.96	430.19	2055.16	4159.67	2104.51
29	2020	5179.02	449.98	5628.99	4351.01	-1277.99
30	2021	4982.01	470.68	5452.69	4551.16	-901.54
31	2022	1552.32	492.33	2044.65	4760.51	2715.86
32	2023		514.98	514.98	4979.49	4464.52
33	2024		538.67	538.67	5208.55	4669.88
34	2025		563.44	563.44	5448.14	4884.70
35	2026		589.36	589.36	5698.76	5109.39
36	2027		616.47	616.47	5960.90	5344.43
37	2028		644.83	644.83	6235.10	5590.27
38	2029		674.49	674.49	6521.92	5847.42
39	2030		705.52	705.52	6821.92	6116.40
40	2031		737.97	737.97	7135.73	6397.76
41	2032		771.92	771.92	7463.98	6692.06
42	2033		807.43	807.43	7807.32	6999.89
43	2034		844.57	844.57	8166.46	7321.88
44	2035	3616.25	883.42	4499.68	8542.11	4042.44
45	2036	3368.48	924.06	4292.54	8935.05	4642.51
46	2037	994.11	966.57	1960.67	9346.06	7385.39
47	2038		1011.03	1011.03	9775.98	8764.95
48	2039		1057.54	1057.54	10225.68	9168.14
49	2040		1106.18	1106.18	10696.06	9589.88
50	2041		1157.07	1157.07	11188.08	10031.01
51	2042		1210.29	1210.29	11702.73	10492.44
52	2043		1265.97	1265.97	12241.05	10975.09
53	2044		1324.20	1324.20	12804.14	11479.94
54	2045		1385.11	1385.11	13393.13	12008.02
55	2046		1448.83	1448.83	14009.22	12560.39
56	2047		1515.47	1525.47	14653.64	13138.17
		33715.06	30823.60	64538.66	298044.07	233505.41

FIRR 0.133866

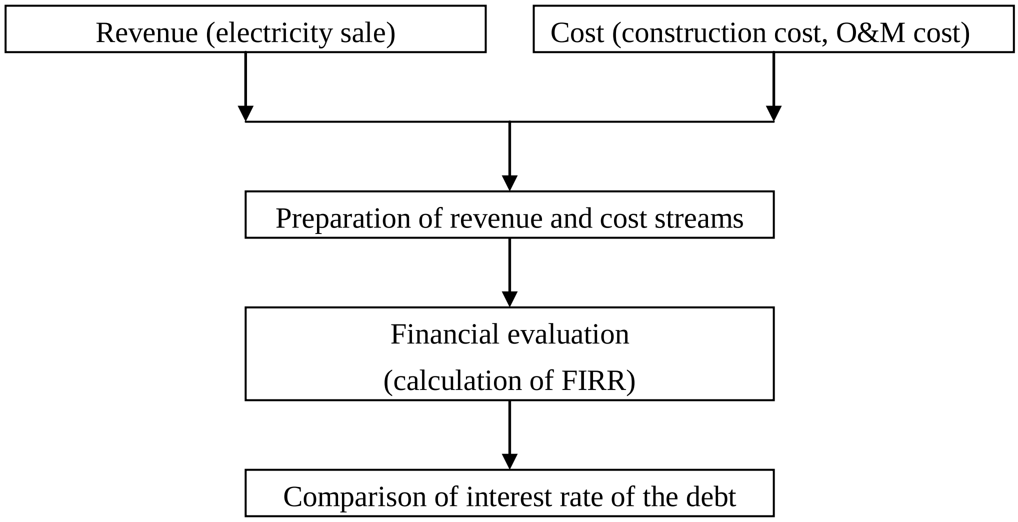


Figure 16-4 Flow Chart of Financial Analysis

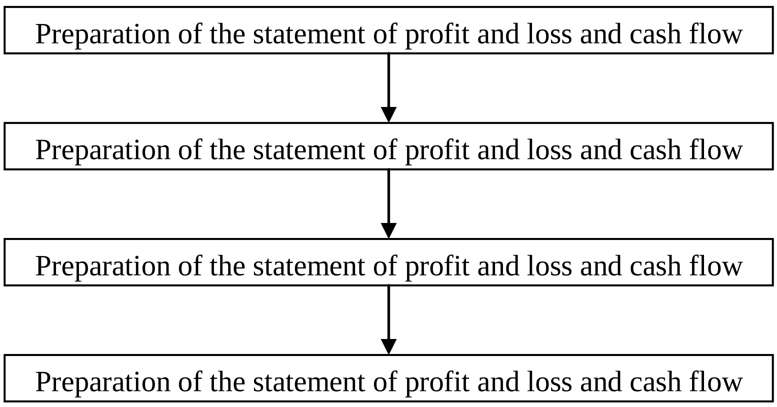


Figure 16-5 Flow Chart of Debt Service Ratio

16.4 Sensitivity Analysis

Costs and benefits in the economic analysis discussed in 16.2 and the financial analysis described in 16.3 are estimated values based on certain assumptions made in the feasibility study.

When changes occur in those assumed conditions and/or values to actual costs and benefits, sensitivity analysis is usually required to see how the changes will affect the indicators for economic analysis (NPV, B/C and IRR) and those of the financial analysis (FIRR).

The probable causes of changes in the benefit stream include “increase or decrease in the electricity sale revenue (unit price of electricity sale)”, “wide deviation of actual runoff from the initial estimate (changes in climate, accuracy of runoff measurement, etc.)”, “price increase (or decrease) of fuel for the alternative thermal power plant”, etc. On the other hand, the causes of changes in costs include “change in construction cost due to price escalation or delays in project completion against the original construction schedule”, “wide difference in actual topographical or geological conditions, etc. to those assumed.”

NPV, one of the indicators of economic analysis, is calculated using a known reliable discount rate. However in practice, there are many unknown factors in the discount rate, and therefore, various discount rates are applied for sensitivity analysis.

In evaluating the project’s economic feasibility, all conceivable cases should be tested in the sensitivity analysis, including those cases where the cost increases, the benefit decreases, the cost increases but the benefit decreases, and applying various discount rates.

Figure 16-6 is an example of sensitivity analysis of an economic analysis under the condition of changed construction cost up to 20% increase.

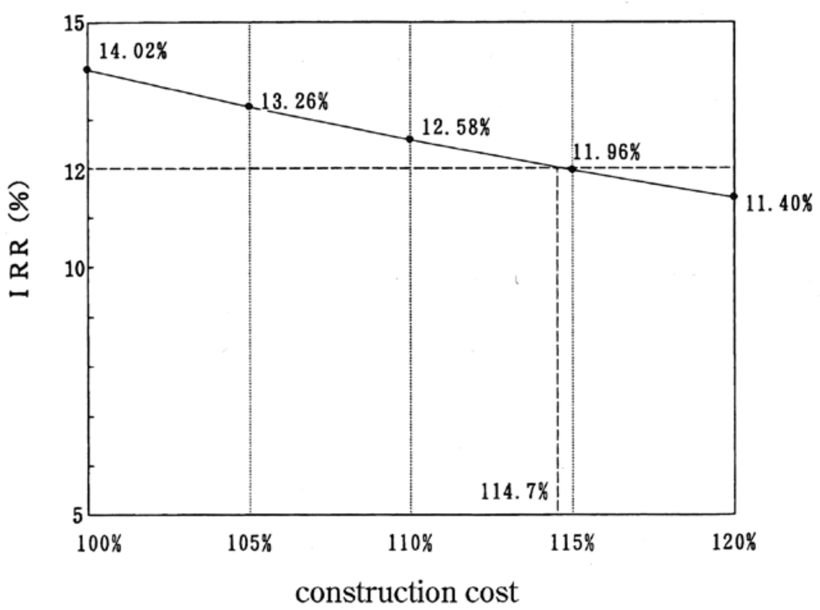


Figure 16-6 Sensitivity Analysis (Economic Analysis)

16.5 Generation Cost

For a utility company, the generation cost is an extremely critical element in determining the electricity sale price. As shown in the equation below, generation cost is derived from total energy generation throughout the period of the plant's service life and the total cost. Total cost is the sum of depreciation, interest, and operation & maintenance cost.

$$\text{Generation cost (monetary/kWh)} = \frac{\text{Sum of depreciation, interest, and O \& M costs}}{\text{Total salable energy during the entire period of Service life}}$$

The values of depreciation and interest are calculated on the basis of the financing conditions.

16.6 Cost Allocation

(1) Concept of cost allocation

Entities joining in the development of multi-purpose dams include power generation, irrigation, flood control, and water supply, etc. The cost of this joint enterprise is called as "joint-use facility cost", and the manner by which the cost is allocated to each participating entity is called "cost allocation". There are following methods of cost allocation, the "alternative justifiable expenditure method" and the "separable-cost alternative justifiable expenditure method".

1) Alternative justifiable expenditure method

In this method, the "alternative facility construction cost (Refer to (2))" and the "justifiable investment" are estimated for each purpose, then the construction cost of joint-use facility is distributed in proportion to the amount which is derived by deducting the "exclusive-use facility cost" from whichever the smaller the alternative facility construction cost or the justifiable investment.

2) Separable-cost alternative justifiable expenditure method

When another party joins in the joint facility which will result in increment of total project cost but without any additional benefit to the other parties in the joint-use facility, the party which is the cause of the incremental cost (separable cost) bears this additional cost to prevent this unfair burden on the other parties.

Methods of cost allocation are as follow:

- The joint-use facility cost to be shared by each party according to the determined allocation ratio.
- Each party should shoulder the construction cost of the exclusive use facility.

(2) Alternative Justifiable Expenditure Method

1) Definition of Terms

(a) Exclusive-use facility cost

The "exclusive-use facility cost" means the cost to be incurred for the construction of facilities including waterway for power, irrigation, etc. of which are utilized in common with the multipurpose dam, but are used solely for one purpose, by one party in the joint undertaking.

(b) Alternative facility construction cost

If a sole use facility with same benefits of joint-use facility and exclusive-use facility is constructed, the estimated cost of the sole use facility is called alternative facility construction cost.

(c) Justifiable investment

Justifiable investment is defined by the following equation:

$$\text{Justifiable Investment} = \frac{\text{Annual benefit} - \text{Annual cost}}{\text{Capitalization factor}}$$

where,

Annual benefit : Estimated value of benefit of the joint-use and exclusive-use facilities.

Annual cost : Estimated cost of operation and maintenance of the joint-use and exclusive use facilities.

3) Example of calculation

Table.16-3 is an example of cost allocation.

In some developing countries, it is not always easy to determine the justifiable investment. In such a case, a simpler method, without calculating the justifiable amount of investment (Item "b") can be applied in cost allocation with the alternative facility construction cost alone.

Table 16-3 Example of Cost Allocation

(Monetary unit)

	Irrigation	Flood control	Power generation	Total
a Alternative facility construction cost	3,500	4,000	7,500	15,000
b. Justifiable investment	3,000	4,500	7,000	14,500
c. Whichever is smaller, a. or b.	3,000	4,000	7,000	14,000
d. Exclusive-use facility cost	500	500	1,000	2,000
e. (c-d)	2,500	3,500	6,000	12,000
f. Share (%)	21	29	50	100
g. Joint-use facility cost allotment	2,100	2,900	5,000	10,000

(Note: Joint-use facility cost 10,000 monetary unit)

(3) The separable-cost alternative justifiable expenditure method

Separable cost is the incremental cost incurred by the inclusion of another use in the common use facilities:

Separable cost = (Construction cost of common use facilities) − (Other purpose facility construction cost)

where,

Other purpose facility construction cost:

In place of common use facilities, the estimated cost of facilities to serve other uses excluding the purpose concerned.

The example is shown in Table l6-4.

Table 16-4 Example of Cost Allocation

(Monetary unit)

	Flood Control	Irrigation	Water Supply	Industrial Water	Power generation	Total
a Alternative facility construction cost	8,280	11,130	7,440	6,870	-	
b. Justifiable investment	18,170	4,446	5,312	4,588	1,419	
c. Whichever is smaller, a. or b.	8,280	4,446	5,312	4,588	1,419	
d. Exclusive-use facility cost	-	2,730	-	-	1,308	
e. (c-d)	8,280	1,716	5,312	4,588	111	
f. Separable cost	1,620	870	720	510	78	3,798
g. Remaining benefit (e-f)	6,660	846	4,592	4,078	33	16,209
h. Share (%)	41.1	5.2	28.3	25.2	0.2	100
i. Allotment of joint-use facility cost remainder	3,211	406	2,211	1,968	16	7,812
j. Amount allocated (f+i)	4,831	1,276	2,931	2,478	94	11,610
k. Share allotted (%)	41.6	11.0	25.3	21.3	0.8	100

Note: Joint-use facility cost 11,610 monetary unit

Reference of Chapter 16

[1] Guide Manual for Development Aid Programs and Studies of Hydroelectric Power Projects, New Energy Foundation, 1996

[2] Statute book on cost allocation (in Japanese), MLIT of Government of Japan, 1982

[3] Lam Ta Khong Pumped Storage Development Project, JICA, 1991

Part 4

Feasibility Study of Pumped Storage Projects

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Chapter 17 Roles of Pumped Storage Projects in Electric Power System

17.1 Characteristics of Pumped Storage Hydropower

17.1.1 Supply Capability Utilizing Surplus Energy of Off-Peak Time

Figure 17-1 shows a daily load duration curve plotted in descending order of load. As seen in this Figure, pumped storage hydropower is a system which pumps up water from a lower pond to an upper pond during night time by utilizing surplus energy of the power system, and it generates during peaking hours when power demand is high. A pumped storage plant can generate about 70% energy of pumped energy (100%), so the energy of 30% becomes loss.

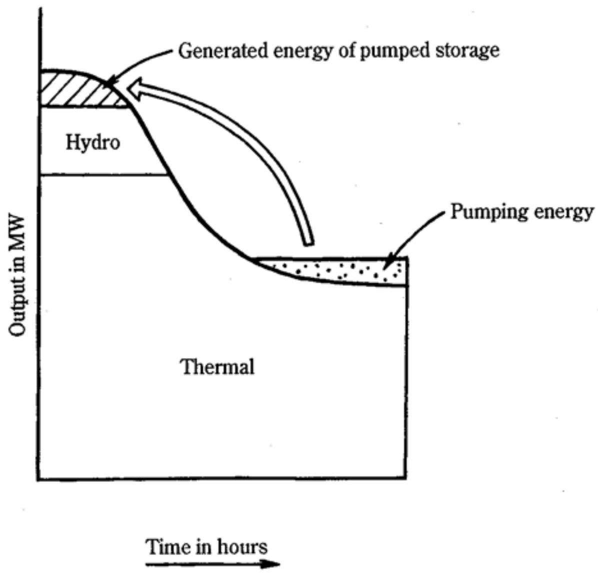


Figure 17-1 Mechanism of Pumped Storage

17.1.2 Load Following Capability

Table 17-1 shows start-up time and load-following capability (LFC) of hydropower, coal fired thermal power, LNG fired thermal power and nuclear power. The hydropower starts up in several minutes and its LFC is more than 50% per minute. On the other hand, the thermal power and nuclear power have characteristics of long start-up time and very low LFC. Therefore since the pumped storage hydropower and reservoir type of conventional hydropower plants have the highest LFC for change in electric power demand, they are utilized as power source to supply for the peak load. The pumped storage power plant has also function to maintain frequency of power system constantly, which is called automatic frequency control (AFC operation).

Table 17-1 Operational Characteristics of Each Power Source

Power source	Start-up time	LFC[1] (%/minute)
Pumped Storage	Several minutes	50 – 60
Oil fired thermal	Several hours	1-3
LNG fired thermal	10 minutes to several hours	1-5
Coal fired thermal	Several hours to more than 10 hours	1-3
Nuclear	Several days	Constant output

17.1.3 Power Supply and Water Usage by Other Sectors

Reservoirs used for hydropower might be also generally utilized for irrigation purpose in developing countries, although power generation is the main purpose.

If a season of water demand for irrigation and power sectors coincides, water usage does not affect power supply. However in the case that a demand season of irrigation sector is a summer and a demand season of power sector for heating is in a winter, demands of both sectors do not get into line and troubles might occur from a view point of stable power supply. In case, irrigation water is released from a reservoir during night time when power needs is low, the stable power supply might be seriously affected.

In order to cope with such situations, pump-turbines and a lower pond are added for conventional reservoir type hydropower plants, and then stable power supply is realized without getting serious influence by irrigation usage of water.

17.2 Role of Pumped Storage in Total Cost of Power System

The annual cost required for operation of a power plant is as follows.

Total Annualized Cost = Fixed Cost + Variable Cost

where,

Fixed Cost : Depreciation and interest, etc. related to project construction cost.
Variable Cost : Energy cost related to the pumping operation.

Figure 17-2 shows the annual cost against the operating hours of major power sources such as base and middle thermal and pumped storage. When the operating hours are short, the pumped storage type shows the lowest annual cost and is, therefore, the most economical. As the operating hours become longer, middle and base thermal power plants become more economical.

- Pumped storage type and conventional type (reservoir type and pondage type) are economical for peak demand of a short period

[1] Load Following Capability

- Base thermal power such as coal fired power is economical for base demand where long operating hours are demanded
- Middle thermal power is economical for intermediate demand.

The total cost of power system becomes the lowest and the system has an optimum combination of power sources by positioning the pumped storage to be the supply power for peak demand,

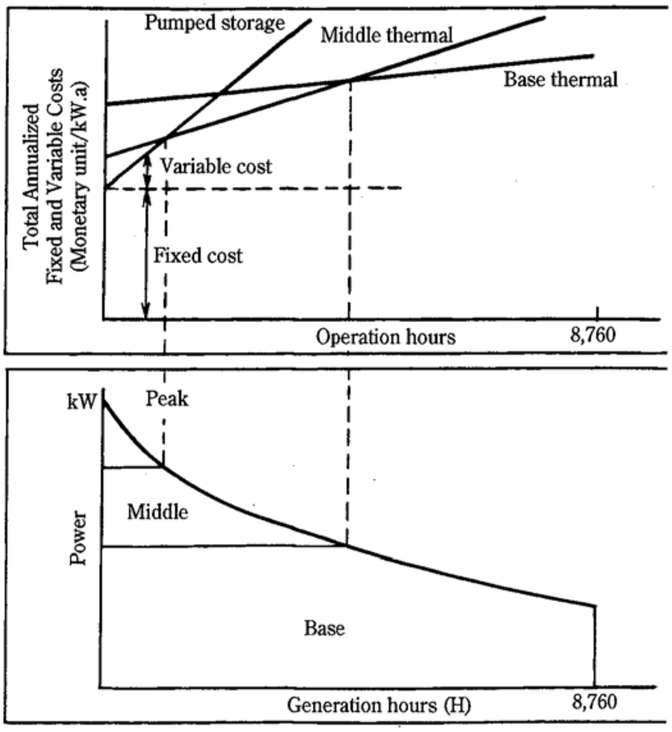


Figure 17-2 Relation between Operating Hours and Annual Cost

Reference of Chapter 17

[1] Guide Manual for Development Aid Programs and Studies of Hydroelectric Power Projects, New Energy Foundation, 1996

[2] Lam Ta Khong Pumped Storage Development Project, JICA, 1991

Chapter 18

Planning of Pumped Storage Projects

Chapter 18 Planning of Pumped Storage Projects

18.1 General

18.1.1 Types of Pumped Storage

Pumped storage hydropower generation is classified into a "pure pumped storage type" and a "pumped and natural flow storage type" as shown in Figure 18-1.

(1) Pure pumped storage type

Electricity of the pure pumped storage type is generated by utilizing the head and circulating water stored in lower and upper ponds. This type is not affected by river flow because the power plant does not use natural inflow water but uses only stored water circulated. Therefore the power output can be set freely in determining the head and maximum plant discharge.

(2) Pumped and natural flow storage type

Electricity of the pumped and natural flow storage type is generated by utilizing both the stored water in lower and upper ponds and natural flow into the upper pond. This type has the merit of reducing the pumping energy by using natural flow into the upper pond. In a case where an upper reservoir is used in common with other sectors, as mentioned in 17.1.3, the power sector might not use it on a priority basis. In such a circumstance the "pumped and natural flow storage type" can supply relatively stable energy in response to the power demand.

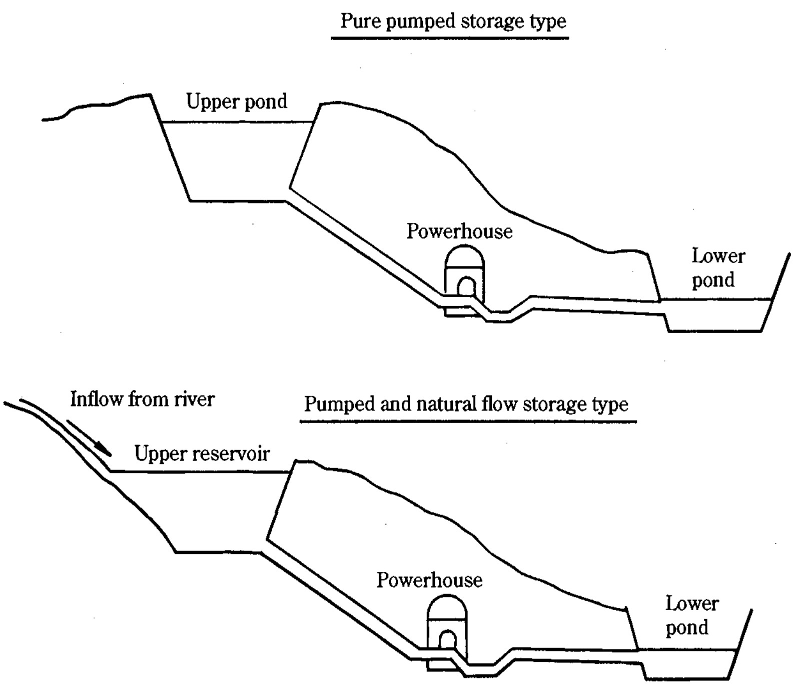


Figure 18-1 Types of Pumped Storage

18.1.2 Consideration for Planning of Pumped Storage

(1) Master plan

A candidate project for feasibility study is selected from the viewpoints of power supply area, pumping energy and transmission line as mentioned in 7.2.2, Chapter 7.

(2) Pumping energy

Since the pumped storage plant itself does not produce energy, it does need pumping energy generated by thermal and nuclear power plants. In this Manual the ratio of generating energy (output energy) to pumping energy (input energy) is defined as "pumping efficiency". The value is determined by length of waterway etc., and is generally about 70%. Since the pumping energy is supplied during the off-peak time when the energy rate is low, the pumped storage power plant can be beneficial even though the power system loses 30% of input energy. The thermal power plants must usually reduce the power output during the off-peak time. As a result the generating efficiency drops and then the generating energy cost increases. Since the pumped storage power plant receives the energy in the nighttime, the thermal power can improve its efficiency of generation.

(3) Transmission capacity

The stability of transmission when a pumped storage plant is put into the system should be studied by taking account of the present transmission capacity and future plans. Then, upper limit of the pumped storage development can be confirmed.

(4) Unit capacity

1) Necessity of study on unit capacity

Generally, there is a tendency that the larger a unit capacity of generating plant is, the smaller the construction cost (civil structures and electrical &mechanical equipment) per kW becomes. The unit capacity of the project should be chosen as large as possible in consideration of the reliability of the plant, manufacturing technique and means of transportation of equipment. While the use of a large unit capacity has the above economical advantage (scale-merit), it has a disadvantage of producing large frequency fluctuations in a power system at starting and stopping of pumping operation. Another disadvantage using a large unit is that it requires large operating reserves for its outages.

Therefore, the unit capacity is determined by considering the following conditions.

(a) It does not exceed the maximum unit capacity of thermal power plants
(b) Power (kW) required for pumping increases by about 10% over the generating output.
(c) System disturbance might be caused by a failure of the unit during its generating operation.

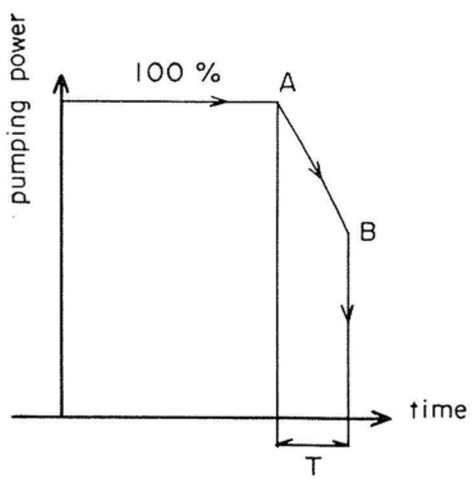
(d) Following should be considered for study

- System capacity of off-peak time generally falls to nearly one half of the level of the peak time although depending on the power system.
- Frequency fluctuations by a pumping unit always occur at an ordinary start and a stop besides at a unit failure.

2) Pumping Operation and Frequency Fluctuation

(a) Process of pumping stop and its effect

In normal operation of a pumped storage power plant, a pumping stop causes very severe frequency fluctuations in the power system. Frequency fluctuation by a pumping stop is a little larger than that by a pumping start. Stopping of pumping operation is normally carried out by the process shown in Figure 18-2.



A: A guide vane starts closing.

B: Pumping stops and the machine is disconnected from the power system.

T: Several tens of seconds

Figure 18-2 Example on Process of Pumping Stop

When pumping stops, because of a sudden disconnection of the load from a power system, the equilibrium condition between supply and demand breaks. The imbalance between power supply and demand causes promptly the increase of power system frequency from the level before the pumping stop. When pumping starts, an imbalance between power supply and demand also occurs. The power system frequency goes down in this instance.

The fluctuation of system frequency becomes larger as the amount of load change is larger, because the frequency control by power sources cannot keep up with a rapid frequency change. The deviation from the specified frequency causes numerous unfavorable influences to the facilities of suppliers and users of electric power. Especially, the resonance between the natural frequency of the turbine blades in the thermal power plants and power system frequency will cause a very serious damage to the machines. So, great care must be taken to avoid this condition and to safeguard the equipment.

(b) Frequency Fluctuations at Pumping Stop

The frequency fluctuation caused by a sudden change in the system load can be estimated by the following equation.

$$\frac{\frac{\Delta P}{P} \times 100}{\Delta F} \times \frac{1}{10} = K$$

Where,

ΔP	: Power supply - demand imbalance (MW)
P	: System capacity (MW)
ΔF	: Amount of frequency fluctuation (Hz)
K	: Power-frequency characteristic (MW/0.1Hz)

A power frequency characteristic K varies depending on the frequency characteristics of the load, kinds of power sources and their operating conditions. However, K value is almost constant for each electric power system in general. Using these values, frequency fluctuation ΔF caused by pumping start and stop of a machine with assumed unit capacity can be estimated in the power system, when the pumped storage power plant will be commissioned. The same calculation is done by changing the unit capacity, and then the unit capacity which power system allows can be determined.

18.1.3 Methodology of Planning

(1) Flow of planning

A flow chart of planning of a pumped storage project is the same as Figure 8-1, Chapter 8, however more precise data are used.

The following should be considered.

1) A master plan for pumped storage should be reviewed from the viewpoint of utilizing pumped storage resources most effectively. (See: 7.3).
2) The most appropriate layout is selected, after comparing several alternative sites from the viewpoints of topography, geology, environment, and economy (See 5.4).
3) Development scale(Maximum output) is studied and decided from the viewpoints of demand & supply balance, unit capacity, capacity of transmission line, stability of power system.
4) Taking into account the present and future constraints for the development, whether the project is developed at full scale or partially is analyzed and decided.

(2) Study on power output

1) Planning of pond

The following method is applied for planning. Concerning the same contents as the conventional hydropower planning, the section number is written.

(i) Preparation of storage-capacity curve, estimate of sedimentation volume, setting of sedimentation level (See 10.2.3(2)), setting of LWL (See 10.2.3(4))
(ii) Study on the maximum output and storage capacity of a pond
(iii) HWL is set for an upper pond and a lower pond considering of LWL and regulating capacity (Ve)

- HWLs and LWLs of the both ponds should not give a harmful effect to pump turbines.
- The water levels of ponds drop fast, so the stability of slope along the ponds might be affected.

2) Elevation of pump center

The pump center should be determined so as to avoid cavitation shown in Figure 5-36 (See 5.4 as a reference)

18.2 Planning

18.2.1 Operation Plan of Pumped Storage Power Plant

(1) Relation between storage capacity of pond and peak duration hours

The pond for a pumped storage type is generally classified into a "daily regulation" type which continuous generating capability is 4 to 6 hours, and a "weekly regulation" type which continuous generating capability is more than about 8 hours. The continuous generating capability is defined as follow.

The storage capacity of the pond for a pumped storage type, as shown in the following equation, can be expressed by continuous generating capability at about the maximum output for peak duration hours.

$$T = \frac{Ve}{Q_{max} \times 3{,}600}$$

Where,

T	: Continuous generating capability (storage capacity of pond; hours)
V_e	: Effective storage capacity of pond ($10^6 m^3$)
Q_{max}	: Maximum plant discharge (m^3/sec.)

(2) Power plant having pond of daily regulation

A schematic figure of upper pond operation for daily operation is shown in Figure 18-3. The water level of an upper pond changes from a high water level (HWL) to a low water level (LW) during the generation, and it changes from LWL to HWL during the pumping within the period of one day.

The daily generating energy is supplied from daily pumping energy as follows.

$$E_0 = \alpha E_{pl}$$

Where,

E_0	: Daily generating energy (kWh)
E_{pl}	: Daily pumping energy
α	: Gross efficiency of pumped storage power plant (generating energy/pumping energy, about 0.7)

The possible daily generating hours is expressed to be smaller value of "Continuous generating capability" or "Continuous generating hours determined from pumping energy to upper pond per day ".

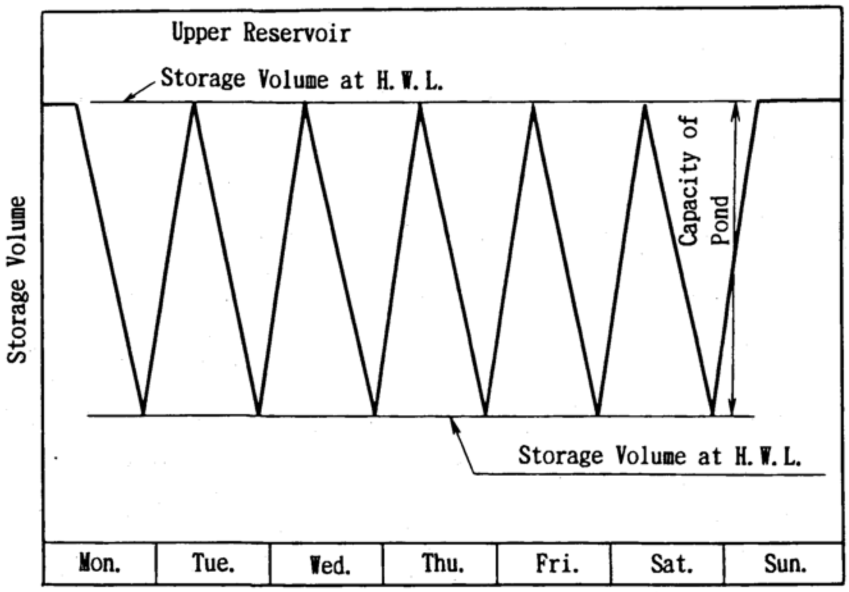


Figure 18-3 Concept of Daily Regulating Pond (Upper Pond)

(3) Power Plant having pond of weekly regulating

Figure 18-4 shows a comparative example of daily load curves between weekday and Sunday. In a case where the power demand decreases obviously on weekends, the pumped storage for weekly operation should be studied as an example below. The power plant generates from Monday to Friday in principle by using the daily and weekly pumping energy. The upper pond is again filled up to HWL usually on Sunday.

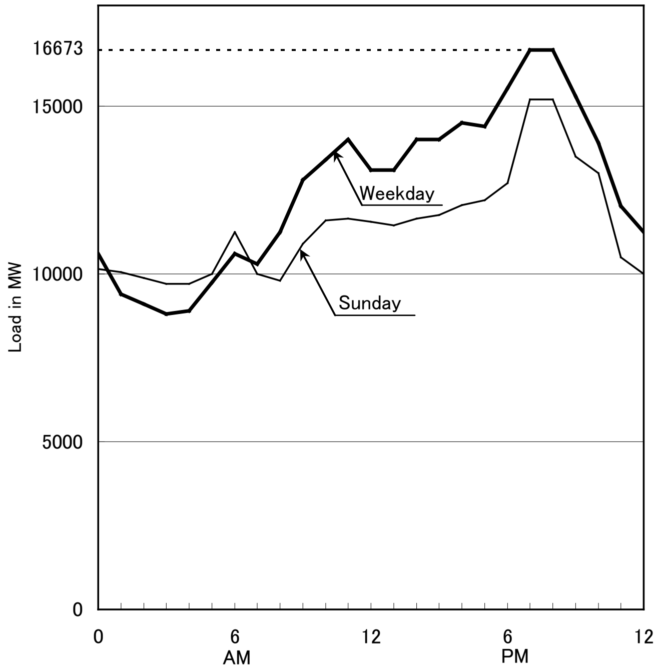


Figure 18-4 Daily Load Curves for Week Day and Sunday

An example of upper pond operation for weekly use is shown in Figure 18-5 and main points are as follows.

- A pumped storage power plant is required to generate and pump up from Monday to Saturday.
- Only pumping is required on Sunday so that the water level of upper pond rises to HWL by Monday morning.
- The daily generating energy is supplied with daily pumping energy and pumping energy supplied on Sunday（5 days for the example）.
- Since generation on Saturday is smaller than week day, the plant generates on Saturday by using the daily pumping energy only.

The daily generating energy is supplied with daily pumping energy and weekend pumping energy as follows.

$$E_0 = \alpha (E_{pl} + E_{p2})$$

Where,

E_0 : Generating energy (kWh)
E_{pl} : Weekday pumping energy (kWh)
E_{p2} : Pumping energy on Sunday(kWh)
α : Gross efficiency of pumped storage power plant

Generated energy by a pumped storage power plant is expressed with number of operation hours at maximum output level. The possible daily generating hours of T_d for Monday to Friday is expressed as follows and shown in Figure 18-5.

$$T_d = (T - J)/D + J$$

where,

T_d : Possible daily generating hours (maximum output level: hours)

T : Continuous generating capability ((storage capacity of pond; hours)

J : Possible daily generating hours related to possible daily pumping hours (hours)

D : Number of days using weekly regulating portion of reservoir capacity

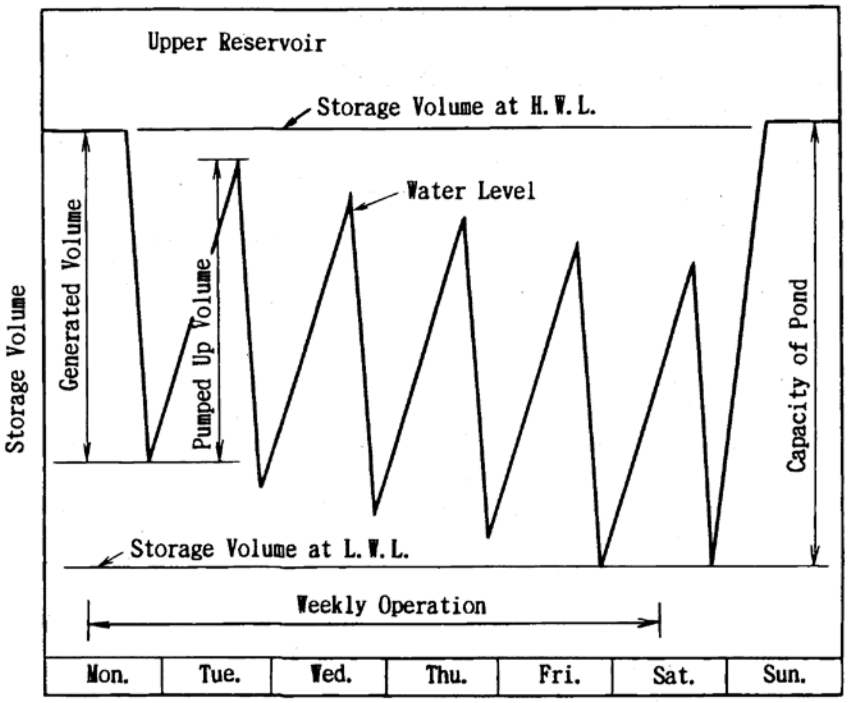


Figure 18-5 Operation of Weekly Regulating Pond

18.2.2 Optimization Study on Maximum Output and Storage Capacity of Pond

(1) Methodology of optimization study

The optimum development plan concerning the maximum output and storage capacity of pond is determined from several alternatives by economic comparison.

Cost (C) of the pumped storage hydropower project is compared with the cost of alternative thermal project having equivalent supply capability (kW, kWh) to the hydropower. Net present value (NPV, B-C), benefit cost ratio (B/C) and/or internal rate of return (IRR) are calculated, and the alternative plan having their maximum value is selected as the optimum development plan. The details are explained in Chapter 16.

(2) Example of Optimization Study

The alternatives are compared by using B-C and B/C values. Output (MW) and continuous generating capability (hr) of alternatives are in the range of 500MW to 1,200MW and 6 hours to 10 hours respectively.

Significant difference in economics between 1,000MW and 1,200 MW cannot be seen for development output. Storage capacity of 8 hours is most economical for the continuous generating capability. As a result of the study, the alternative having the maximum output of 1000 MW and continuous generating capability of 8 hours is selected.

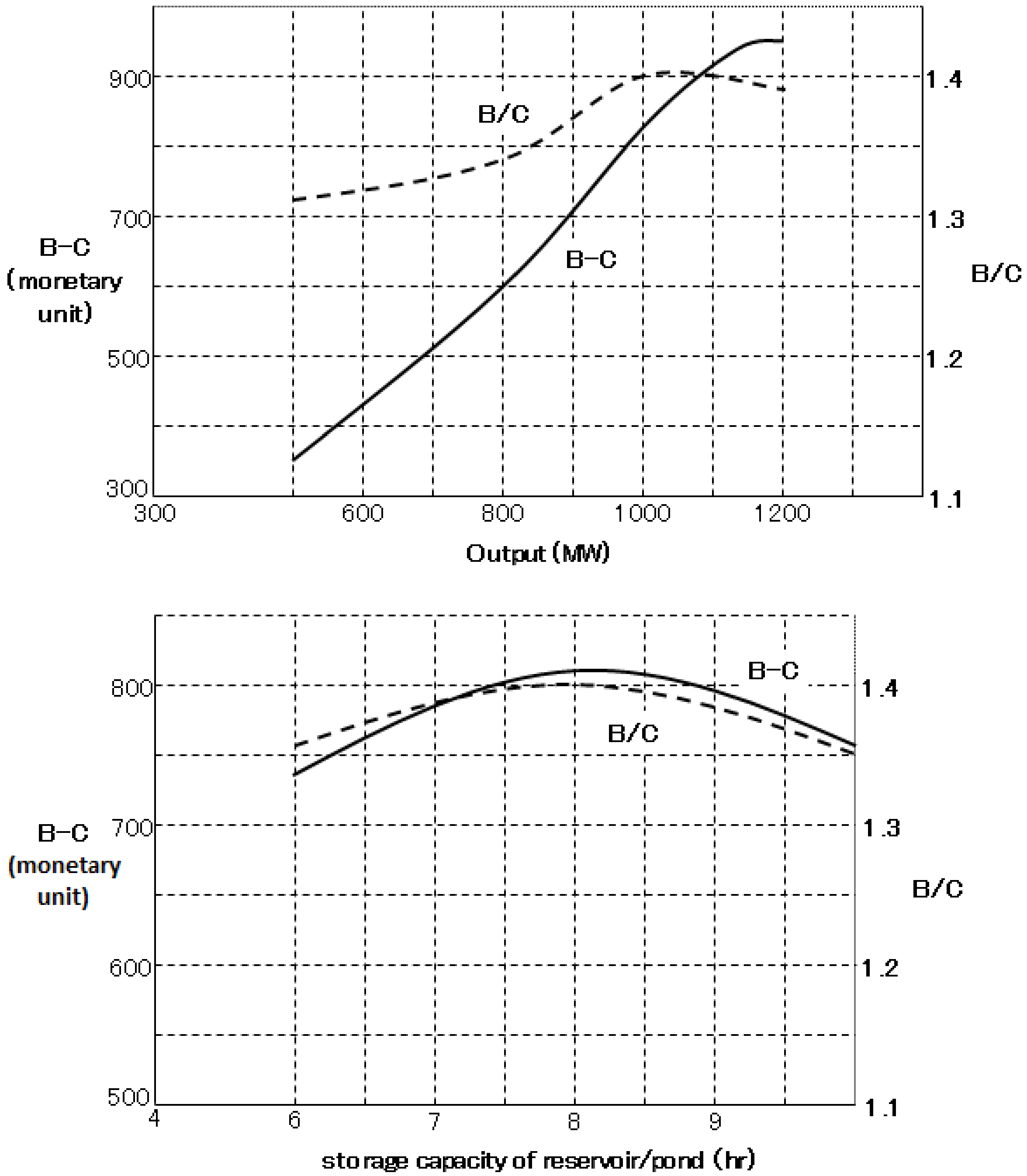


Figure 18-6 Optimization Study

18.3 Study on Timing of Development and Development Method

18.3.1 Constraints of Development

(1) Power demand is less than the maximum output

In this situation, only a part of total output can be used, for example 500MW can be used instead of 1,000MWdevelopment. Although the power demand increases year by year, in a case where it takes a long time to use the total output, a study on the partial development should be taken into account.

(2) Capacity of transmission line, and scale of power system

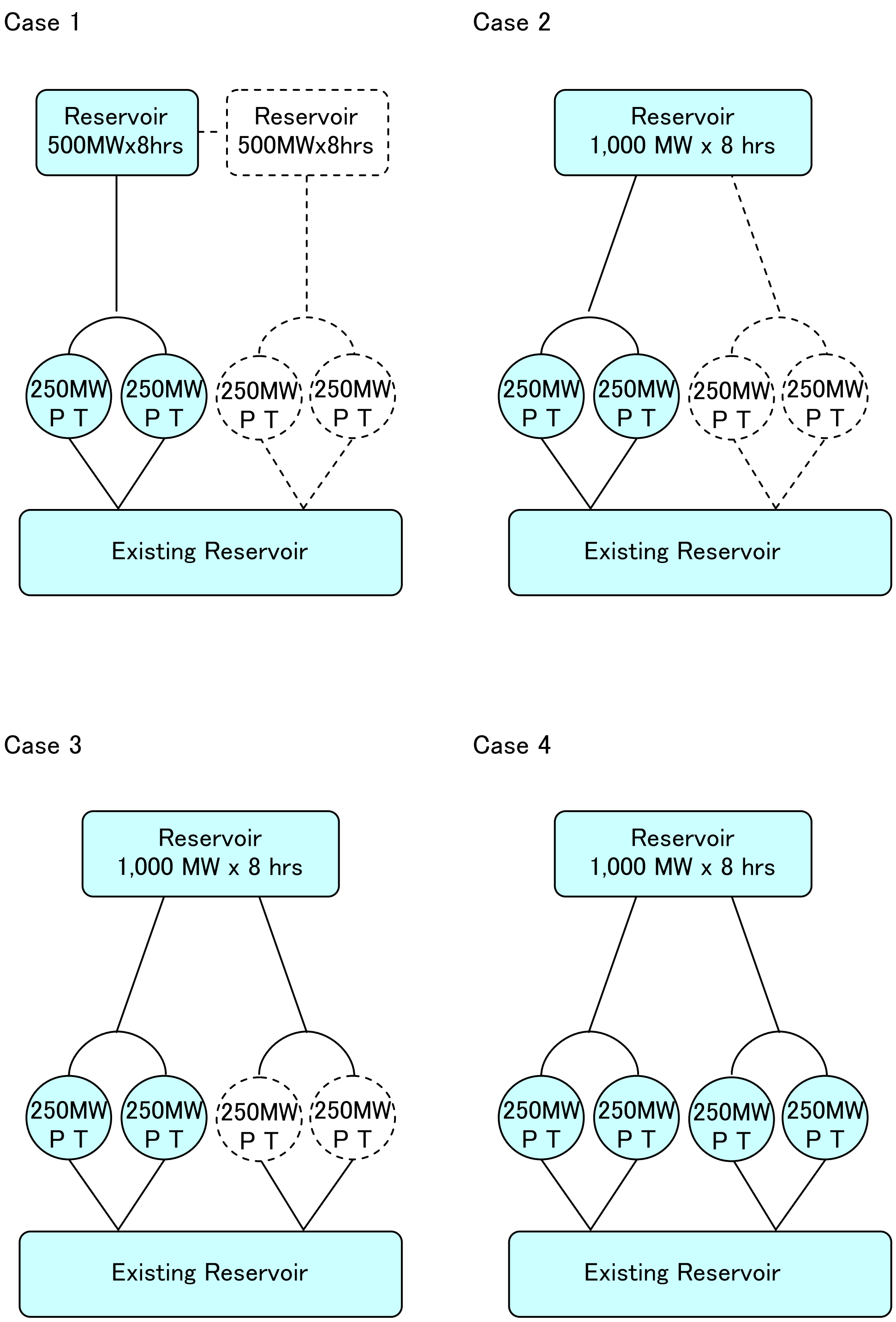
In a case where the capacity of transmission line is not enough to receive and send the full power of the pumped storage plant, it is not possible for the plant to generate at full capacity. And there might be also a problem in a case where the scale of power system is small compared with the unit capacity of pumped storage.

18.3.2 Development under Constraints

Figure 18-8 shows an example of 1,000MW (250MW x 4units) and storage capacity of 8 hours continuous operation. As an existing reservoir is used as a lower pond in this example, an upper pond, waterway, powerhouse, electro-mechanical equipment are newly constructed and installed. Four development methods are considered as shown in Table 18-1 and Figure 18-8. Selection of these alternatives is determined taking into account the prospect to ease the constraints.

Table 18-1 Development Case Under Constrains

CASE	Development scheme
Case1 1st Stage	Upper reservoir/pond (500 MW x 8 hours), Other civil works for 250 MW x 2 units Generating Facilities 250 MW x 2 units
2nd Stage	Development after n years from 1st stage Upper reservoir/pond (500 MW x 8 hours), Other civil works for 250 MW x 2 units, Generating facilities 250 MW x 2 units
Case 2 1st Stage	Upper reservoir/pond (1000 MW x 8 hours), Other civil works for 250 MW x 2 units, Generating Facilities 250 MW x 2 units
2nd Stage	Other civil works for 250 MW x 2 units (Waterway and powerhouse), Generating Facilities with 250 MW x 2 units
Case 3 1st Stage	Upper reservoir/pond (1000 MW x 8 hours) Other civil works for 250MW x 4units Generating Facilities 250MW x 2units
2nd Stage	Generating Facilities 250 MW x 2 units
Case 4 1st Stage	Upper reservoir/pond (1000 MW x 8 hours) Other civil works (1000MW x 8 hours) Generating Facilities 250 MW x 4 units



Note: Solid line: 1st stage development
Dotted line: 2nd stage development
PT: Pump & turbine
Existing reservoir: Lower reservoir

Figure 18-7 Example of Development

Reference

[1] Feasibility Study on Lam Ta Khong Pumped Storege Development Project, JICA, 1991

Chapter 19

Design of Pumped Storage Projects

Chapter 19 Design of Pumped Storage Projects

19.1 Civil Structures

(1) Dam

The design of a dam for a pumped storage hydropower plant is basically the same as for a general hydropower plant described in Chapter 11. Characteristic of a dam for pumped storage hydropower is a deep-excavated pond type, which is described here.

A dam of a deep-excavated pond type is a fill dam which is formed at a rather flat site by excavation of the ground and is made of fill materials with surface faced structure by asphalt facing etc. The dam height is determined ensuring a pond capacity to meet peak demand operation as well as balancing the volume between excavation and embankment. A plane shape of the pond is circular, close-to-circle rectangular, and polygonal depending on the surface facing materials.

An example of a deep-excavated pond type of dam is shown in Figure19-1 and Figure19-2.

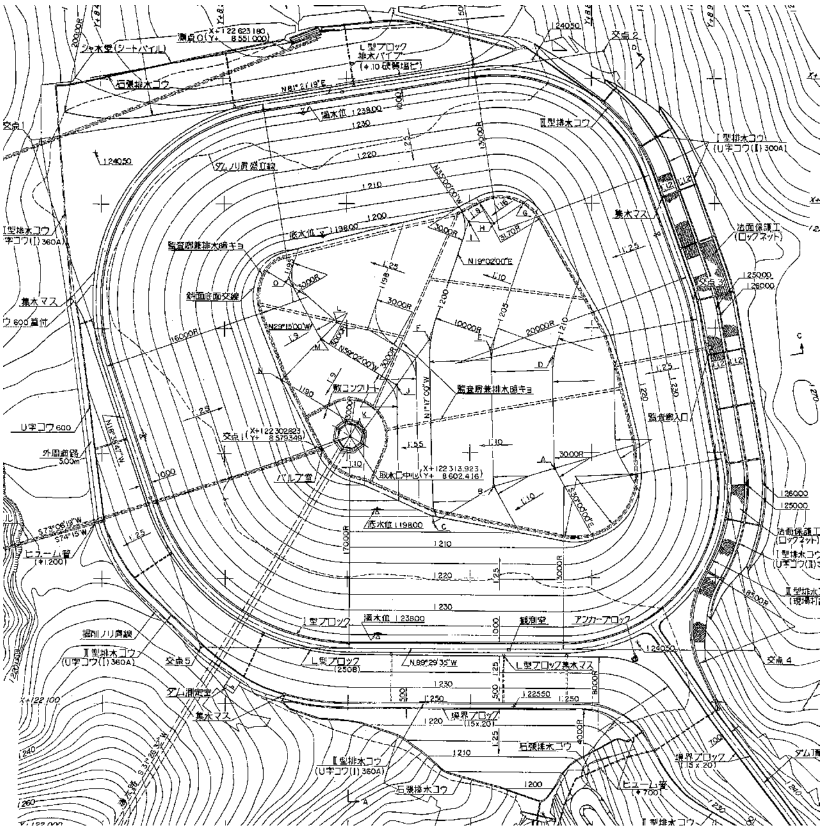


Figure 19-1 Example of Deep-Excavated Pond Dam; Plan

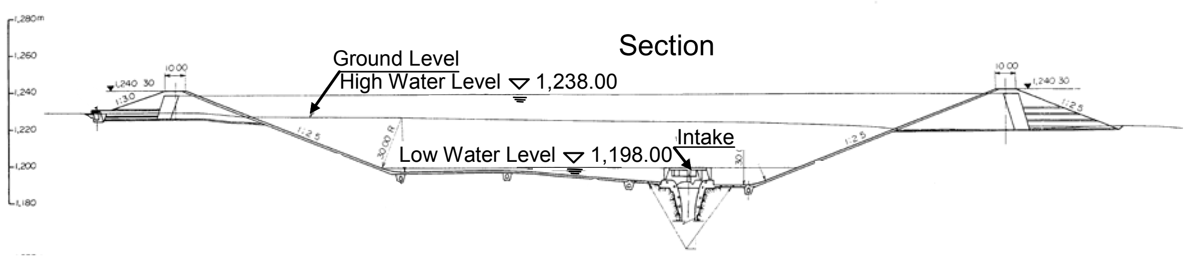


Figure 19-2 Example of Deep-Excavated Pond Dam; Typical Section

(2) Intake・Outlet

An intake for a pumped storage hydropower plant is basically the same as that of a pressure waterway. However, it is reminded when designing that an intake functions as an outlet at the time of pumping. The velocity in a waterway for pumped storage hydropower is set at a larger value compared with that in run-of-river type hydropower waterway. This is because the velocity loss does not matter much to a pumped storage hydropower scheme with such a high head inherently. It is necessary to have an intake and an outlet shaped well to the rather high velocity in order to ensure a smooth flow.

For a pure pumped storage hydropower plant, adopted in many cases are a multi-box culvert sectional horizontal intake and a morning glory shaped vertical intake.

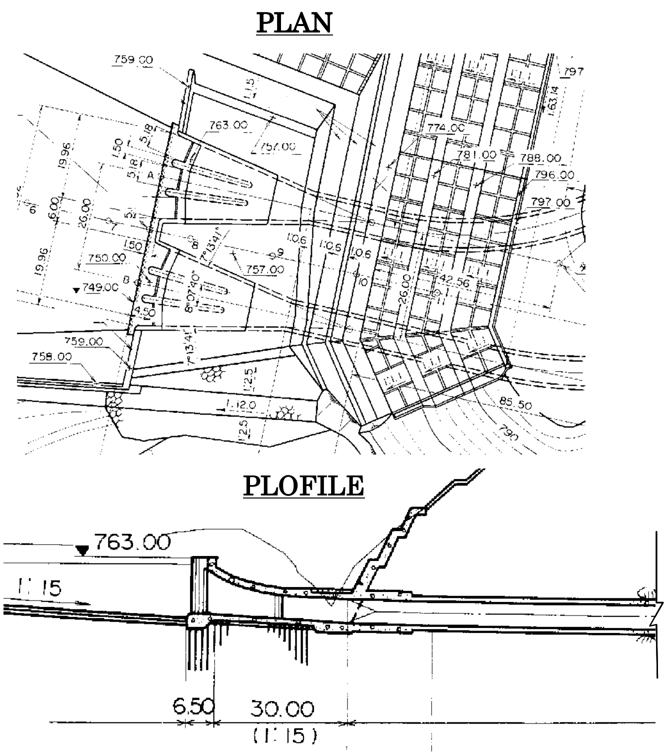


Figure 19-3 Example of Horizontal Intake for Pumped Storage Hydropower

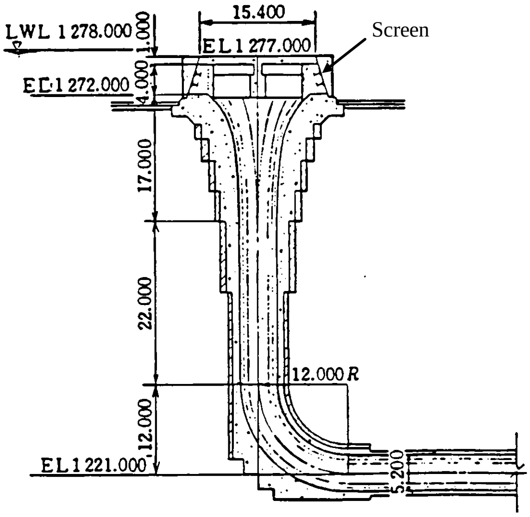


Figure 19-4 Example of Morning Glory Intake for Pumped Storage Hydropower

(3) Headrace

The design of headrace for a pumped storage hydropower plant is the same as for a general hydropower plant, and it is usually planned as a pressure tunnel. The design velocity is set at a little higher value compared with that for general hydropower. Reference is made to 11.4, Chapter 11.

(4) Surge tank

A surge tank for a pumped storage hydropower plant is planned in the same way as for general hydropower and is usually designed as a pressure surge tank. Reference is made to 11.5.

(5) Penstock

A penstock of a pumped storage hydropower plant is planned in the same way as for general hydropower and the velocity is usually set at a little higher value than that for general hydropower. Reference is made to 11.6.

(6) Power Plant Foundation

Because a suction head is required during pumping for a pumped storage hydropower plant, it is usually planned underground. The design of a power plant is the same as for general hydropower. Reference is made to 11.8.

(7) Tailrace

A tailrace for a pumped storage hydropower plant is usually planned as a pressure tunnel. The design is the same as for general hydropower.

In the case of a long tailrace (an approximate value of 3,000 m^2/sec or more by multiplying the tunnel length and flow velocity), it is necessary to study the necessity of a tailrace surge tank. Reference is made to 11.9.

19.2 Electro-Mechanical Equipment Design for Pumped Storage Power Plant

19.2.1 Pump Turbine

Design parameters for a pumped storage power scheme such as water level conditions of intake and tailrace (maximum, normal, and minimum) and a maximum generating discharge, storage capacity of upper reservoir and operation time, maximum output are selected by civil engineers. Electro-mechanical engineers begin pump turbine and generator motor design for each project case. The following describe selection and design methods for pump turbines, generator motors, auxiliary equipment and electrical circuits.

(1) Pumped storage type

For main unit arrangement of a pumped storage power plant, there are a reversible (binary) type which installs a pump waterwheel and generation motor having both functions of pumping and generating, and a tandem (ternary) type which installs a pump and a turbine on coax with one generator motor, and a separate unit type which installs independently one turbine/generator and one pump/motor. These systems are shown in Figure 19-5.

1) Separate unit system

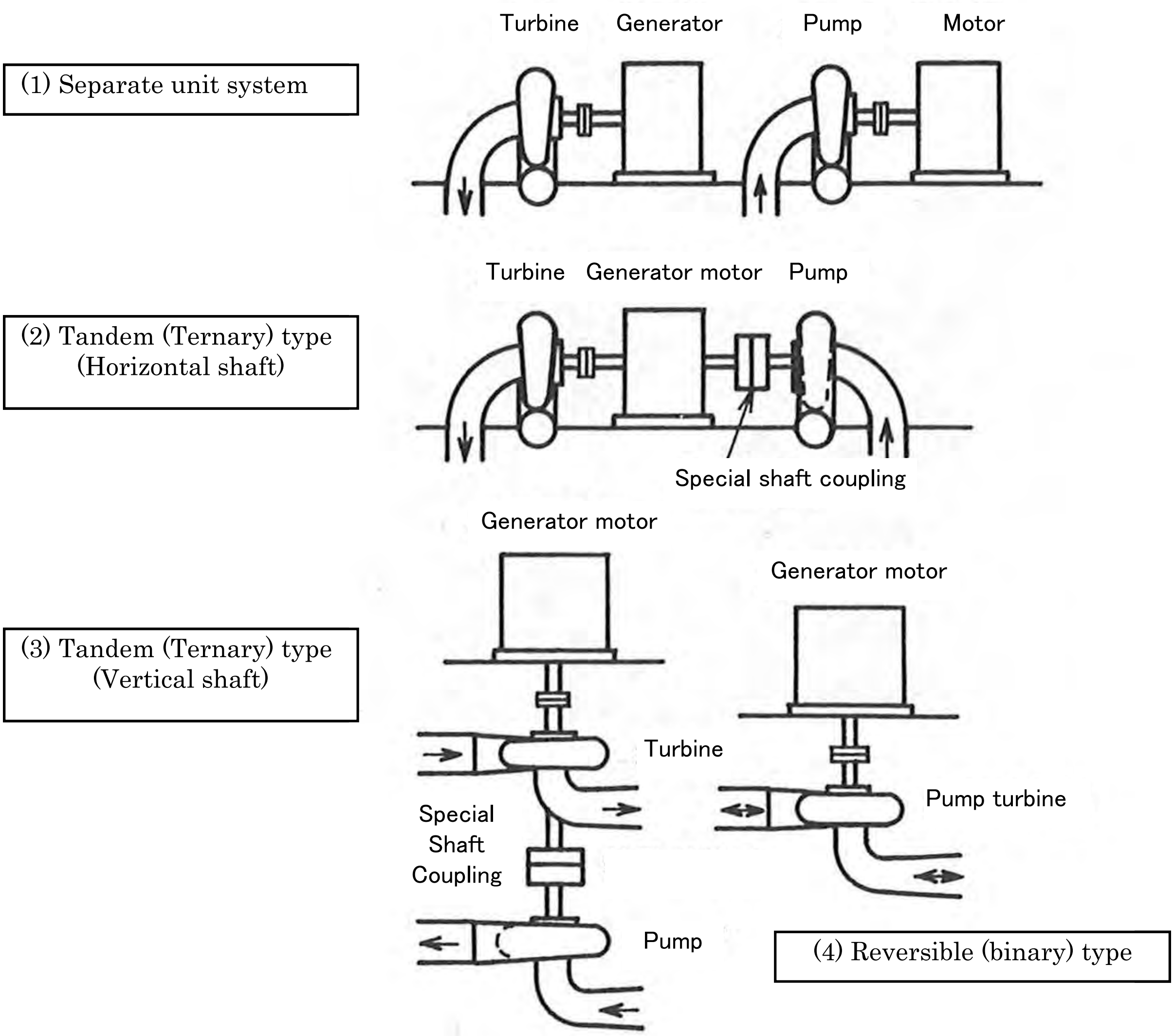
In a separate unit type, the turbine/generator and pump/motor are installed separately. The machine and powerhouse building are expensive and it is not used at present.

2) Tandem (ternary) type

In a tandem type (ternary), the same electric machine is used for pumping and generating. The rotation direction of the turbine and pump can be made the same and turbine operation and pump operation can be switched in a short time. The turbine and pump can be designed separately to be operated on optimum conditions respectively to attain high efficiency. As this type has some other advantages, it is still used today. Recently, the introduction ratio to an electric power system of renewable energy such as wind-power generation has been steadily increased. Because tandem type can adjust operation by sudden pumping and quick generating, and carry out simultaneous operation of the turbine and pump, this type has been reevaluated. It is a demerit of this type that machine and powerhouse building are expensive, though not so much as a separate unit system.

3) Reversible (binary) type

A reversible (binary) type uses the same hydraulic machine and electrical machine for generating and pumping. The rotating direction is usually reverse between turbine operation and pump operation, and therefore, this is called a reversible type. Although its operation characteristics are inferior to those of the separate unit type and tandem type, its electrical and civil works construction costs are lowest. It is also advantageous in easy maintenance and inspection. Due to the advantages described above, this type of large capacity vertical shaft machine has been used for the latest pumped storage power schemes.

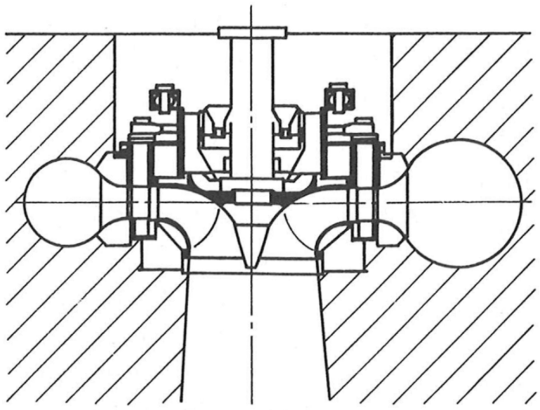


Source: Hydro turbine

Figure 19-5 Pumped Storage Type

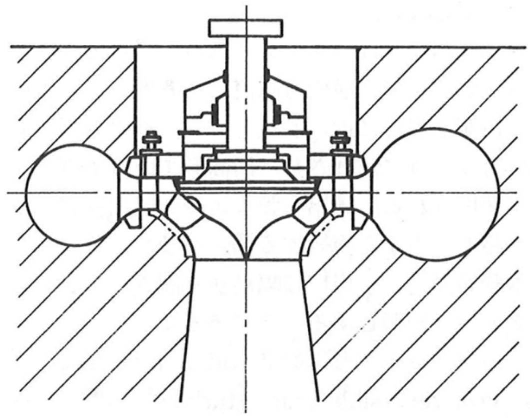
(2) Classification of pump turbine

In the case of a unit type and a tandem type of pumped storage type classification, the characteristics and structure of the main machine are on the extension of a conventional water turbine or pump, and there is little specialty as pump turbine. Therefore, described below is mainly the reversible type pump turbine. Of this type are a Francis type pump turbine, a diagonal-flow type pump turbine, and a propeller type pump turbine as shown in Figure 19-6, Figure 19-7 and Figure 19-8.



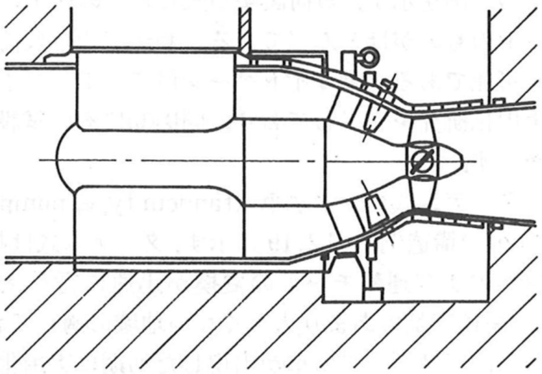
Source: JIS-B0119 Hydraulic turbine and reversible pump turbines-Vocabulary

Figure 19-6 Vertical Shaft Single Stage Francis Type Reversible Pump Turbine



Source: JIS-B0119 Hydraulic turbine and reversible pump turbines-Vocabulary

Figure 19-7 Vertical Shaft Diagonal Type Reversible Pump Turbine

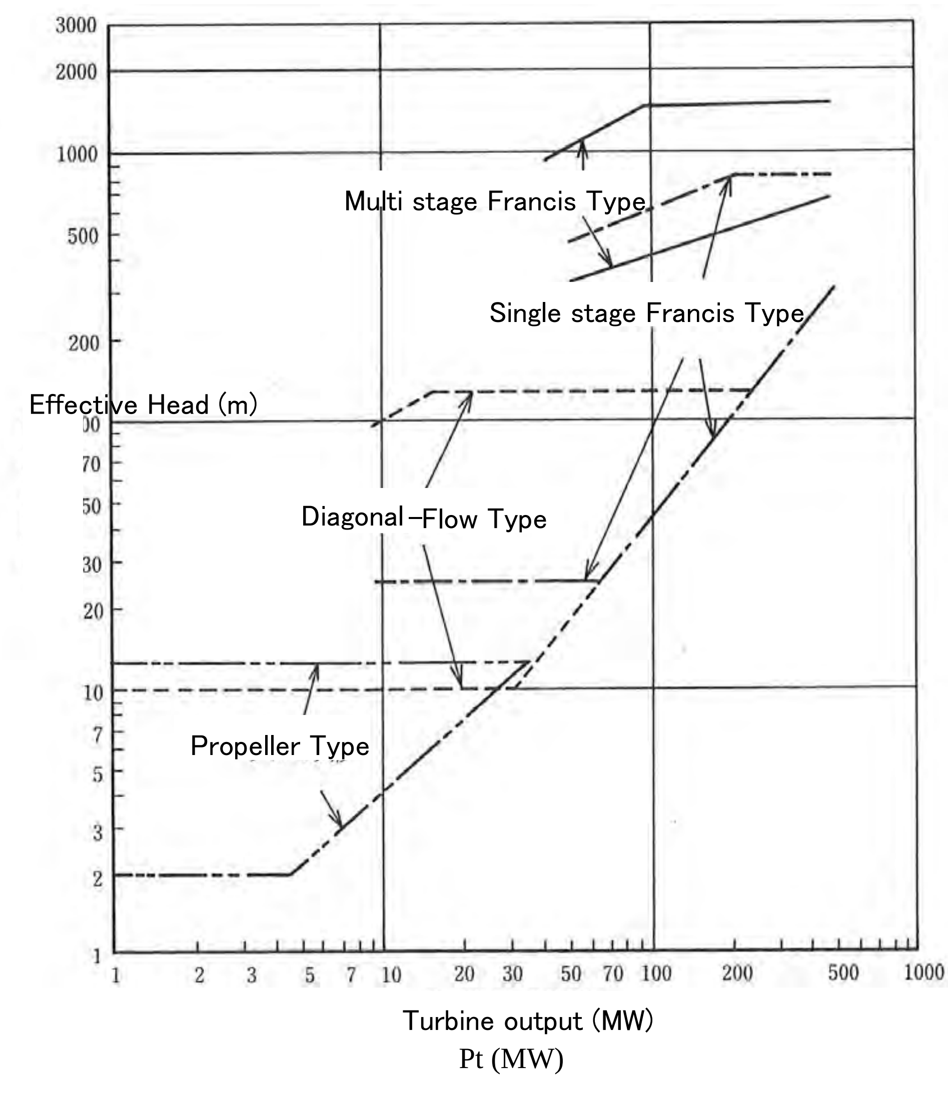


Source: JIS-B0119 Hydraulic turbine and reversible pump turbines-Vocabulary

Figure 19-8 Horizontal Shaft Tubular Type Pump Turbine

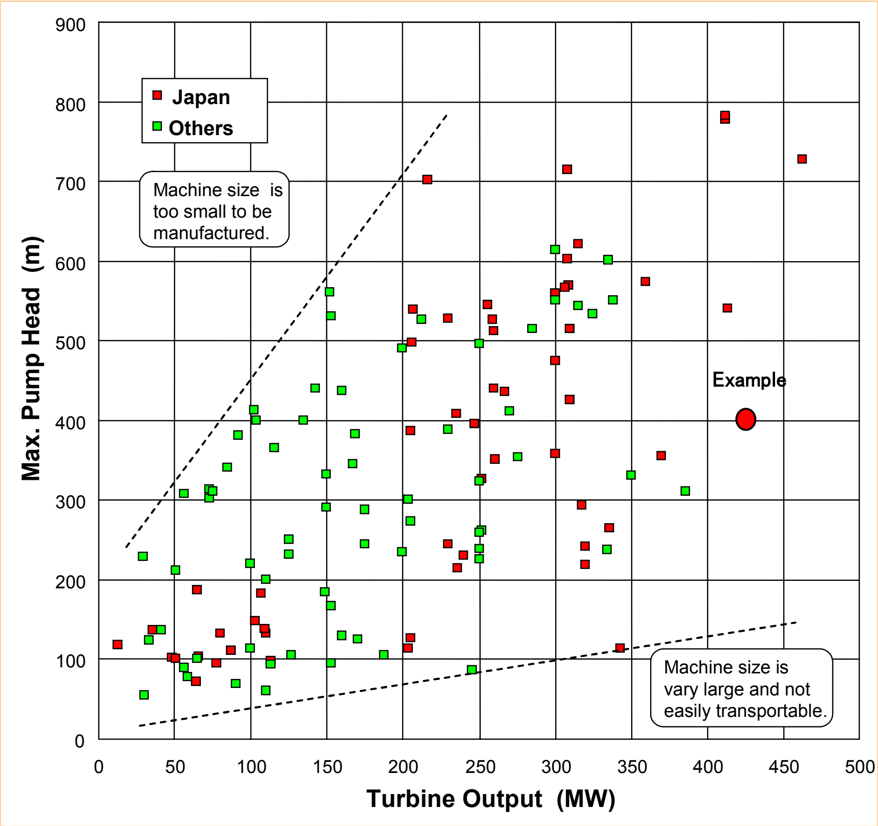
(3) Selection of pump turbine

The designer can choose a type of pump turbine referring to a selection map shown in Figure 19-9 from the pump turbine output and the effective head. But the pump turbine has limits of production and transportation as shown in Figure 19-10 from the manufacture records, and a newly planned plant needs attention so that it may be within these limits. In addition, shown in Figure 19-11 is a record of a tendency to a higher pumping head for pumped storage hydropower schemes.



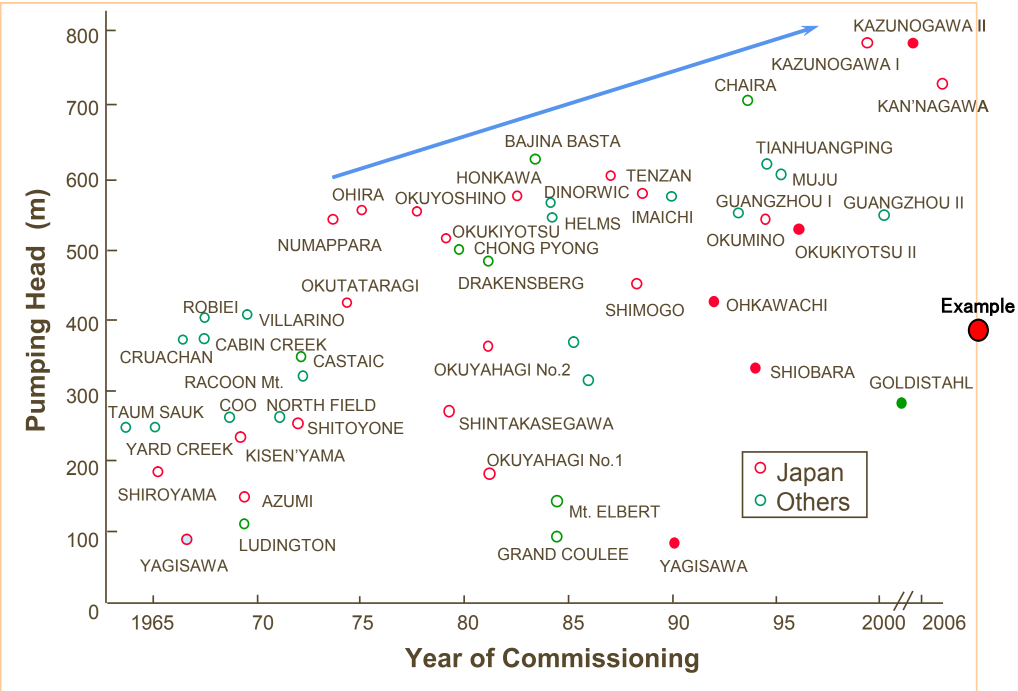
Source: Hydro turbine

Figure 19-9 Coverage of Pump Turbines



Source: Development of Pumped Storage and Its Future Role in Japan for Hydro 2001

Figure 19-10 The Manufacture Records of Francis Type Reversible Pump Turbine



Source: Development of Pumped Storage and Its Future Role in Japan for Hydro 2001

Figure 19-11 Record of High Pumping Head of Francis Type Reversible Pump Turbine

(4) Design of pump turbine

A design example of the vertical shaft single stage Francis type reversible pump turbine for a 400m effective head, 400MW output class is shown below.

Table 19-1 Design Data As an Example

Item	Unit	Plannning data		
Maximum Discharge	m^3/sec	121		
Normal effective head	m	389.4		
Number of unit		2		
Frequency	Hz	50		
Upper reservoir		Maximum	Normal	Minimum
Intake water level (Tailrace water level)	EL. m	515	500	480
Lower reservoir				
Tailrace water level (Intake water level)	EL. m	98	100	102
Turbine operation				
Effective head	m	400	389.4	361.5
Pumpping operation				
Pumping head	m	424	—	388
Pumping water	m^3/sec	—	—	92.3

1) Calculation of the turbine output

Turbine output is calculated from the formula below.

$$P_t = 9.8 \times Q_t \times H_t \times \eta_t = 9.8 \times 121 \times 389.4 \times 0.880 \approx 406{,}000\text{kW}$$

where,

P_t : Rated turbine output

Q_t : Maximum turbine discharge per unit=121m^3/sec

H_t : Normal effective head=389.4m

η_t : Assumed turbine efficiency=88.0%

2) Calculation of maximum pump input

Maximum input of the pumping operation is calculated from the formula below.

$$P_p = 9.8 \times Q_p \times H_p \div \eta_p = 9.8 \times 92.3 \times 388 \div 0.910 \approx 385{,}000\text{kW}$$

where,

P_p : Maximum pumping input

Q_p : Maximum pumping discharge=92.3m^3/sec

H_p : Minimum pumping head=388m

η_p : Assumed pump efficiency=91.0%

3) Calculation of rated revolving speed

Rated revolving speed is calculated from below formula of the maximum specific speed n_{splim} same method of turbine design.

$$n_{sp\lim} = \frac{12{,}500}{Hp + 80} + 13$$

$$N = n_{sp} \times H_p^{\ 3/4} / Q_p^{\ 1/2}$$

$$N = 120 \times f / Po$$

where,

n_{splim}	: Maximum specific speed of the pump turbine (m-m^3/sec)
H_p	: Minimum pumping head=388m
N	: Rated revolving speed of pump turbine (min^{-1})
Q_p	: Maximum pumping discharge=91.2m^3/sec
f	: Power system frequency=50 Hz

$$n_{sp\lim} = \frac{12{,}500}{Hp + 80} + 13 = \frac{12{,}500}{388 + 80} + 13 = 39.7$$

$$N = n_{sp\lim} \times H_p^{\ 3/4} / Q_p^{\ 1/2} = n_{sp\lim} \times 388^{3/4} / 91.2^{1/2} = 365$$

375(min^{-1}) is selected from Table 12-2. The standard revolving speed of a generator (JEC-4001).

4) Efficiency of pump turbine

It is necessary to presume the efficiency of the pump turbine for the calculation of output of the plant and for the economical evaluation. The efficiency is precisely measured by model tests that use an analogue turbine model of about 500mm in runner diameter at a factory in accordance with IEC or JIS standard. In the feasibility study stage, the value of efficiency of a pump turbine is calculated from similar model test data. The pump characteristics of a pump turbine are expressed at the specific speed for the pumping discharge unlike a water turbine. The pump specific speed is calculated from the formula below.

$$n_{sq} = \frac{n \times Q_p^{\ 1/2}}{H_p^{\ 3/4}} = 41.2$$

where,

n_{sq}	: Pump specific speed (m-m^3/sec)
Q_p	: Pumping discharge=91.2m^3/sec
H_p	: Pumping head=389.4m

Therefore, the pump specific speed of the pump turbine of the example plan is 41.2m- m^3/sec which is derived from 41.3m- m^3/sec model data (Model runner data for Kokura PSP project). The efficiency curves are shown in Figure 19-12 and Figure 19-13.

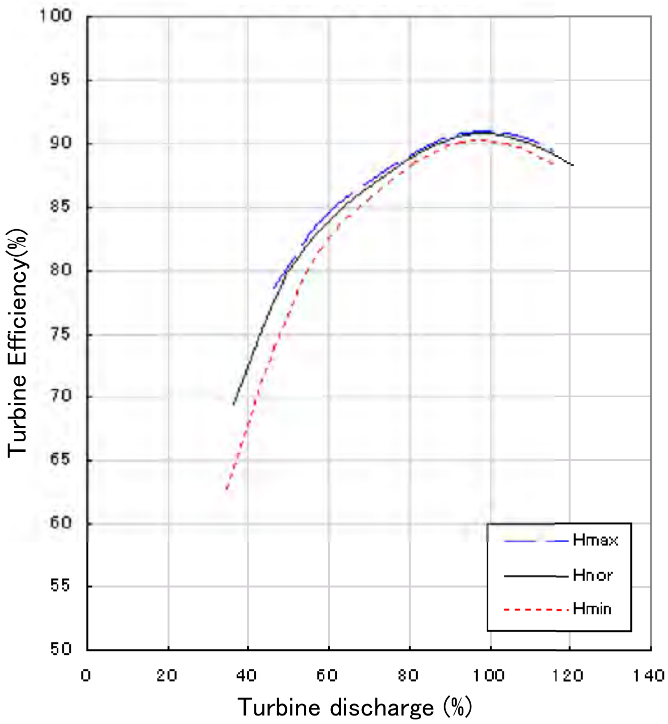


Figure 19-12 Turbine Efficiency Curve of the Pump Turbine

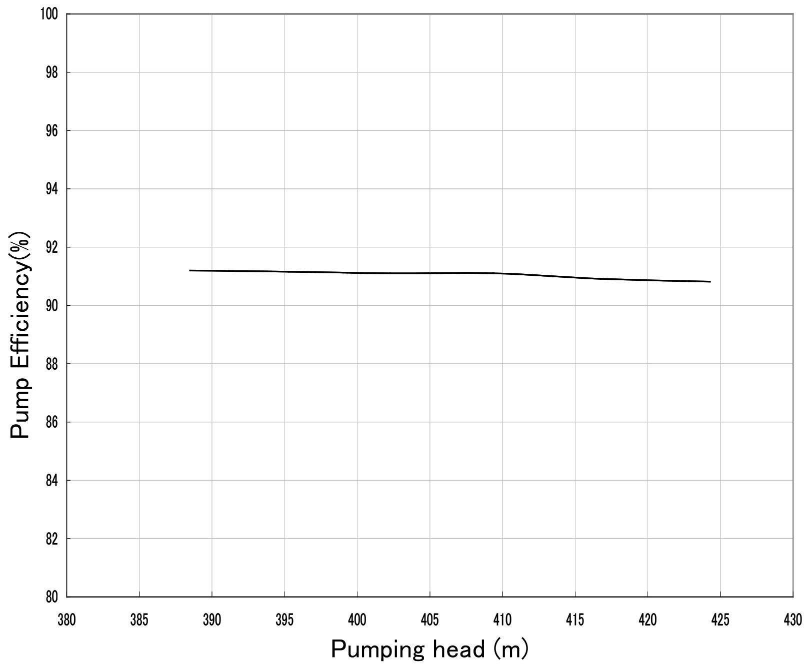


Figure 19-13 Pump Efficiency of Pump Turbine

The designer recalculates (1) the water turbine output and (2) the maximum pump input with the given turbine and pump efficiency. The design results of an example plan are shown in Table 19-2.

Table 19-2 The Design Results for Example

Item	Unit	Plannning data		
Maximum Discharge	m^3/sec	121		
Normal effective head	m	389.4		
Number of unit		2		
Frequency	Hz	50		
Revolving speed	min^{-1}	375		
Upper reservoir		Maximum	Normal	Minimum
Intake water level (Tailrace water level)	EL. m	515	500	480
Lower reservoir				
Tailrace water level (Intake water level)	EL. m	98	100	102
Turbine operation				
Effective head	m	400	389.4	361.5
Output	MW	407	407	362
Efficiency	%	89.4	88.2	88.3
Pumpping operation				
Pumping head	m	424	－	388
Maximum pump discharge	m^3/sec	77.0	－	92.3
Pump input	MW	352	－	385
Pump efficiency	%	90.8	－	91.2

5) Static suction head of the pump turbine

The static suction head of the pump turbine is calculated like a draft head of a water turbine, and the installation level of the pump turbine is determined.

The static suction head of pump turbine is obtained as below.

$$Hs = Ha - Hv - NPSH - Hloss + d$$
$$= 10.41 - 0.323 - 72.928 - 2 + 1.3 \fallingdotseq -63.541$$

where,

Hs : Static suction head (m)
Ha : Atmospheric pressure at pump suction supply (EL. -66m) =10.41 m
Hv : Saturated vapor pressure (at 25ºC) =0.323m
NPSH : Net positive suction head σp× H =0.172×424＝72.928m
σp : Plant cavitation factor as 0.172 referring to the empirical data.
H : Maximum pumping head = 424m
Hloss : Head losses in tailrace at pump discharge =2m
d : Difference between runner vane lower end and runner center=1.3 m

The pump turbine installation level is determined as EL. 34.459 ≈ EL. 34m from an above result with the lower reservoir minimum water level of EL. 98m.

(5) Inlet vale and pump turbine auxiliary equipment

An inlet valve and auxiliary equipment for a pump turbine are basically the same as for a water turbine. In the case of a single stage and two stages reversible Francis type pump turbine, high pressure air is supplied to the pump turbine at the time of pumping start and the water level in the pump turbine is depressed to start pumping. For this purpose, some compressors and air tanks are necessary. The high pressure air is exhausted from inside the pump turbine to the drainage pit, but it is not until the pump turbine reaches to the rated revolving speed to start pumping operation. At the time of the capacity design of the drainage pit, the designer must consider the quantity of water to be released by the exhaust.

A reversible pump turbine and a generator motor turn round in both direction, and oil lift up pumps supplying high oil pressure to the thrust bearings are accordingly provided, which make oil surfaces on the thrust metals and thrust runner in advance.

19.2.2 Generator Motor

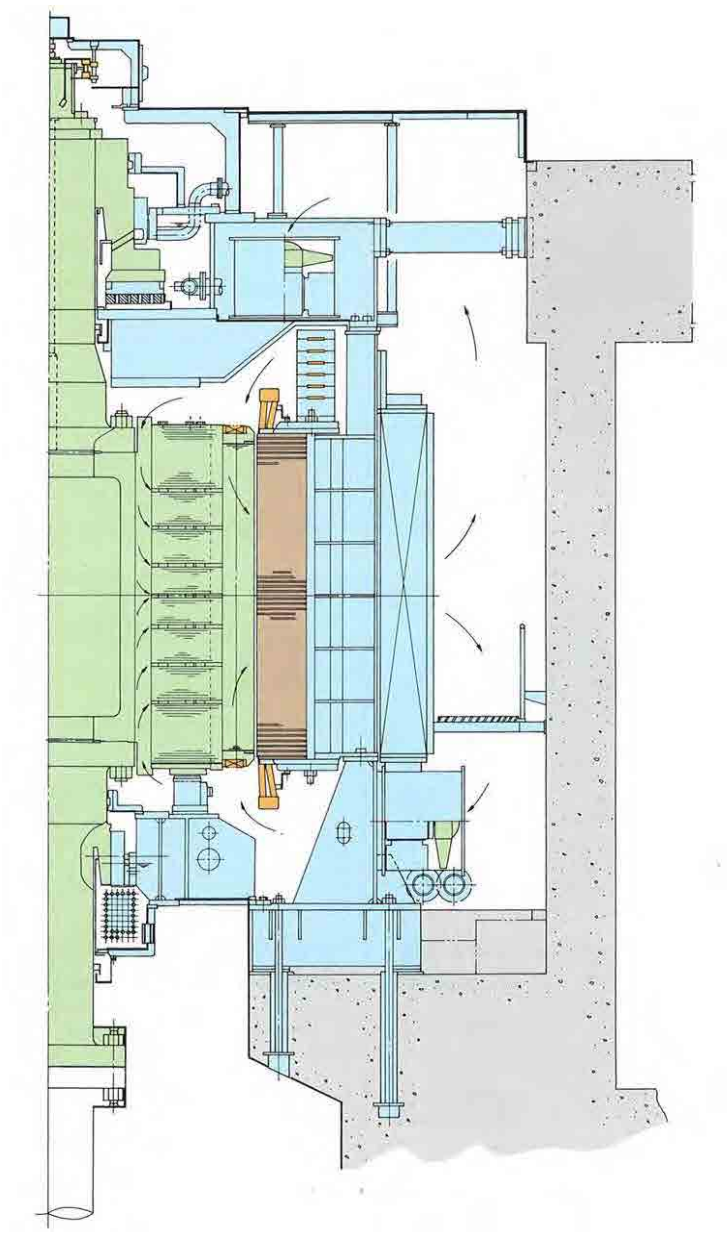
(1) Classification and structure of generator motor

The classification of a generator motor is almost the same as of a generator for a water turbine. A generator motor differs from a generator for a water turbine as follows.

- It works as a synchronous motor when pumping operation.
- It is a reversible rotating machine and the bearings have the same rotating characteristics in either direction.
- Its cooling system has the same characteristics in either direction.
- It endures severer heat cycles than a generator for a peak load.
- It endures strong vibrations at the time of frequent start/stop and pumping.

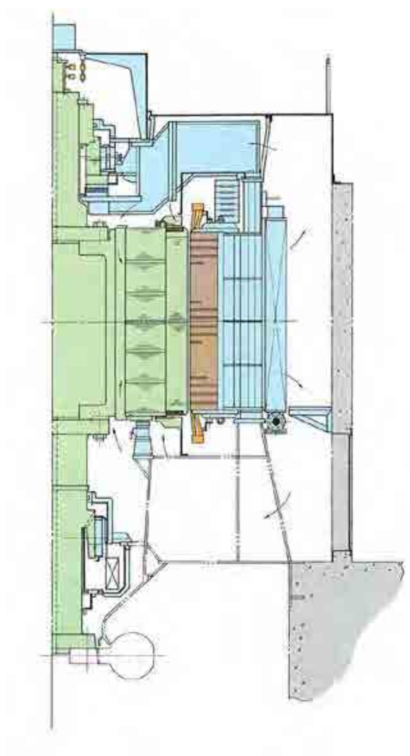
Structurally, a stator and brackets of the generator motor must have sufficient strength and rigidity to withstand the vertical loads of the rotor, external magnetic forces, and horizontal forces caused by vibration forces generated by a reversible pump turbine. A rotor is also rigid enough, for it increases the peripheral speed due to the development of high-speed large-capacity generator motors. To accommodate high-speed large-capacity generator motors, various measures are taken such as increasing an allowable bearings load and employing reversible rotation thrust bearings. Figure 19-14 shows an assembled cross section of a generator motor.

As for an air cooling system for a conventional large capacity generator, a self-ventilating system with fans installed on the rotor was mainly used. For a generator motor, centrifugal fans were used for reversible rotation. As a generator motor becomes high-speed large-capacity, however, the iron core becomes longer and uniform ventilation to the center of the iron core becomes more difficult. Thus, a forced air cooling system with separately installed motor driven fans, and a rim duct ventilation system are now used. Figure 19-15 shows ventilation cooling systems for generator motors.

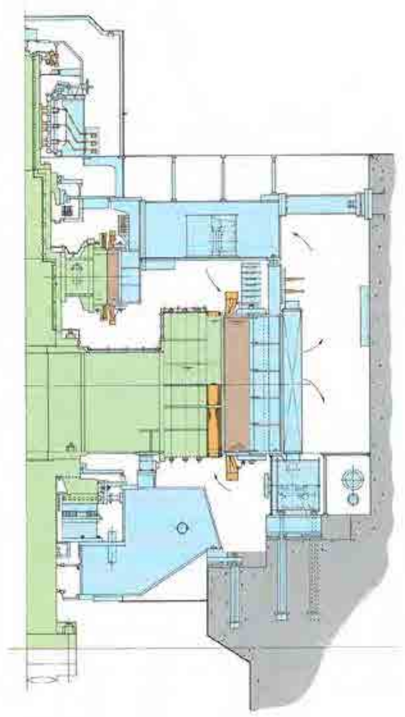


Source: Toshiba Co.ltd. Catalog HYDRO GENERATORS

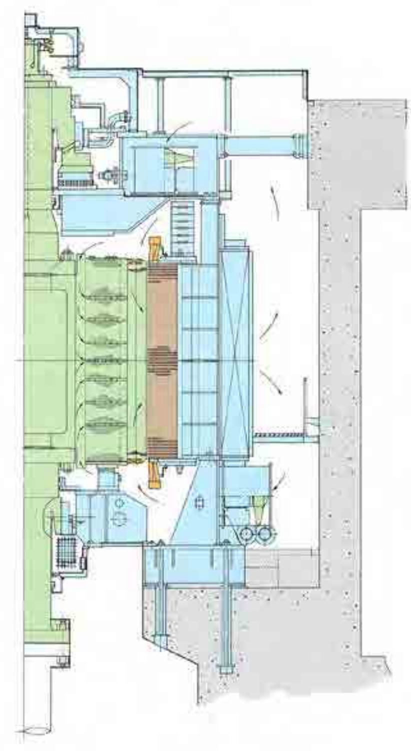
Figure 19-14 Cross Section of Generator Motor



Ventilation by self-ventilation fan



Forced ventilation by motor driven fan



Rim duct ventilation system

Source: Toshiba Co.ltd. Catalog, HYDRO GENERATORS

Figure 19-15 Ventilated Cooling System for Generator Motors

(2) Designing generator motor

There is not big difference in an outline design stage between a generator motor and a generator for a water turbine. Larger maximum capacity between generation operation and motor operation becomes the capacity of a generator motor. Therefore, the maximum capacities are designed including power factors so that the values may not be much different between generator operation and motor operation.

Actually a 500MVA generator motor was manufactured for Kannagawa Pumped Storage Power Plant (Tokyo Electric Power Company) and there are no significant technical difficulties for it. As for the rated revolving speed of a generator motor, there is a production record up to 600min^{-1} combined with a present high specific speed pump turbine for Omarugawa Pumped Storage Power Plant (Kyushu Electric Power Company).

Normally a pumped storage power plant is expected to have the capability to adjust system voltage, and the rated power factor is determined by the operation voltage width of the power system. The rated power factors are adopted, as a design example, 0.90 for generating operation and 0.95 for pumping operation. A generator motor is equipped with a static type (thyristor type) excitation system deriving its energy from the excitation transformer connected to the generator motor main circuit. The generator motor is water cooled by closed air circulation through water-cooled surface air coolers provided outside around the stator frame.

A design example as a pump turbine is shown below.

1) Calculation of rated generator output

The rated generator output is calculated from the formula below.

$$Pg = Pt \times \eta g / Pfg = 407{,}000 \times 0.983 / 0.9 \approx 444{,}000\text{kVA}$$

where,

Pg	: Rated generator output (kVA)
Pt	: Rated turbine output=407,000kW
ηg	: Generator efficiency assumed as 98.3%
Pfg	: Rated power factor taken as 90%

2) Calculation of maximum input of motor

The maximum input of motor is calculated from the formula below.

$$Pm = Pp / \eta m \times Pfm = 385{,}000 / 0.985 \approx 391{,}000\text{kW}$$

where,

Pm	: Maximum motor input (kW)
Pp	: Maximum pump input=385,000kW
ηm	: Motor efficiency assumed as 98.5%

And the maximum motor capacity is obtained as below.

$$Pi = Pm / Pfm = 385{,}000 / 0.95 \approx 412{,}000\text{kVA}$$

where,

Pi	: Maximum motor capacity (kVA)
Pfm	: Rated power factor of motor taken as 95%

3) Rated voltage and rated current

The selection and calculation of the rated voltage and rated current of a generator motor are carried out by the same method of a generator for a water turbine. The result of a design example is as below.

Rated voltage	16.5 (kV)
Rated current	14,416 (A)

4) Excitation system

It conforms to the design procedure of a large capacity generator for a water turbine. A thyristor excitation system, which enables high-speed voltage control, is adopted for an exciter of a pumped storage power plant. Normally, at the time of generating, AVR operation is carried out, and at the time of pumping driving, constant power factor operation by APFR is carried out.

19.2.3 Transformer

Classifications of a transformer and a main transformer of a pumped storage power station conform to those of a large-scale hydropower station. A pumped storage station is constructed underground from a necessary condition for a installation height of a pump turbine. Therefore, a main transformer is often installed underground together with gas insulated switchgears for transmission lines. For this reason, a forced oil water cooled type is often adopted for the main transformer of a pumped storage power plant. In consideration of the transportation and installation in an underground power station, there are many adoption cases of a transformer such as a three single phase combined transformer, a special three phase transformer, or a special three phase six division transformer that has two sets of coils coupled with two transformers.

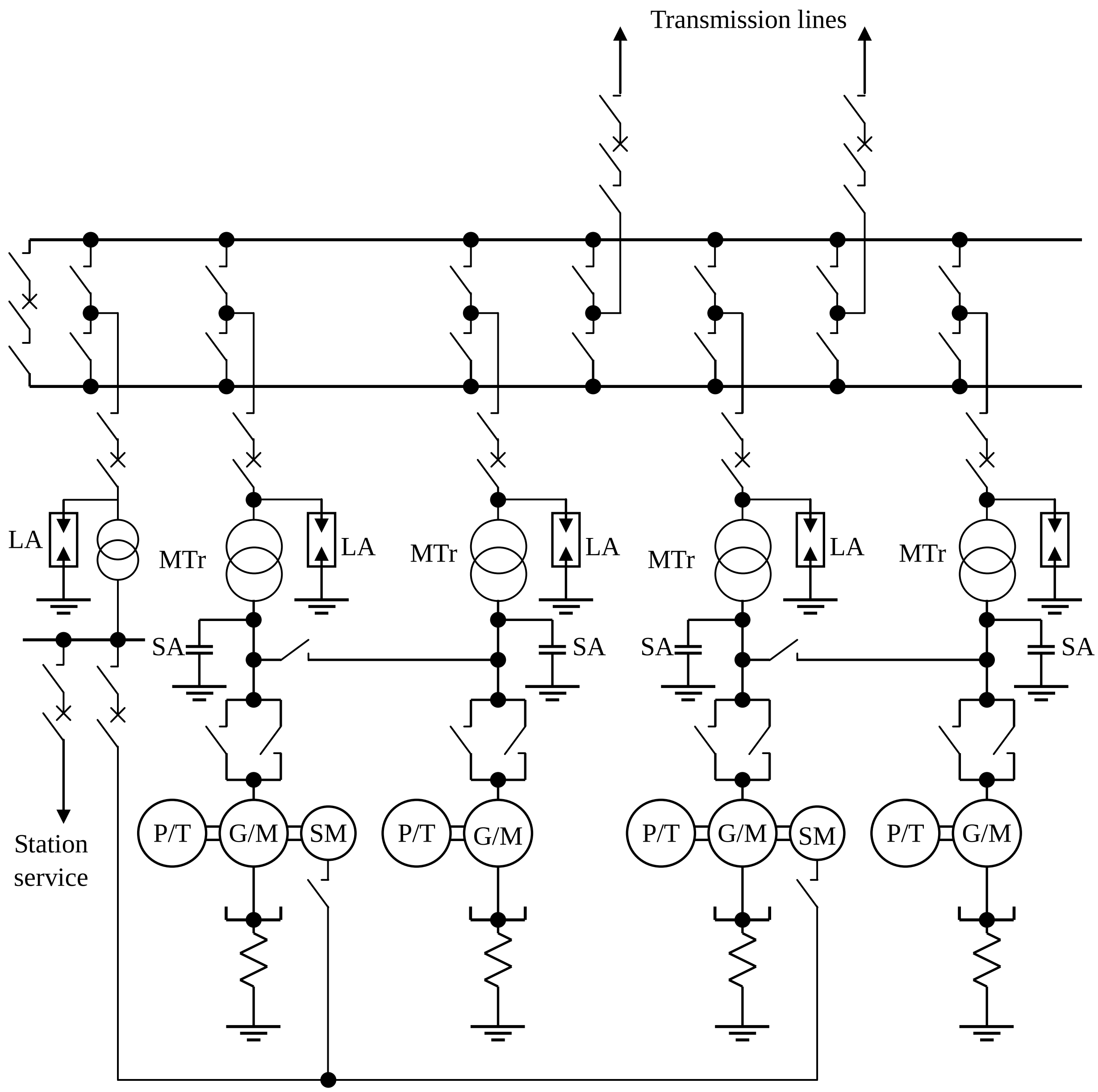
The capacity of the main transformer for a pumped storage power station is not necessarily set the same as the rated capacity of a generator for a general hydropower station. A larger value between the maximum generator capacity and the maximum generator motor capacity plus station service transformer capacity is adopted for the main transformer capacity.

Station service transformers conform to those of a large-scale hydropower station. There are some adoption examples in which the station power service circuits of a pumped storage power plant are designed with a duplex system including station service transformers, bus lines and switchgears by placing importance on the power system.

19.2.4 Main Circuit Connection and Electrical Equipment

(1) Main circuit connection

The following are considered when selecting the main circuit connection for a pumped storage power station: the number of generator motors and their capacities, the number of transmission lines and the connection method, the pumping start method to be mentioned later, the limitations of power plant space, the station service power receiving method and the existence or non-existence of the distribution lines, the construction cost and transportation conditions of transformers and switchgears, the range of power failure caused by in-station accidents, and the safety and ease of repair and maintenance. These aspects are considered in perspective with the viewpoint of reliability and economy of the power plant as well as technology. A pump starting circuit is added to the items for study for a pumped storage power plant. Typical main circuit connections are shown below.



LS: Disconnecting Switch
CB: Circuit breaker
Tr: Transformer
PF: Power fuse
SA: Surge absorber

LA: Arrester
P/T: Pump turbine
G/M: Generator motor
SM: Starting motor

Figure 19-16 High Voltage Synchronous Pumped Storage Power Plant (Synchronous Staring System and Directly Coupled Motor System)

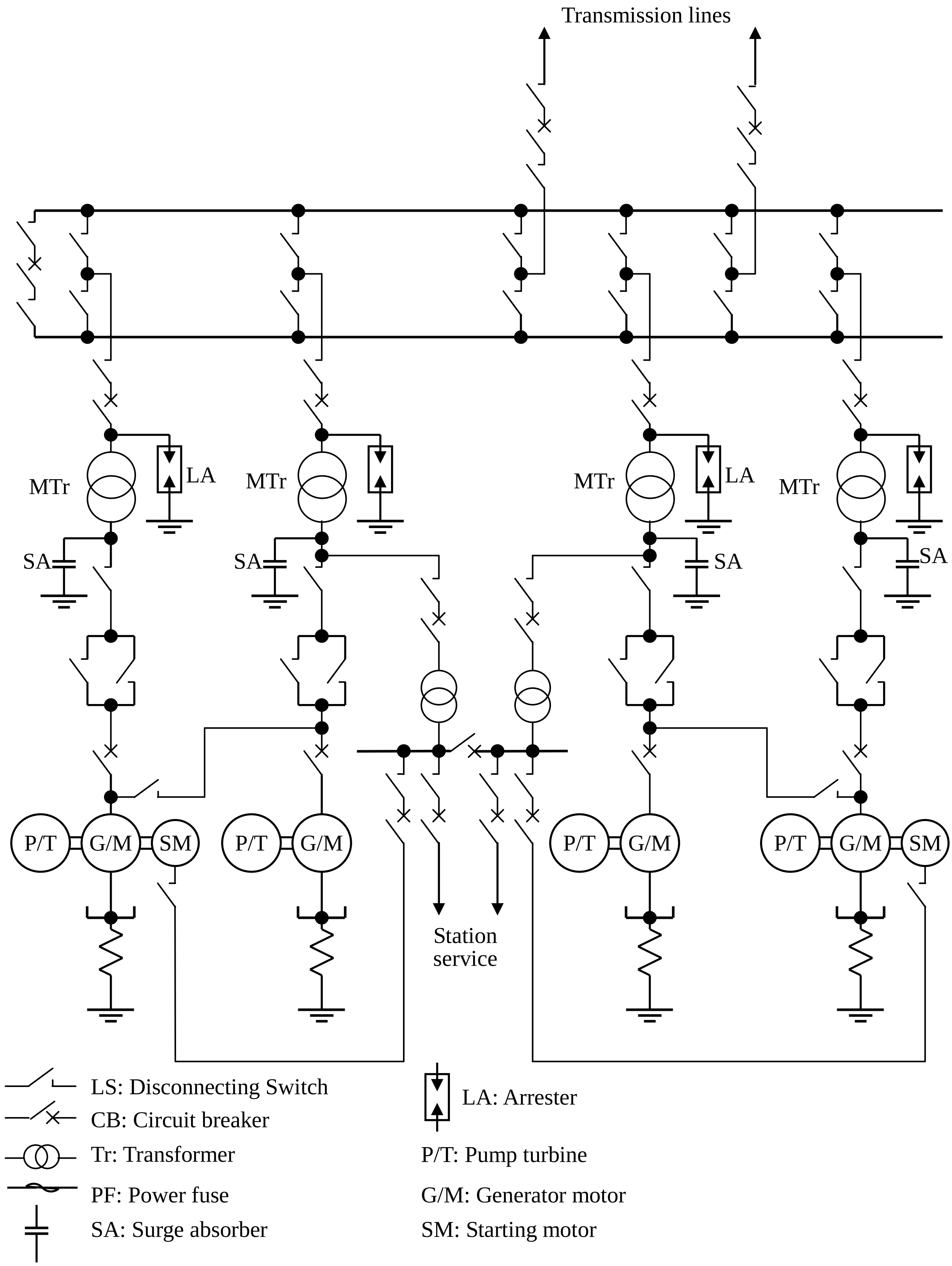


Figure 19-17 Low Voltage Synchronous Pumped Storage Power Plant (Synchronous Staring System and Directly Coupled Motor System)

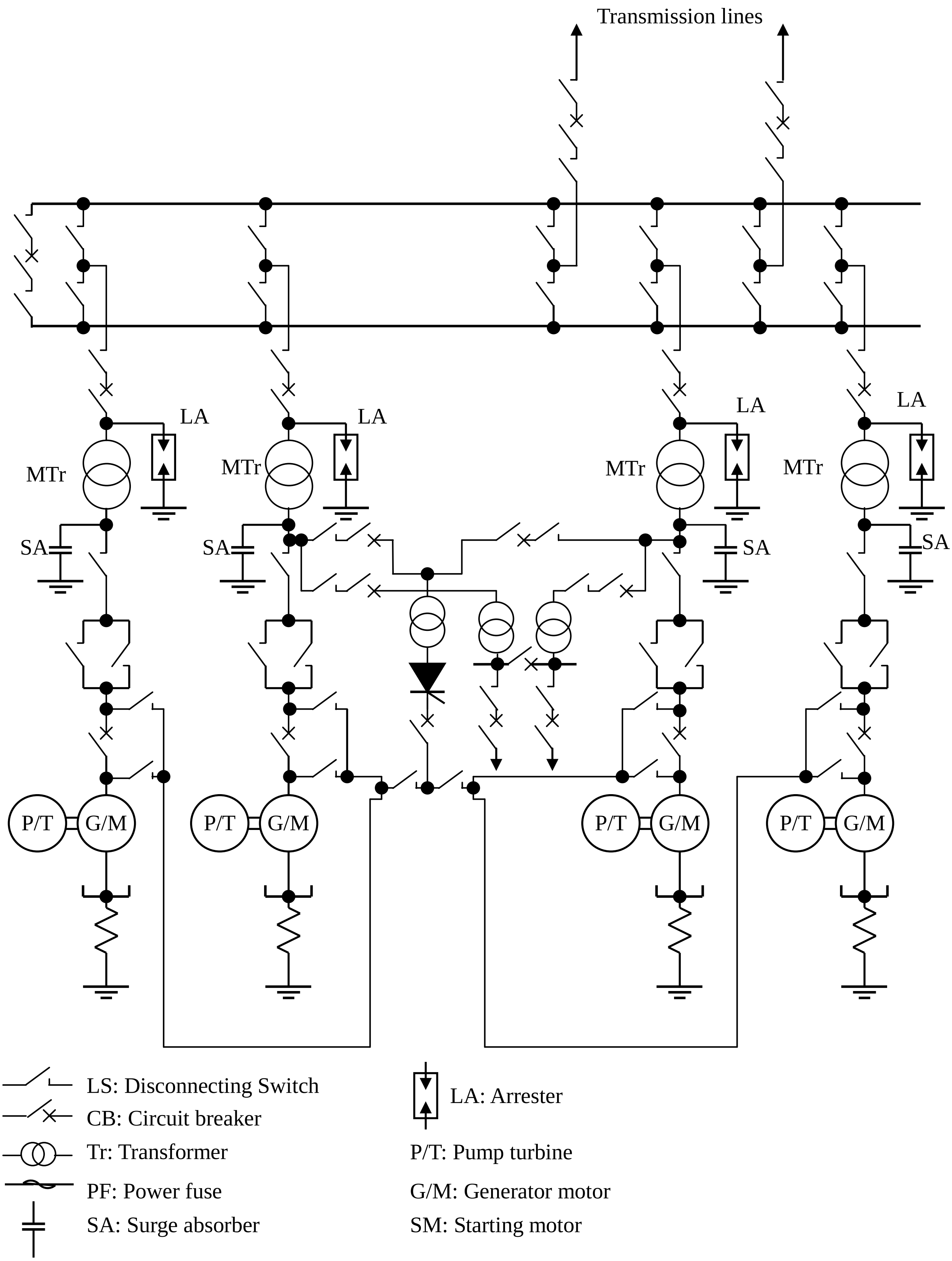


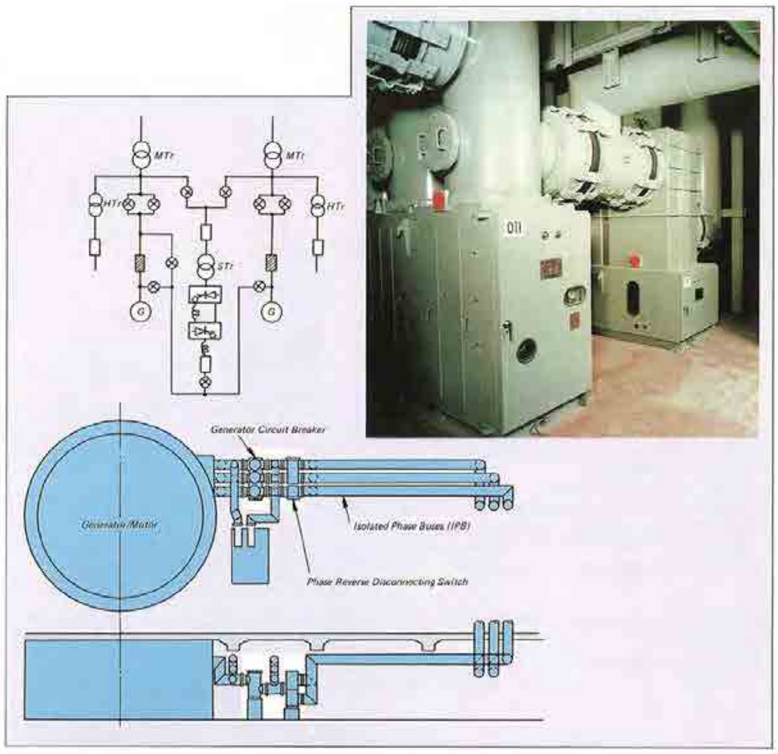
Figure 19-18 Low Voltage Synchronous Pumped Storage Power Plant (Tyristor starter start up system)

19.2.5 Electrical Equipment for Pumped Storage Power Plant

Basically, electrical equipment conforms to that of a large-scale hydropower station. For a reversible type pumped storage power plant, a phase reverse disconnecting switch, which changes the phase turn of the generator motor at the time of generation and pumping, need to be installed. In addition, at a recent pumped storage power station, a large-capacity generator motor is equipped with an electric brake to shorten the required time for a change of mode between generation and pumping operation.

In recent years a breaker for low voltage and large current circuit has been developed. A high voltage synchronization method to connect a generator motor to a power system on the high voltage side had been applied previously, but in recent years a low voltage synchronization method has become the mainstream to connect it on the low voltage side. Due to this development and the adoption, it is possible for a start-up transformer and a station service transformer to be connected to the low voltage side, which results in the reduction in cost and the simplification of the starting circuit and station service circuit. GMCS (Generator Main Circuit Switchgears) which is a package of an electric brake, a phase reverse disconnecting switch, and a low voltage and large current circuit breaker is also developed. An example of GMCS is shown in Figure 19-19.

A pumped storage power station is important in power system operation. Accordingly there are many cases where emergency power equipment is installed to secure the safety power supply at the time of a blackout and enable the black start operation after it. For this emergency power system, a diesel engine generator, a gas turbine generator, a small water turbine generator, etc. are applied.



Source: Hitachi co. ltd. Catalogue Generator Main Circuit Switching Equipment

Figure 19-19 GMCS (Generator Main Circuit Switchgears)

19.2.6 Pump Starting Method

At the start of pumping operation, a generator motor is started as a motor. After having been synchronized with the power system, pumping is started by synchronous motor operation of a generator motor. To reduce the energy on start up, the draft water level is depressed by pressurized air to run the runner in the air, and the generator motor is started as a motor using any of the methods listed in Table 19-3 showing the pump/motor start up methods.

Table 19-3 Pump Operation Start up Methods

Name	Methods	Features
(a) Half voltage start up (Full voltage start up)	The damper coil of the generator rotor is utilized to start the generator motor as an induction machine.	This method is not suitable for starting a large capacity motor because it imposes great shock on the power system when connected to it as an induction machine.
(b) Synchronous start up (Back-to-Back start up : BTB start up)	Two sets of generators and motors are directly connected in the stationary state, with one set started as a turbine and the other as a generator to drive the generator motor using synchronized force.	The last unit requires a separate device for self starting.
(c) Direct coupling motor start up (Pony motor start up)	Directly connected starting induction motor is mounted coaxially with the generator motor to start the generator motor.	This method requires auxiliary devices, such as a starting motor, a starting transformer, and a liquid resistor for speed control.
(d) Tyristor starter start up	A thyristor starter (frequency converter) is equipped to start the generator motor by applying a low frequency up to the rated frequency.	Two or more pump turbines and generator motors can be effectively utilized on sequential start up. The fewer the units, the higher the cost.

19.2.7 Adjustable Speed Pumped Storage System

(1) Outline of Adjustable speed system

An adjustable speed pumped-storage system was planned for the purpose of adjusting motor input in a pumping mode to meet the frequency change of the power system in the nighttime. The following are considered as a method of changing motor input for a pumped storage hydropower station.

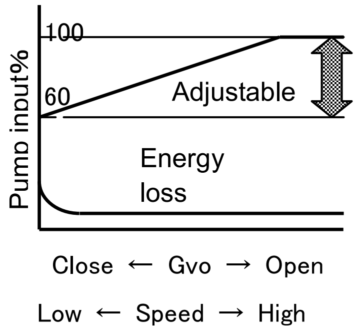
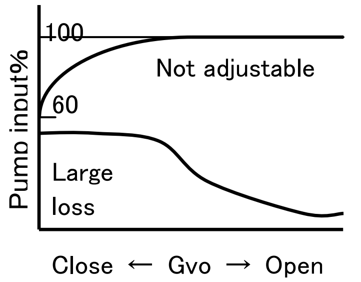
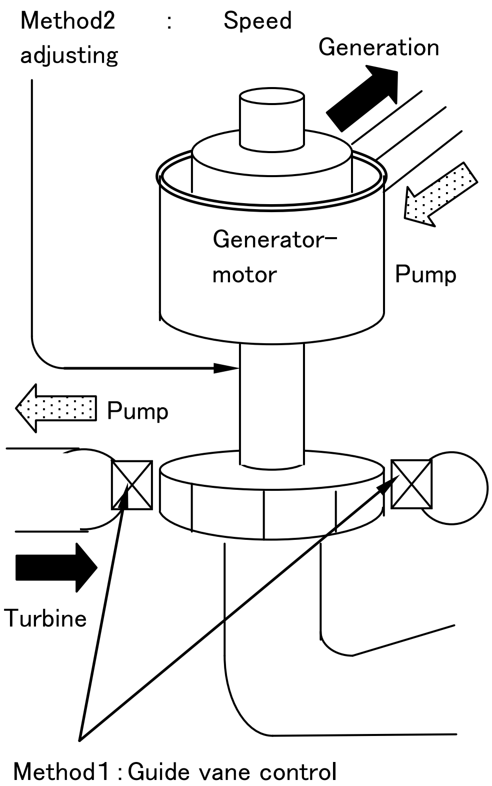
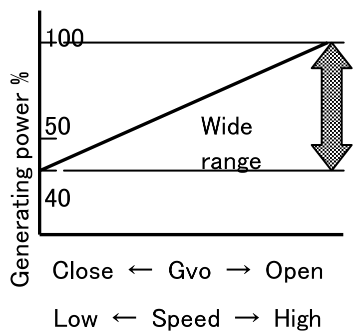
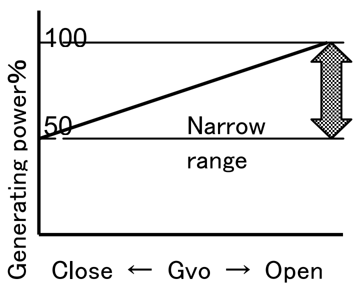
- To adjust the guide vane opening of a pump-turbine
- To adjust the revolving speed of a pump-turbine

As is shown in Table 19-4, a conventional synchronous generator motor can operate only at a constant speed determined by its pole number and system frequency.

Though pump input can be theoretically adjusted by controlling the guide vanes of a pump-turbine, it is not practical because of an increasing energy loss and vibration. In the case of Okukiyotsu

No2 #2unit and a seawater pumped-storage pilot plant, adopted is an adjustable speed pumped-storage system using a double-fed machine, which consists of a frequency converter with GTO (Gate Turn Off Tyristor) and a wound type generator motor, and enables an adjustment of its pumping energy by controlling both the revolving speed and the opening of guide vane. Moreover, during generator operation, an operation range can be controlled close to an optimum point by changing its speed compared with a conventional machine, and then the operating range is widened.

Table 19-4 Comparison between Adjustable System and Conventional System

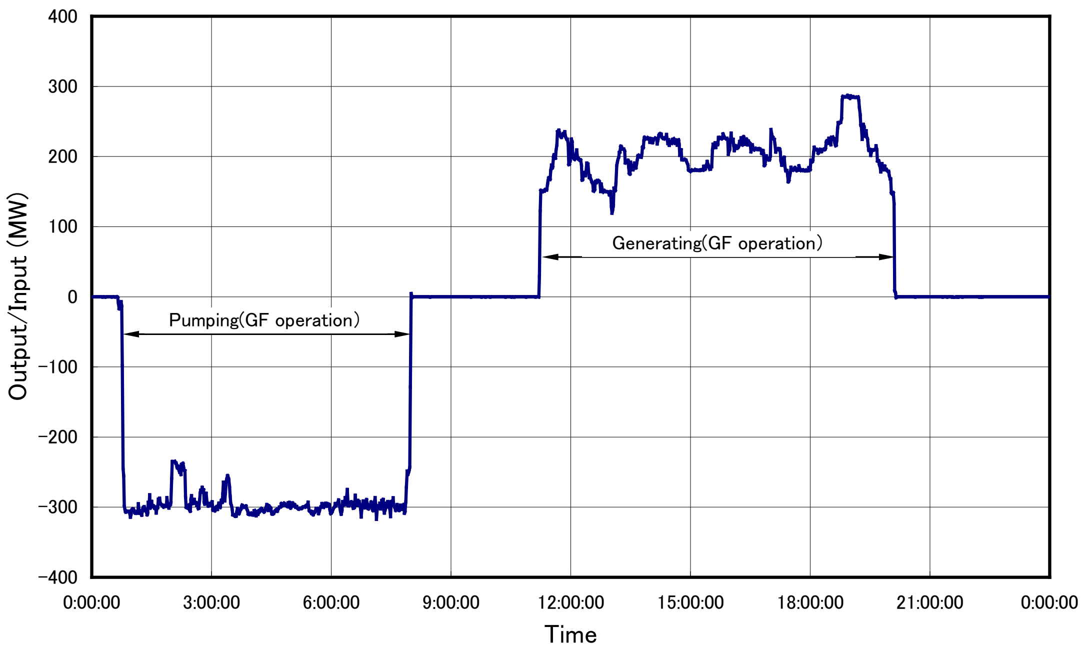
	Adjustable speed pumped–storage plant	Conventional pumped–storage plant	Method of adjusting pump input
Pump operation	Pump input can be smoothly controlled by varying the rotating speed; also energy loss is comparatively small. 	Pump input cannot be controlled without energy loss and large vibration except optimum point. 	
Turbine operation			

The following concrete advantages are expected by adjusting the revolving speed for pumped storage hydropower.

- Pump input can be adjusted, and governor-free operation can be applied in pumping operation.
- Shock to power system at a pump start can be reduced for a smaller input increase.
- Pump input at a pump stop can be reduced
- Higher efficiency can be achieved in generator operation
- Partial load performance can be improved to widen the range.

- High speed control of generator motor output can contribute to the power system stability

An example of GF (Governor Free) operation at the pumping operation is shown in Figure 19-20, and examples in Figure 19-21 and Figure 19-22 show that the adjustable speed pumped storage system reduces the input changes at the time of the pumping start and the pumping stop.



Source: Operation record of Okukiyotsu No2 #2 unit (August 16, 2010)

Figure 19-20 Example of GF Operation at Adjustable Speed Pumped Storage Power Station

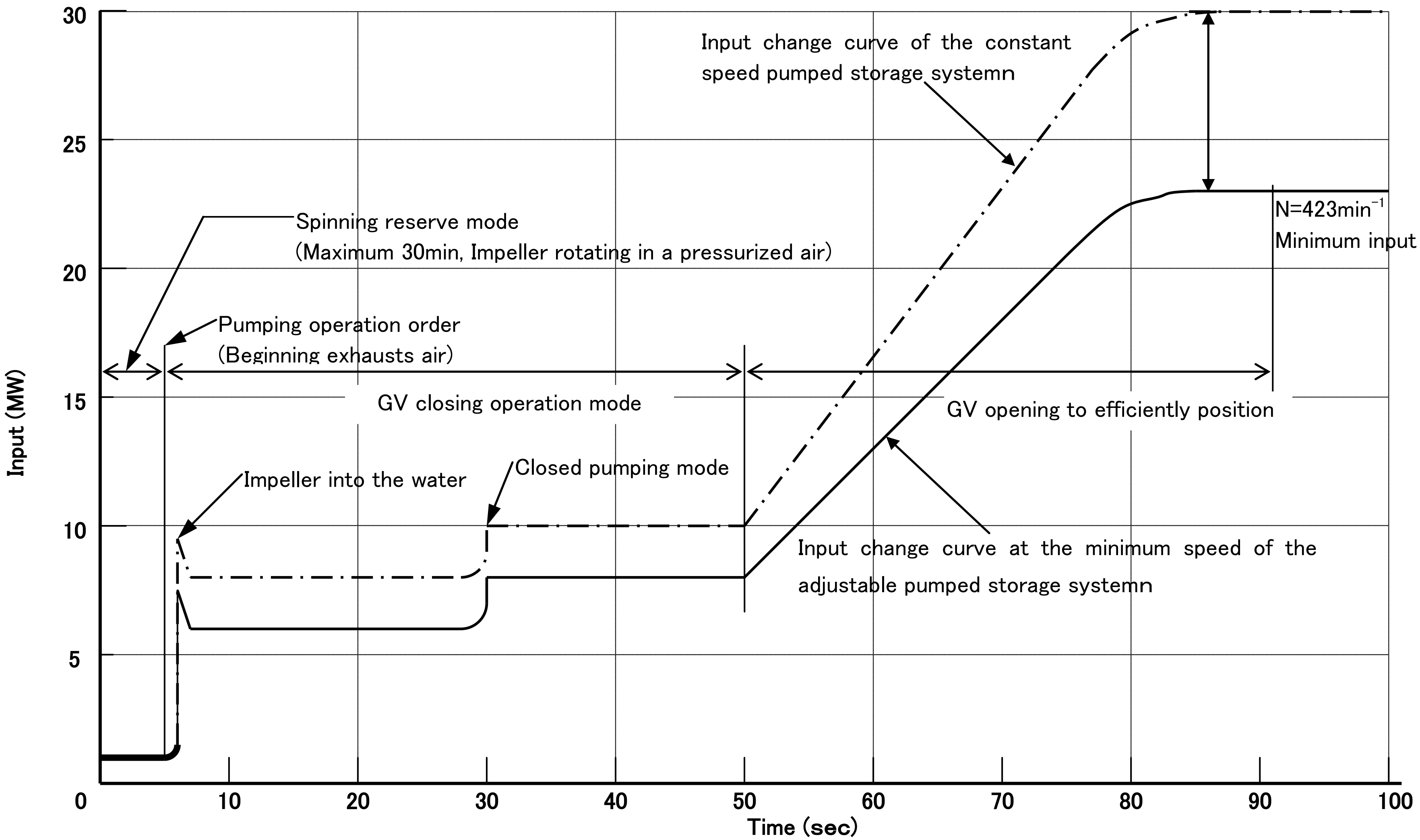


Figure 19-21 Input Characteristics at Pumping Start of Adjustable Speed Pumped Storage Power Station (Okinawa Seawater PSP)

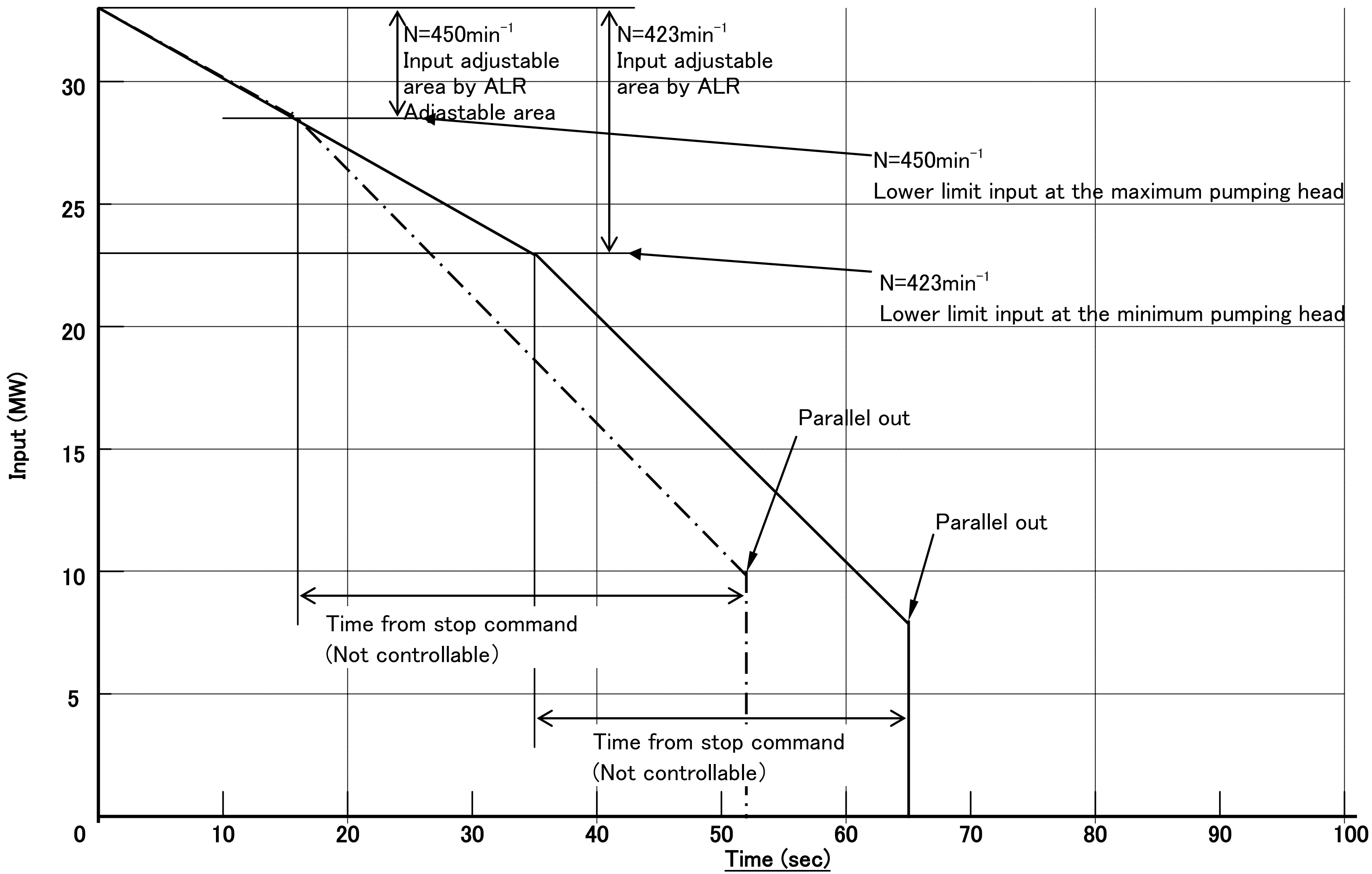


Figure 19-22 Input Characteristics at Pumping Stop of Adjustable Speed Pumped Storage Power Station (Okinawa Seawater PSP)

(2) Adjustable speed system mechanism

The excitation frequency must be controlled in order to synchronize mechanically to system frequency in the event of varying the turbine revolving speed. A double fed system enables this beneficial control of generator-motors; there are two kinds of adjustable speed excitation methods of CYC (Cycloconverter) method and the INV-CON (inverter converter) method. Let us take power system frequency of 60Hz and the rated revolving speed of 450min^{-1} as an example. Assuming that the pump turbine is operated at the revolving speed "n", the rotating magnetic field expressed by the following equation can exist in the air gap by feeding the current of excitation frequency $\pm f_E$ to the field winding of rotor which has "P" poles, the revolving speed of magnetic field observed on the stator shall be $N = n \pm n_E$ ($\pm n_E = \pm 2f_E \times 60/P$).

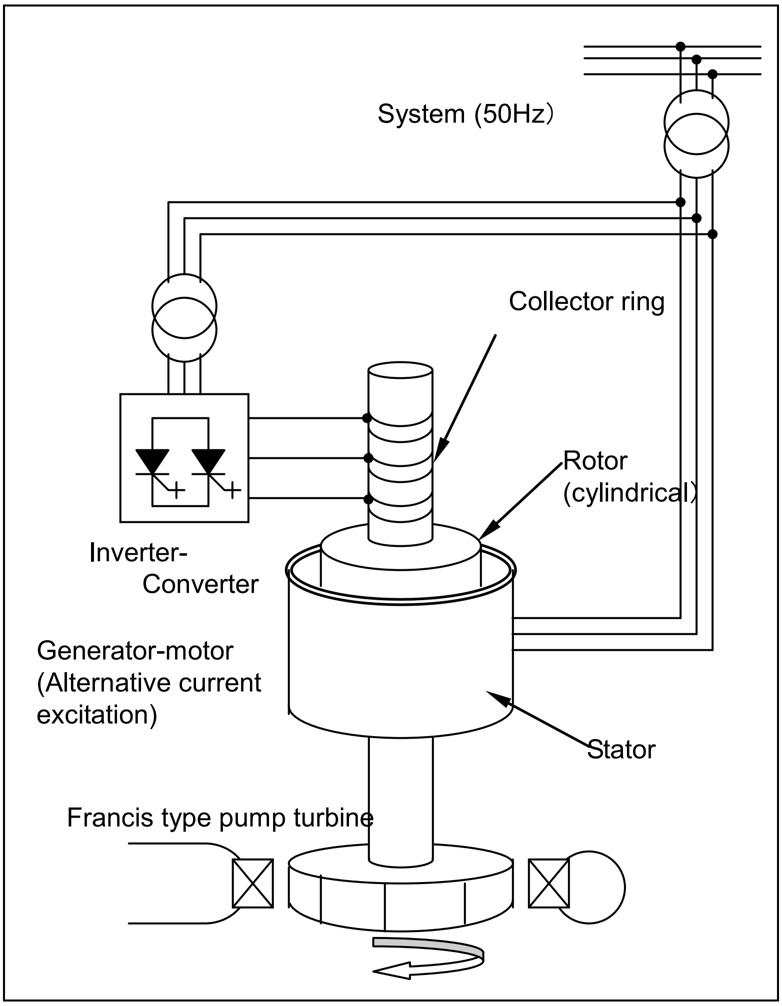


Figure 19-23 Coordinate to System -95%Speed (407min^{-1})

In the case of conventional synchronous machines, the value of n_E shall be equal to zero because of direct current excitation, and then turbine is operated at a speed of "N_0" determined by the system frequency.

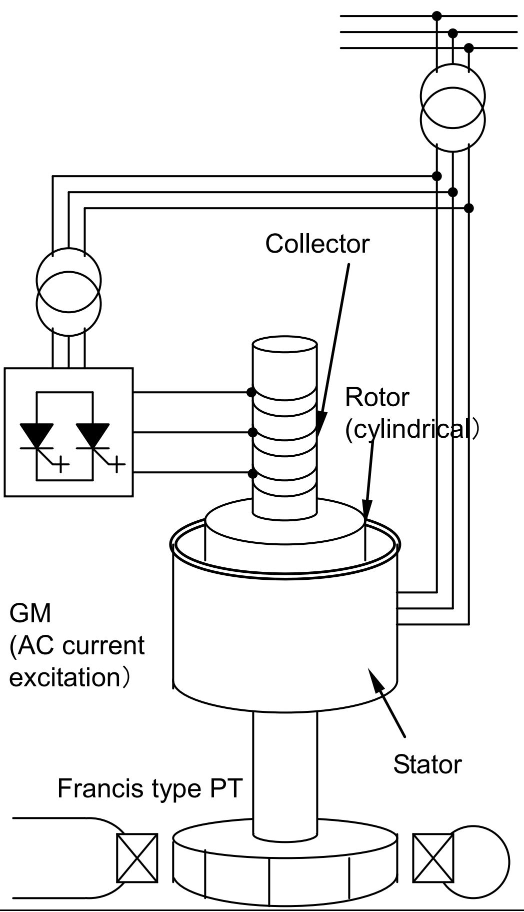
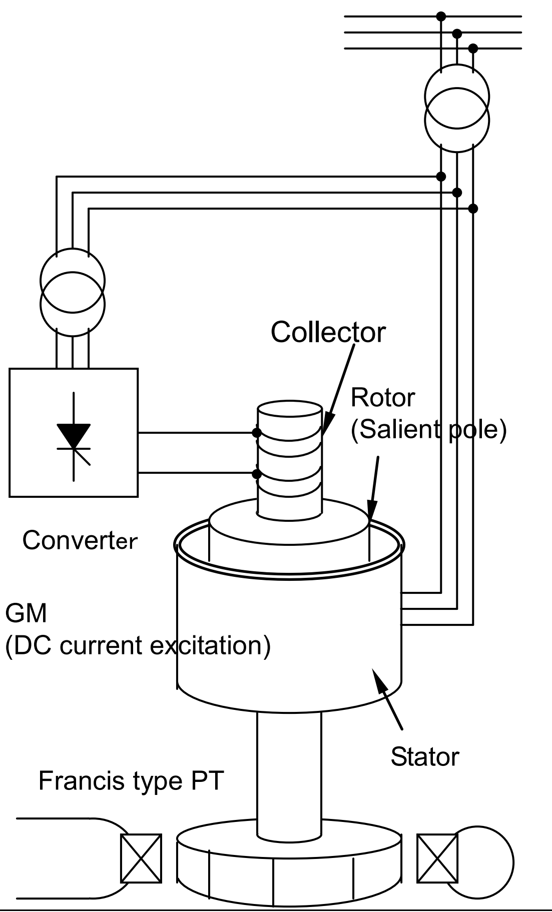
On the other hand, in an adjustable system, the synchronization can be kept by feeding to the rotor winding low frequency excitation current corresponding to "$n_E = N_0 - n$".

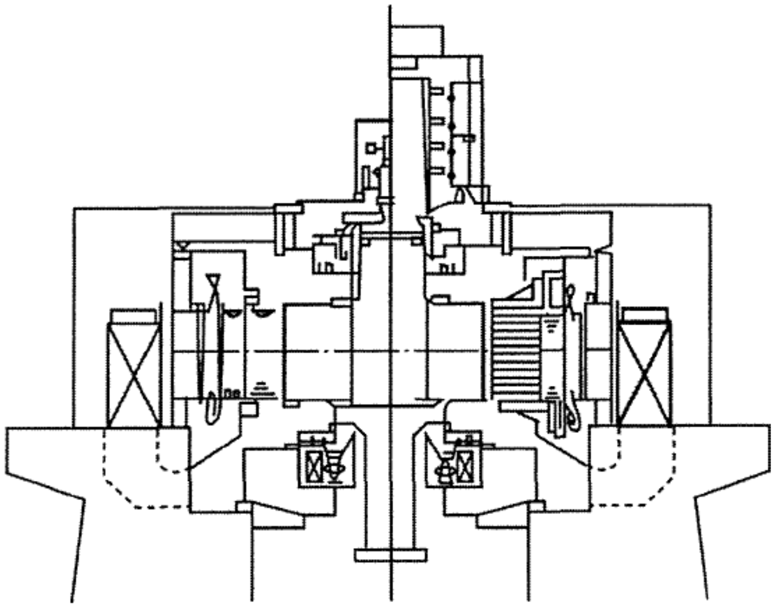
For example, assuming that an adjustable machine is operated at 94% of synchronous speed, the relation among system frequency, revolving speed and excitation frequency is shown below:

Frequency synchronized to system (N_0) = Revolving speed (n) + Excitation frequency (n_E)
60Hz = 56.4Hz + 3.6Hz
450min^{-1} = 423min^{-1} + 27min^{-1}

The comparison between a conventional system and an adjustable system is shown in Table 19-5.

Table 19-5 Comparison between Conventional System and Adjustable System

	Adjustable speed pumped-storage	Conventional pumped-storage
System Configuration		
Stator current	Three phase 60Hz	Three phase 60Hz
Rotor current	Three phase	Direct current
Speed	60Hz – 3.6Hz → 423min^{-1} 60Hz + 3.6Hz → 477min^{-1} } Variation range	450min^{-1}(Constant)
Excitation system	PWM GTO Inverter Converter	Thyristor etc.
Governor free operation	Both pumping and generating mode	Only generation mode
Automatic Load Regulator operation	Same as above	Same as above
Automatic Frequency Control	Same as above	Same as above
Partial load	About 40% of Full load	About 50% of Full load
Pump start	Reducing pump input at pump start Converter is used by starter as well as excitation system	Large pump input at pump start Another starter is necessary
Performance at system fault	- To contribute to system stability - Over voltage protection is necessary for secondary winding in case of fault current flowed into GTO inverter-converter.	- Considerably disturbed by system fault - Field windings and exciter of salient pole machine are not affected by system fault current
Capital cost	Rotor and excitation system is expensive. Approximately 3-5% of project cost.	-



Conventional Rotor

Adjustable Speed Rotor

Figure 19-24 Comparison Between Conventional Generator and Adjustable Speed Generator Motor Configuration

(3) Outline of excitation system

There are two typical adjustable speed methods which have been practically used for a pumped storage hydropower plant, a CYC (cycloconverter) method and a INV-CON (Inverter-Converter) method. These circuit diagrams are shown in Figure19-25 and Figure19-26. Characteristics of both methods are described below. And Table 19-7 shows a list of adjustable pumped storage power plants in the world including the plants commissioned, under construction and under planning.

1) Var compensation

CYC needs a large amount of reactive power and its capacity amounts to 1.7 to 1.8 times of its own capacity. This reactive power must be supplied from the generator motor itself or another facility, and then the generator capacity is comparatively larger than a conventional system. On the other hand, INV-CON can be operated at 1.0 of power-factor so that no reactive power should be supplied and the capacity of generator-motor is smaller than that of Cycloconverter system.

2) Reliability

Adjustable speed pumped-storage hydropower plants using CYC have long experiences at such as Yagisawa Pumped storage power plant (85MVA: Tokyo Electric Power Co.), Ohkawachi pumped storage power plant (395MVA Kansai Electric Power Co.), Shiobara pumped storage power plant (360MVA: Tokyo Electric Power Co.), Goldisthal pumped storage power plant (331MVA: Germany), and Omarugawa pumped storage power plant #4, #1unit (345MVAeach, Kyushyu Electric Power Co.).

Though the history of GTO system is comparatively short, GTO using inverters have been primary applied for motor control such as train motors, mill motors and pump motors for water service. In recent years, as an improved semiconductor technology, GCT element and the IGBT element of a large-capacity high voltage types are developed, and these begin to be adopted in a system of the second generation. Adjustable speed pumped storage hydropower plants using GTO converter system have been put into operation at Takami pumped storage power plant (105MVA: Hokkaido Electric Power Co.), Okukiyotsu No.2 pumped storage power plant (345MVA: Electric Power Development Co.), Avce (195MVA Slovenia), and Omarugawa #3unit (340MVA Kyushyu Electric Power Co.). They have been successfully operating for several years. Both theses systems have sufficient reliability for a practical use.

3) Operation performance in disturbed system condition

CYC supplies the alternated field current amounting a few or a little more Hz slip frequency to the rotor winding of generator-motor directly converting from the system frequency power. For this reason, it is difficult to control the output voltage, frequency, phase, and current in the case of fluctuating system voltage at the time of a system fault, in particular a drop of voltage. Moreover, in the case of a system fault, a current including transient DC component tends to flow from the rotor winding to CYC due to the supply of a fault current from a generator motor. CYC cannot flow its current and another device such as “voltage clipper control”, “OVPS (Over Voltage Protection System)”, “current canceling control (HITACH’s patent)” needs to be applied to restrict high voltage. If OVPS is operated, CYC stops for several hundred milliseconds until the completion of a reset, which may reduce an improvement effect on system stability at a system fault. On the hand, INV-CON has a DC link condenser between a converter and an inverter. Power stored in this condenser is supplied by an inverter to the rotor winding. Being different from CYC system, short time voltage fluctuation of system does not disturb the control of GTO Inverter, because DC link voltage is not directly influenced by the fluctuation of line voltage. Moreover, at a system fault, the fault current from the rotor winding is charged in a DC link condenser through a flywheel diode equipped with GTO module. If a fault occurs far from the power plant, DC link voltage can be controlled by the chopper against a direct voltage rise, and quickly restart control by an inverter can be used, which enable to minimize interrupted control and contribute to the system stability at a system fault.

4) Design for Harmonics Filter

Twelve phase CYC supplies 12n±1st harmonics current to AC power source as well as their sidebands. Accordingly the design of filter is very complicated considering these sidebands as well. On the other hand in the case of INV-CON, only harmonics oriented from PWM (Pulse Width Modulation) modulation frequency are produced, which enable to adopt simple design of filter compared to CYC. Moreover, for example, Neutral Point Clamp connection of GTO module, which is adopted at Okukiyotsu No.2 pumped storage power plant, enables to double the harmonics frequency to be so beneficial that the filter can be eliminated as actually seen at Okukiyotsu No2. In the case of a sea water pumped-storage pilot project, 4n and 10n (“n” is PWM modulation frequency) harmonic filters are installed at low voltage terminal of a main-transformer as the result of computer simulation and total harmonics distortion is achieved at less than 1% at high voltage terminals of transformer.

5) Installation space

The installation space of CYC is considerably large than that of conventional excitation device because of its large capacity. INV-CON is similar in size to CYC.

6) Alternative for an example project

The first case study for 30MW class adjustable speed system to be adopted in a seawater pumped-storage pilot plant is shown below between CYC and INV-CON. Though in both cases the system performance in control ability is at a sufficient level for a practical use, INV-CON is actually selected by reason of space saving, less total loss, 1.0 of power-factor, less generator motor capacity, and so on.

Table 19-6 Comparison between INV-CON and CYC

Item	INV-CON	CYC system
Rating	GTO Inverter： 3.96MVA-915V-2500A GTO Converter： 2.8MVA-860V-1880A	7.45MVA-1520V-2830V
Installation space	100%	120%
Energy loss	100%	140%
Harmonics distortion	Approximately 1.0% (with 6.5MVA Filter)	Approximately 0.5% (with 7.5MVA Filter)
Element number	GTO: 30, Diode: 30	Thyristor: 72
Controllability	Individual control for active and reactive power and high speed control can be made	Individual control for active and reactive power and high speed control can be made
Generator capacity	Comparatively small	Comparatively large
Gate block interruption	Comparatively short	Comparatively long

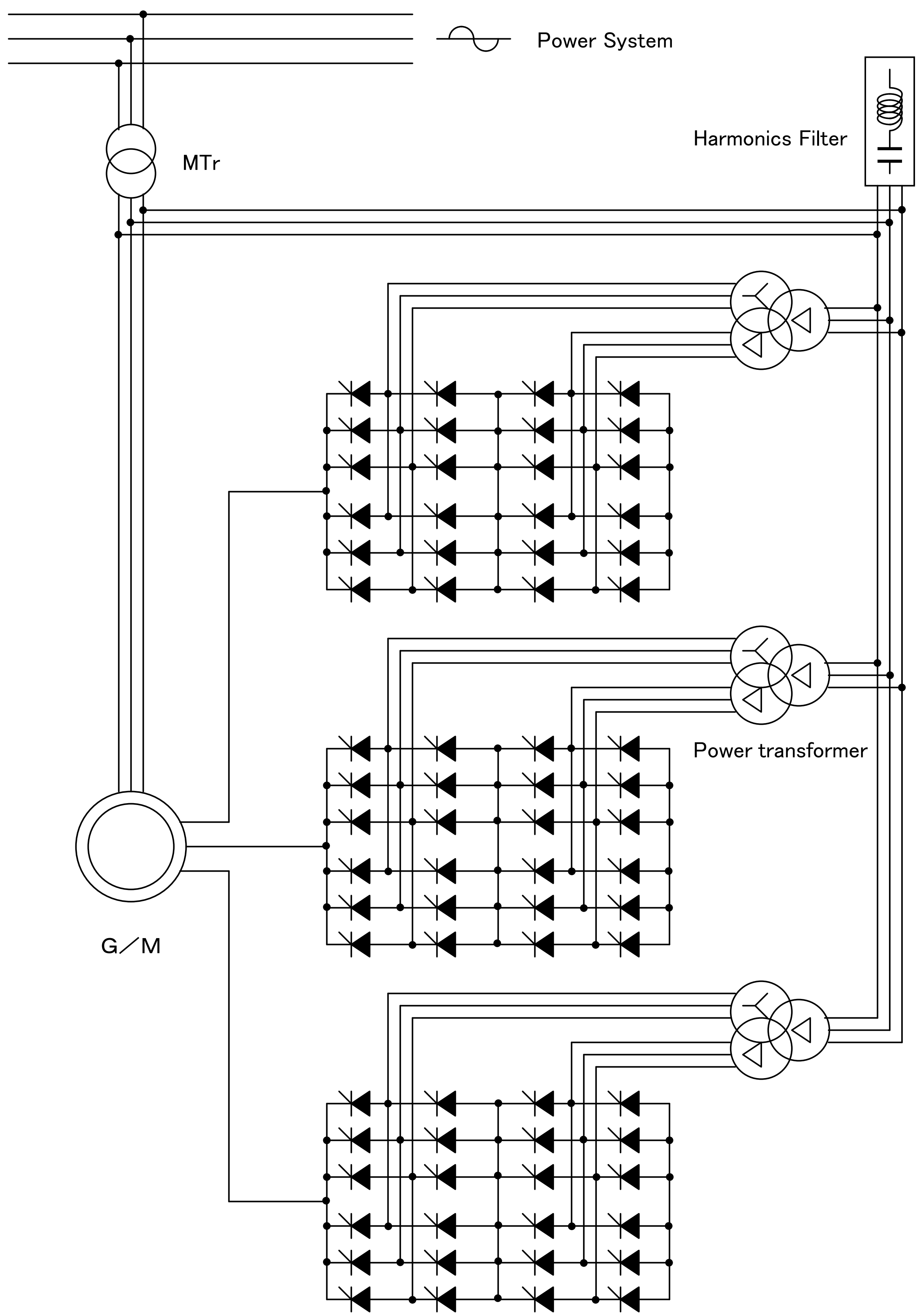


Figure 19-25　CYC (Cycloconverter) Adjustable Speed System

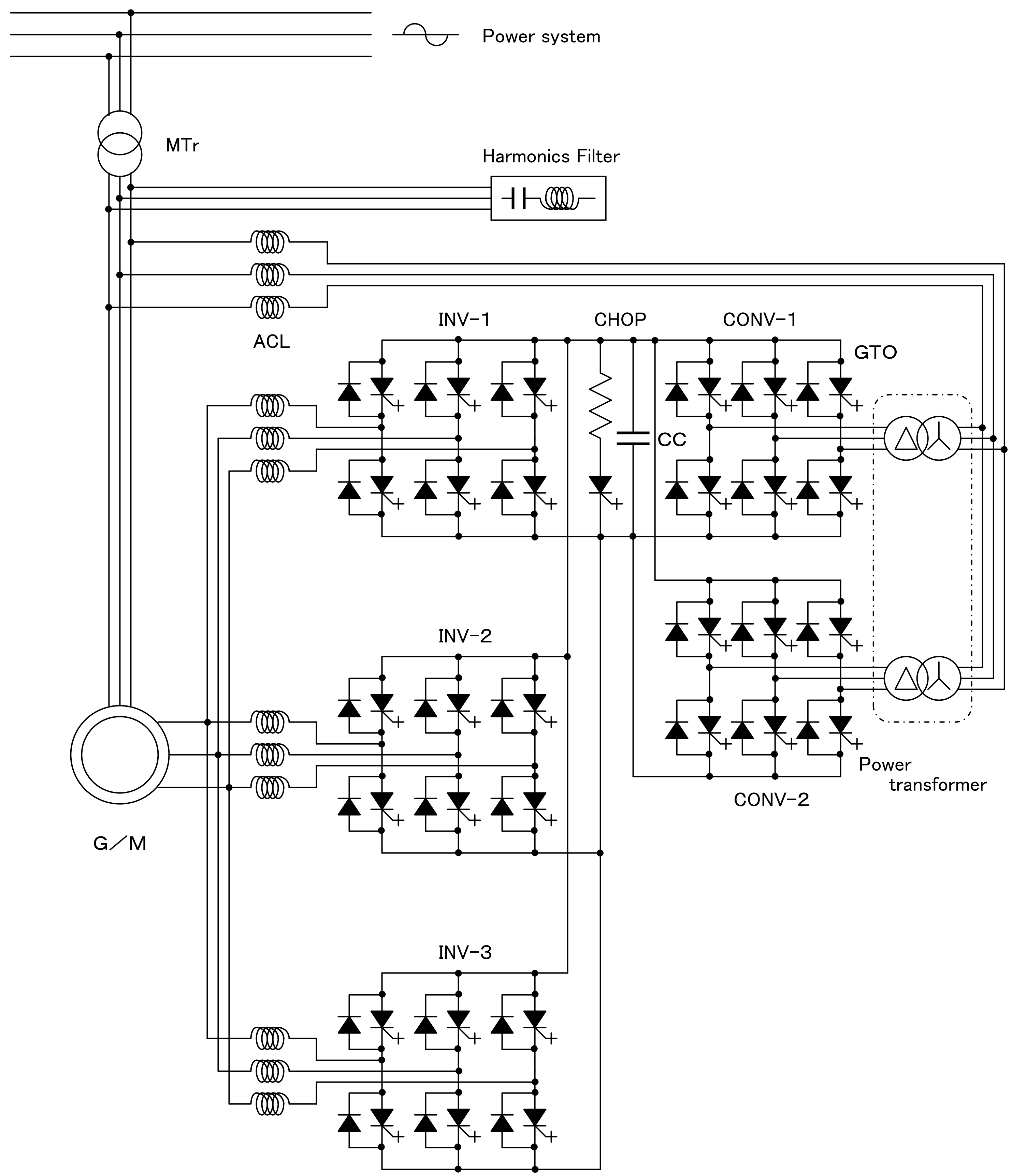


Figure 19-26　INV-CON (GTO Inverter-Converter) Adjustable Speed System

Table 19-7 List of Adjustable Pumped Storage Power Plant

No.	Name	Output MVA/MW	Speed min^{-1}	Method	Owner	Manufacturer	Commissioning year	Country
	Commissioned							
1	Yagisawa PSP #2	85/82	130-156	CYC	Tokyo Electric Power	Toshiba	1990	Japan
2	Takami PSP #2	105/140	208-254	INV-CON	Hokkaido Electric Power	MHI/MELCO	1993	Japan
3	Ookochi PSP #4	395/388	330-390	CYC	Kansai Electric Power	Hitachi	1993	Japan
4	Ookochi PSP #3	395/388	330-390	CYC	Kansai Electric Power	Hitachi	1995	Japan
5	Shibara PSP #3	333.3/341	345-405	CYC	Tokyo Electric Power	Hitachi / Toshiba	1995	Japan
6	Okukiyotsu No2 #2	345/340	407-450	INV-CON	J-POWER	Toshiba	1996	Japan
7	Okinawa Seawater PSP	31.5/31.8	423-477	INV-CON	J-POWER	Toshiba	1999	Japan
8	Goldisthal PSP (2units)	331/330	300-345	CYC	Vattenfall	VOITH/ANDRITZ	2002	Germany
9	Omarugawa PSP #4	345/330	576-624	CYC	Kyusyu Electric Power	Hitachi	2007	Japan
10	Omarugawa PSP #3	340/330	576-624	INV-CON	Kyusyu Electric Power	MHI/MELCO	2008	Japan
11	Omarugawa PSP #1	345/330	576-624	CYC	Kyusyu Electric Power	Hitachi	2010	Japan
12	AVCE (1unit)	195/185.3	576-626	INV-CON	SENG	MHI/MELCO	2010	Slovenia
	Under construction							
13	Omarugawa PSP #2	340/330	576-624	INV-CON	Kyusyu Electric Power	MHI/MELCO	2011	Japan
14	Okutataragi PSP#2	Under design	285-315	CYC	Kansai Electric Power	Hitachi	2013	Japan
15	Okutataragi PSP#1	Under design	285-315	CYC	Kansai Electric Power	Hitachi	2014	Japan
16	Kyogoku PSP #1	222/230	475-525	INV-CON	Hokkaido Electric Power	Toshiba	2014	Japan
	Under planning							
17	Kazunogawa PSP #3	475/460	480-520	INV-CON	Tokyo Electric Power		After 2017	Japan
18	Kazunogawa PSP #4	475/460	480-520	INV-CON	Tokyo Electric Power		After 2017	Japan
19	Nant de Drance (4 units)				Alpiq, SBB, FMV	ALSTOM	2016	Switzerland
20	Limmern (4 units)				Linth-Limmern AG	ALSTOM	2016	Switzerland
21	Venda Nova 3							Portugal
22	Kozjak							Slovenia
23	Kühtai 2							Österreich

MHI : Mitsubishi Heavy Industry MELCO : Mitsubishi Electric Corporation

Reference of Chapter 19

[1] Guide manual for Development Aid Programs and Studies of Hydro Electric Power Project, New Energy Foundation, 1996

[2] Hydro turbine (in Japanese), Turbo machinery Society of Japan, 1991

[3] Electrical Engineering Handbook 6th Edition (in Japanese), IEEJ

[4] Development of Pumped Storage and Its Future Role in Japan for Hydro, 2001

[5] Catalogue HYDRO GENERATORS, Toshiba Corporation

[6] Catalogue Generator Main Circuit Switching Equipment, Hitachi Corporation

Part 5

Operation and Maintenance

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Chapter 20

Operation and Maintenance

Chapter 20 Operation and Maintenance

20.1 General

In the operation and maintenance of hydropower plants, the manual should describe the essentials of operation, maintenance and inspection for civil structures, and electro-mechanical facilities. Operation and maintenance should be conducted based on the manual.

Operation and maintenance personnel are required to observe the following:

(1) Follow all rules at all times. Operate and maintain all facilities systematically and efficiently.

Make every effort to improve both the facilities and skill of operators toward achieving highly efficient operation.

(2) Be totally familiar with the structure and performance of all civil structures, electro-mechanical facilities, and other equipment and facilities in the hydropower plant. Be prepared to implement the necessary countermeasures against any possible accident.

(3) Be constantly aware of the conditions of each piece of equipment, device and facility.

Promptly report to the chief operator any abnormality, interruption or problem that are encountered and observed. Where emergency measures are required concerning the operation, provide the appropriate countermeasures such as the removal of defective part(s) or conduct troubleshooting operation to restore normal operation as soon as possible.

(4) Follow the safety standards at all times. Prevent accidents which may result in an injury or a death. Improve the equipment and facility as required.

20.2 Operation

20.2.1 General

A power plant should be operated based on the operations manual which specifies the operation procedures during normal operation and the countermeasures required in the event of any abnormality.

Operators are required to observe the following:

- ➢ Be constantly aware of the conditions of the transmission system and of the load both inside and outside the power plant in order to take speedy and appropriate measures in response to any accident.
- ➢ Follow the chief operator's instructions on all matters related to the operation.
- ➢ Operation of equipment should be confirmed by the chief operator.
- ➢ Check all the related instrumentation, display lamps and indicators, both before and after operating each component.

(1) Precautions during normal operation

During operation, in addition to the monitoring of all the instruments, the power plant should be patrolled at least once a day.

The following are the key check items during normal operation:

- Vibration or abnormal noise of equipment
- Lubricant and cooling water levels
- Temperature of each part
- Abnormal instrument indication:

  Generator load conditions including voltage, electric current, output, power factor, etc.
- Performance of compressors and oil pressure pumps
- Abnormality of equipment and other installations inside and outside the plant

(2) Key items before start-up

1) When restarting the turbine and generator after a long shutdown period due to inspection or repair, a visual inspection and simple tests and measurements must be conducted to check for moisture absorption on the generator coil, rust on water tubes, rust on bearings, chemical changes on brush contact surfaces, foreign matter intrusion, and defective wiring during the said inspection or repair.

   The major activities are described below:

   - Measure the insulation resistance of each circuit.
   - Inspect the condition of the brushes.
   - Check the bearing oil level and check for oil leak.
   - Inspect the cooling equipment.
   - Confirm that the regulators and other components are correctly positioned.
   - Inspect the oil pressure supply system and speed governor peripherals.

2) Should the generator automatically stop due to an accident, determine the cause and repair the defect. Ensure complete recovery and then restart the operation.

(3) Key steps to shutdown

The key steps are described below.

- At parallel off, the generator circuit breaker is opened after main current is set to zero.
- At shutdown, apply brake at approximately l/3rd the rated revolving speed. Prevent long operation at low rotation.
- Stop the cooling water. Close the generator air duct shutters.
- Patrol the plant after shutdown.
- Provide anti-dewing measures for a long shutdown.

20.2.2 Countermeasures against Hazards

(1) Countermeasures against flood

During a flood, it is recommended that the generator be stopped and that both the intake and outlet gates be closed to prevent sediment and driftwood from entering the waterway.

(2) Countermeasures against earthquake

Depending on the seismic intensity, an earthquake could adversely affect the entire power plant facilities. It is, therefore, necessary to inspect all the components and facilities.

- Inspect for cracks, breakage, tilting and other structural damage.
- Inspect the turbine and generator shaft cores.
- Conduct an overall inspection of all electrical components.
- Inspect all the other facilities.

20.2.3 Other Precautions

The key factors in hydropower plant operation in developing countries are described herein, factors which must be fully understood by all the staff concerned. It is important to prepare an operating manual for operators to carry out their operation tasks in an efficient manner.

(1) Turbine operative range

Operation shall be conducted within an operative range depending on the discharge and the head. When the water level or head is extremely low, the operation shall be stopped in order to prevent unnecessary wear of the runner.

Where multiple units are installed, the number of units to be operated should be controlled depending on the inflow, specifically to avoid operation at a low power discharge per unit.

(2) Regulating load during operation

In developing countries, an electricity load may peak in the evening. The load may also fluctuate significantly during this period. It is necessary, therefore, to carefully control the number of units to be operated corresponding to the load.

The operations manual should be prepared on the distinct understanding of the role of the power plant in load sharing (output adjustment, base load, etc.).

(3) Recovery of isolated system

Loading in stages is recommended when starting plant operation in an isolated power system.

If the entire load is loaded at once, the generator may trip due to a momentary overload.

20.3 Maintenance

20.3.1 General

For the stable operation of a hydropower plant and the prevention of accidents, it is necessary to properly maintain the civil structures and electro-mechanical facilities, maintain/restore their performance efficiency, and try to detect abnormalities at an early stage.

It is, therefore, important to conduct periodic patrols, inspections, and measurement of civil structures and electrical equipment. It is also important that the results of these inspections and measurements be recorded and stored in the specified forms. These records are then used to determine the operational trends and patterns of the said equipment and facilities.

It is recommended to conduct the periodic inspections simultaneously for the equipment and facilities that require a turbine/generator shutdown to minimize the shutdown period.

20.3.2 Civil Structures

(1) Patrol

Patrols are conducted to detect abnormalities in the civil structures and to assess the surrounding conditions. It is recommended to pre-determine the patrol route and to carry it out approximately once a month.

(2) Inspection

The inspection of civil structures is generally classified below and the contents of inspection are shown in Table 20-1.

- Inspection
  - Periodic inspection
    - Visual inspection
    - Internal inspection
  - Emergency inspection
    - Emergency inspection
    - Emergency detailed inspection

An external l inspection is conducted approximately once every six months or every year periodically to determine the conditions of civil structures, to detect any abnormalities, and to check their performance.

An internal inspection is conducted by dewatering the waterway approximately once every five years to inspect the presence of any abnormalities of the channel interior and to observe the functions of the waterway.

An emergency inspection is conducted after an earthquake, a flood, heavy rain, etc., as deemed necessary. An emergency detailed inspection is conducted when deemed necessary after a patrol, external inspection, internal inspection, or emergency inspection.

(3) Measurements

Measurements are conducted to determine the current conditions of hydrology and meteorology, civil structures, and other equipment and facilities. Hydrologic and meteorological observations are conducted at the intake or power plant site. The observation items are weather, temperatures (highest and lowest), humidity, rainfall, air pressure, wind direction, wind velocity, etc.

Measurements carried out in Japan for civil structures include the thickness of the penstock shell and spillway conduit shell, which are measured once every ten years approximately. In addition the paint film thickness is also measured approximately once a year.

Measurements items regarding the reservoir and regulating pond include sedimentation, water quality and behavior of surrounding ground slopes.

20.3.3 Electro-Mechanical Facilities

(1) Patrol

The purpose of plant inspection patrol is to check for abnormalities and to check the operational condition of electrical components in general. The key detection factors are abnormal noise, smell and vibration.

The inspection patrol is conducted daily on a predetermined course.

(2) Inspection

In general, the inspection of electro-mechanical facilities is classified as follows:

- Emergency
  - Visual inspection
    - Emergency inspection
    - Emergency detailed inspection
  - Internal inspection

An external inspection is conducted periodically approximately once every one to three years. The turbine and generator are shut down during this inspection to check for abnormalities and to check their performance.

An internal inspection is conducted periodically approximately once every five to ten years. The turbine and generator are overhauled, thoroughly cleaned and repaired to restore their performance. It is recommended that the inspection cycle be so set as to consider the inspection results and the operation conditions.

An emergency inspection is conducted when an abnormality or problem occurs in an electrical component. The turbine and generator are shut down during this inspection.

The major periodic inspection items of electrical facilities are listed in Table 20-2.

Table 20-1 Inspection Items of Civil Structures

Facility	Installations	Inspection items
1. Intake weir	Dam	· Damage, frost damage, and cracks on surface · Location, volume and turbidity of water leakage
	Peripheral valley slope;	· Water leakage, cracks, collapse, landslide, scouring, etc.
	appurtenant structures;	· Frost damage, abrasion, cracks, displacement, etc.
	Other facilities; Water stage recorder Staff gage Security fence Lighting facility	· Damage, loss, rust, etc.
2. Waterway:		
(1) Intake		· Damage, deformation, cracks, frost damage, abrasion, scouring, etc. · Screen clogging
(2) Settling basin	Basin interior;	· Conditions of basin interior · Abnormalities and their state
(3) Headrace	Peripheral bedrock;	· Collapse, landslide, spring water, etc.
	Headrace interior;	· Leakage, spring water, cracks, scouring, deformation, sedimentation, paint film deterioration, etc.
(4) Head tank		· Damage, deformation, cracks, frost damage, abrasion, scouring, etc.
	Peripheral bedrock;	· Collapse, landslide, spring water and other abnormalities, and their state
(5) Penstock and spillway	Steel penstock;	· Damage, deformation, settlement, etc.
	Penstock and Spillway conduit;	· Damage, deformation, vibration, leakage, paint film deterioration, etc., on pipe shell and saddle · Damage, deformation, paint film deterioration and other abnormalities, and their state
	Peripheral bedrock;	· Collapse, landslide, spring water, etc.
(6) Powerhouse	Foundation and peripheral structures;	· Deformation, cracks, spring water, etc.
(7) Tailrace	Structure;	· Damage, deformation, cracks, frost damage, abrasion, scouring, etc.
3. Other installations	Spoil bank;	· Collapse, landslide, spring water, etc
	Access road;	· Surface conditions · Abnormalities on retaining wall, bridge and other structures, and their state
	Screen;	· Damage, deformation, loose fixing bolts, paint film deterioration, etc.
	Gate;	· Damage, deformation, etc., to gate guide · Damage, deformation, abrasion, greasing, paint film deterioration, and other damage to gate and hoist · Abnormalities and the state of switchboard terminal, wiring, electro-magnetic switch contactor relay performance, insulation resistance, etc. · Conditions of indicators, switches, display lamps of each component
	Trash boom and trash rake;	· Damage to rake and traveling device, abnormality to conveyer, corrosion, paint film deterioration, etc.

Table 20-2 Inspection Items for Electro-Mechanical Facilities

Component	Periodic inspection	
	External inspection	Internal inspection
1. Turbine	(Turbine internal) Inspect and measure for abrasion, cracks, erosion, and rust on the runner, guide vane and casing interior. Measure the runner gap and guide vane gap. Check the bearing lubricant quality. * Test: automatic start/stop	(Turbine overhaul) Measure abrasion loss at each part. Inspect the sliding area and packing for damage and fine cracks. (Bearing overhaul) Measure the damage and gap on the sliding surface. Calibrate the cooling water pipe pressure resistance, thermometer and oil gauge. * Replace worn parts. * Tests: load rejection, vibration measurement, stroke output, automatic start/stop
2. Speed governing device	(Mechanism) Inspect for abrasion of movable parts, loose wiring/lever, and strainer overhaul (Controller) Inspect the conditions of the printed circuit board and position transducer. Measure the insulation resistance.	(Mechanism overhaul) Overhaul movable part and PMG. Replace worn parts. * Tests: characteristics and load rejection
3. Inlet valve	(Inlet valve internal) Measure leakage. Inspect for abrasion and erosion. Measure sheet surface clearance. Inspect position indicator conditions.	(Operation mechanism overhaul) Inspect for damage to movable part and sliding area. (Valve body overhaul) Inspect for abrasion and erosion. Inspect for damage to the packing and the sealing condition. * Replace worn parts.
4. Oil pressure supply and lubrication oil system	(Performance) Measure load operation time. Test oil quality. (Oil filtration) Test oil quality.	(Oil pressure supply and lubrication oil system overhaul) Inspect for abrasion and damage to internal movable part and sliding area, and motor insulation resistance. (Performance test) Measure pump discharge and grease feed volume.
5. Water supply and drainage system	(Strainer overhaul) Inspect abrasion and erosion	(Pump overhaul) Inspect for abrasion and damage to internal movable part and sliding area, and motor insulation resistance. (Performance test) Measure water supply and drain volume.
6. Automatic turbine control system	(Performance test of all relays)	
7. Generator	(Generator internal) Inspect for loose electric circuit terminals, discolored, peeled or loose coil, abrasion and damage to slip ring, loose and rusted revolving part. Measure brush contact pressure and the insulation resistance of electric circuit. (Control system) Inspect for shoe abrasion loss and operation state. (Neutral grounding resistor) Measure resistance and insulation resistance.	(Rotor lifting) Inspect for loose rotor core and winding. Measure winding deterioration. Inspect loose wedge, flaking varnish, and rust. (Control system, bearing, and air cooler overhaul) * Measure shaft current. * Exciter characteristic test

20.3.4 Spare Parts

Spare parts are stored for the quick corrections of and recovery from abnormalities and breakdowns. The variety and quantity of these spare parts are determined considering their frequency of breakdowns, the manufacturing period and the importance level of the parts. The storage location is determined based on the haul distance and other conditions. The required quantity of consumables such as brushes and fuses is stored separately.

An example of spare parts for hydropower stations is listed in Table 20-3.

Table 20-3 Common Spare Parts

Component	Part name	Quantity	Remarks
Turbine			
Main unit	Main bearing	for 1 unit	
	guide vane weak point pin	for unit	
	sealing packing	for 1 unit	
	runner	for 1 unit	
	guide vane	for 1 unit	During repairs, the damaged parts are repaired and stored as auxiliary parts
	nozzle tip	for 1 unit	
	bucket	for 1 unit	
Governor	printed circuit board	1 each	
	moving coil	1	
	various springs	1 each	
Oil pressure supply and lubrication oil system	Oil pressure lubricant pump	1 unit	Only where an auxiliary system is not installed
	unloader spring	1	
Compressed air generator	safety valve spring	1	
	pressure reduction valve	1	When not included in the auxiliary equipment as a set. Not stored if available from other power plant stocks.
Automatic control system	solenoid for electromagnetic valve	1	
Generator	thrust bearing metal	for 1 unit	
	guide bearing metal	for 1 unit	
	stator coil	5 - 10	
	brush holder	for unit	
Exciter	printed circuit board	1 each	
	field breaker coil	1	
	semiconductor rectifier	for 1 phase	
Transformer	Bushing	for 1 phase	Not stored if available from other power plant stocks.
	bursting board	1	
Switchgear			
Breaker	bushing	for 1 phase	
	fixed/movable contact	for 1 phase	Not stored if available from other power
	switching coil	1 each	
Breaker	switching coil	1 each	
Others			
Generator main circuit	current transformer (per model)	1 each	Not stored if available from other power plant stocks.
	instrument transformer (per model)	1 each	

Reference of Chapter 20

[1] Guide Manual for Hydropower Development, New Energy Foundation, 1996

APPENDIX

Appendix Contents

A-5-1: Case Study of Hydropower Planning by Reconnaissance Method

1. Run-of-river Type

Main study points are described below in accordance with the study procedure written in Chapters 5 and 6.

(1) Planning of project

1) Study of waterway route

The power generation method for project site A is studied from topographic map. As the project site is topographically unsuitable for the construction of a regulating pond, the run-of-river type is studied. The waterway route was determined in accordance with 5.3.3 (2). (Details are omitted)

2) Catchment area

Measured from topographic map, the area is 55 km^2.

3) Preparation of flow duration curve

The flow duration curve is prepared by the daily flow data at the project site. (Refer to Figure A-5-1.)

4) Firm discharge (Q_f)

Firm discharge (Q_f) corresponding to 90% discharge (328 days) is obtained from Figure A-5-1. In this example, the firm discharge (Q_f) of 90% discharge is used for the local condition.

$$Q_f = 2.00 m^3/sec$$

5) Maximum plant discharge (Q_{max})

Flow for about 70% utilization factor of runoff (UF) is obtained from Figure A-5-1 and is taken as the maximum plant discharge.

$$Q_{max} = 7.50 m^3/sec$$

6) Waterway profile

The profile of the waterway route determined above is shown in Figure 5-15 Chapter 5.

7) Calculation of head loss ($H\ell$) and normal effective head (rated head: He)

$$Hg = IWL - TWL = 410 - 350 = 60m$$

$$H\ell = \frac{2{,}000}{1{,}000} + \frac{100}{200} + 0.6 = 3.1$$

$$He = Hg - H\ell = 56.9m$$

The length of headrace (L_1) is 2,000m, and length of penstock (L_2) is 100m.

8) Selection of turbine type and combined efficiency

As the maximum plant discharge of 7.5m^3/sec, and effective head of 56.9m are obtained, Francis turbine is selected from Figure 12-16 in Chapter 12. The combined efficiency is 0.84 from Table

5-2 in Chapter 5 assuming a plant output of 4,200kW.

$(Q_{max} \times He \times 9.8 = 4{,}200kW)$

9) Calculation of maximum output and firm output

Maximum output

$$P = 9.8 \times Q_{max} \times Hes \times \eta = 9.8 \times 7.5 \times 56.9 \times 0.84 = 3{,}500kW$$

Firm output

$$Pf = 9.8 \times Q_f \times Hes \times \eta_f = 9.8 \times 2.0 \times 56.9 \times 0.63 = 700kW$$

The number of turbine is one unit, and η_t is obtained by the following equation taking into account difference between 0.67 for $Q_f/Q_{max} = 0.27$ of the Francis turbine from Figure 5-16 and combined efficiency at 5MW and 50MW at the time of 100% load.

$$\eta_f = 0.67 - (0.88 - 0.84) = 0.63$$

10) Annual energy generation

Table A-5-1 shows an example of energy calculation.

$$E = 21.3 \times 10^6 \text{ kWh}$$

Primary energy of the above energy is 1.6×10^6 kWh.

11) Plant factor

エラー! ブックマークが定義されていません。$Pf = \dfrac{21.3 \times 10^6}{3{,}500 \times 8{,}760} \times 100 = 69\%$

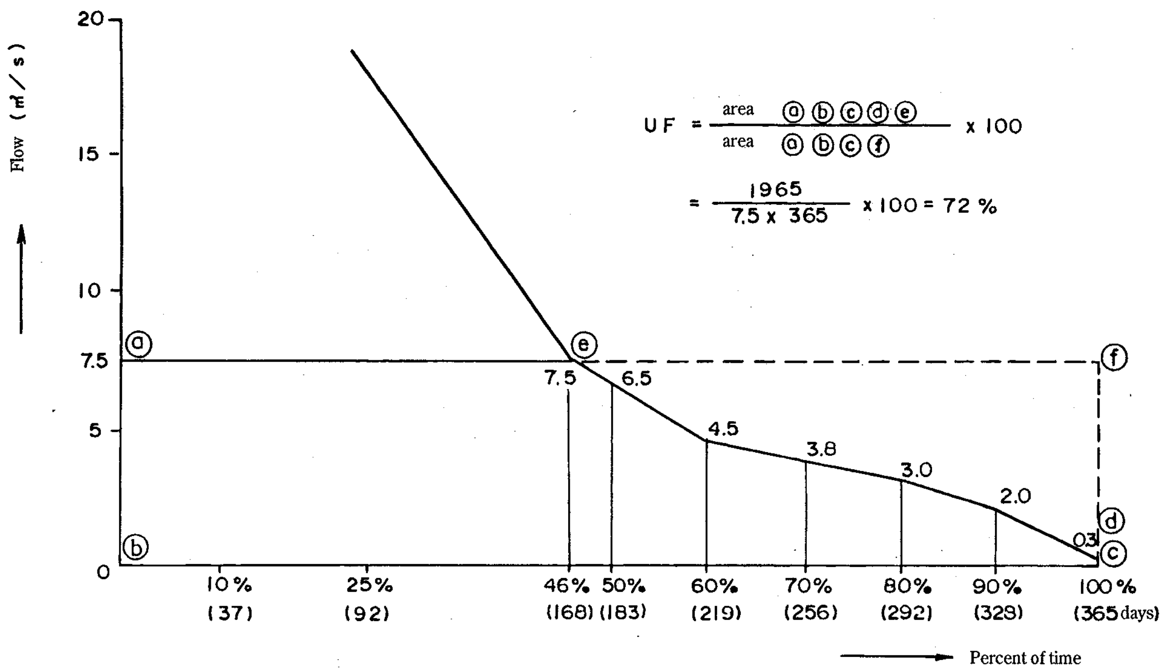

Figure A-5-1 Flow Duration Curve

Table A-5-1 Energy Calculation (Run-of-river Type)

(1) Days	(2) Difference of (1)	(3) q (m^3/s)	(4) Mean of (3) (m^3/s)	(5) (4)/Qmax (%)	(6) ξ	(7) Q (10^6m^3)	(8) E (10^6kwh)
168	168	7.50 (2.00)	7.50 (2.00)	100 (27)	0.84 (0.18)	108.9 (29.0)	14.17 (0.81)
183	15	6.50 (2.00)	7.00 (2.00)	93 (27)	0.85 (0.18)	9.1 (2.6)	1.20 (0.07)
219	36	4.50 (2.00)	5.50 (2.00)	73 (27)	0.85 (0.18)	17.1 (6.2)	2.25 (0.17)
256	37	3.80 (2.00)	4.15 (2.00)	55 (27)	0.79 (0.18)	13.2 (6.4)	1.62 (0.18)
292	36	3.00 (2.00)	3.40 (2.00)	47 (27)	0.75 (0.18)	10.5 (6.2)	1.22 (0.17)
328	36	2.00	2.50 (2.00)	33 (27)	0.67 (0.18)	7.8 (6.2)	0.81 (0.17)
365	37	0.30	1.15 (1.15)	15 (15)	0 (0)	3.7 (3.7)	0 (0)
Total						170.3	21.2 (1.57)

Notes:

Turbine type: Francis

ξ= Combined efficiency of turbine - generator (Combined efficiency shown on Figure 5-16, is the difference 4%, of combined efficiency between 5MW and 50MW deducted from the efficiency of 5MW)

In case of Column (4)/ Q_{max} < 0.2, ξ= 0.0 is assumed.

In case of ξ>0.84, ξ = 0.84 is assumed.

(7) = (2)×(4)×24×3,600

(8) = 9.8×(6)×He×(7)×1/3,600

He = 56.9 m

Bracket means primary energy

(2) Calculation of quantities of works

1) Intake dam

Height (Hd) = 10 m, Crest length = 30 m

Quantities

$Ve = 8.69\times(Hd\times L)^{1.14} = 5{,}800\ m^3$

$Vc = 16.1\times(Hd2\times L)^{0.695} = 4{,}200\ m^3$

$Wr = 0.0274\times Vc^{0.830} = 28$ tons

$Wg = 0.145\times Q_f^{\ 0.692} = 10$ tons

Annual rainfall of 1,100 mm gives runoff coefficient "a" = 170 of district H

A = 55 km^2, q = 8.2 m^3/sec/km^2, Q^f = 8.2×55 = 450 m^3/sec

2) Intake

Non-pressure type, Q = 7.5 m^3/sec, tunnel inner diameter D = 2.2 m (from Figure 6-2, Chapter 6) and radius R = 1.1 m

Quantities

$Ve = 171\times(R\times Q)^{0.666} = 700\ m^3$

$Vc = 147\times(R\times Q)^{0.470} = 400\ m^3$

$Wr = 0.0145\times Vc^{1.15} = 14$ tons

$Wg = 1.27\times(R\times Q)^{0.533} = 4$ tons

$Ws = 0.701\times(R\times Q)^{0.582} = 2$ tons

3) Settling basin

$Q = 7.5\ m^3/sec$

Quantities

$Ve = 515\times Q^{1.07} = 4{,}400\ m^3$

$Vc = 169\times Q^{0.936} = 1{,}100\ m^3$

$Wr = 0.120\times Vc^{0.847} = 45$ tons

$Wg = 0.910\times Q^{0.613} = 3$ tons

$Ws = 0.879\times Q^{0.785} = 4$ tons

4) Headrace

Non-pressure tunnel, I = 1/1,000, $Q_{max} = 7.5\ m^3/sec$, L = 2,000m, D = 2.2m (From Figure 6-2)

Quantities

$Ve = (0.893\times D^2+1.07\times D+0.321)\times L = 14{,}000\ m^3$

$Vc = (1.07\times D+0.321)\times L = 5{,}400\ m^3$

$Wr = (0.00911\times D+0.00273)\times L = 46$ tons

5) Head tank

$Q = 7.5\ m^3/sec$

Quantities

$Ve = 808\times Q^{0.697} = 3{,}300\ m^3$

$Vc = 197\times Q^{0.716} = 830\ m^3$

$Wr = 0.051\times Vc = 42$ tons

6) Penstock and spillway channel

(a) Penstock

$Q = 7.5\ m^3/sec$, He = 56.8 m

Inner diameter of steel pipe: Dm = 1.8 m (From Figure 6-3)

Length of steel penstock: L = 100 m (From waterway profile)

Design head: H = 60 m

Quantities

$Ve_1 = 10.9\times D_m^{1.33}\times L = 2{,}400\ m^3$

$Vc_1 = 2.14\times D_m^{1.68}\times L = 570\ m^3$

$Wr_1 = 0.018\times Vc = 10$ tons

$t_m = 0.0362\times H\times D_m+2 = 6$ mm

$Wp_1 = 7.85\times\pi\times D_m\times t_m\times 10^{-3}\times 1.15\times L = 31$ tons

(b) Spillway channel

Q_{max} = 7.5 m_3/sec, mean gradient 1/1

D = 0.8 m, L = 100 m (Installed parallel with penstock)

Quantities

$Ve_2 = 9.87 \times D^{1.69} \times L = 680\ m^3$

$Vc_2 = 2.78 \times D^{1.70} \times L = 190\ m^3$

$Wr_2 = 0.029 \times Vc_2 = 6$ tons

$Wp_2 = 0.165 \times D^{1.25} \times L = 12$ tons

(c) Total quantities

$Ve = Ve_1 + Ve_2 = 3{,}100\ m^3$

$Vc = Vc_1 + Vc_2 = 60\ m^3$

$Wr = Wr_1 + Wr_2 = 16$ tons

$Wp = W_{p1} + W_{p2} = 43$ tons

7) Powerhouse

$Q_{max} = 7.5\ m^3$/sec, He = 56.9 m, Number of unit, n = 1

Quantities

$Ve = 97.8 \times (Q \times He^{2/3} \times n^{1/2})^{0.727} = 3{,}000\ m^3$

$Vc = 28.1 \times (Q \times He^{2/3} \times n^{1/2})^{0.795} = 1{,}200\ m^3$

$Wr = 0.046 \times Vc^{1.05} = 79$ tons

8) Tailrace

Non-pressure type, Q = 7.5 m^3/sec, R = 1.1 m

Quantities

$Ve = 395 \times (R \times Q)^{0.479} = 1{,}100\ m^3$

$Vc = 40.4 \times (R \times Q)^{0.684} = 170\ m^3$

$WR = 0.278 \times Vc^{0.61} = 6$ tons

(3) Total construction cost

The unit prices of "A" country are applied to the above quantities of works to arrive at the estimated total construction cost of 138 million monetary units. The basis for calculation is given below, and Tables A-5-2 and A-5-3 give the details of calculation example.

- The cost of electric equipment vs. the parameter of $P/H^{1/2}$ (where, P: output kW, He: effective head m) are plotted on a bilateral logarithmic coordinate paper referring to examples in the said country, and the cost is estimated for $P/H^{1/2} = 463 \times 10^3$ of this project.
- The mean interest rate of local and foreign currencies is used.
- Unit prices for items of work are based on past examples.
- The construction period is assumed to be 3 years.

Table A-5-2 Construction Cost (Run-of-river Plant)

(10^3 monetary unit)

Description	Estimated Cost	Note
1. Preparatory work (1) Access Road (2) Camp & Facilities	 2,800	 (3 Civil work) × 0.05
Sub total	2,800	
2. Envirormental Mitigation Cost	550	(3 Civil work) × 0.01
3. Civil Works (1) Intake Weir (2) Intake (3) Settling Balin (4) Headrace (5) Head tank (6) Penstock (7) Powerhouse (8) Tailrace channel (9) Tailrace (10) Miscelloneous Works	 11,960 1,330 3.840 23,330 3,510 2,440 5,380 — 640 2,670	 ((1)～(9)) × 0.05
Sub total	55,100	
4. Hydraulic Equipment (1) Gate & Screen (2) Penstock	 1,900 2,600	
Sub total	4,560	
5. Electro-mechanical Equipment	38,000	
Direct Cost	100,950	1 + 2 + 3 + 4 + 5
6. Administration & Engineering fee	15,100	Direct Cost × 0.15
7. Contingency	10,150	Direct Cost × 0.1
Total	126,200	i = 8%, T = 3 years
8. Interest during Construction	11,800	(Total) × 0.4 × i × T
Total Cost	138,000	

Table A-5-3 (1) Civil Works Cost (Run-of-river Type)

(10^3Monetary Unit)

Work Items	Unit	Unit Price		Quantity		Total Amount
1. Intake Weir						11 960
Excavation	m3	80		5,800		464
Concrete	m3	2,000		4,200		8 400
Reinforcement bar	ton	12,000		28		336
Others	L. S.	—		30%		2,760
2. Intake						1,330
Excavation	m3	80		700		56
Concrete	m3	2,100		400		840
Reinforcement bar	ton	12,000		14		168
Others	L. S.	—		25%		266
3. Settling Basin						3,840
Excavation	m3	80		4,400		352
Concrete	m3	2,100		1,100		2,310
Reinforcement bar	ton	12,000		45		540
Others	L. S.	—		20%		638
4. Headrace						23,330
Excavation	m3	600	—	14,000	—	8,400
Concrete	m3	2,100	—	5,400	—	11,340
Reinforcement bar	ton	12,000	—	46	—	552
Others	L. S.			15%	30%	3,038
5. Head tank						3,510
Excavation	m3	80		3.300		264
Concrete	m3	2,100		830		1,743
Reinforcement bar	ton	12,000		42		504
Others	L. S.	—		40%		999
6. Penstock & spillway						2,440
Excavation	m3	80		3,100		248
Concrete	m3	2,100		760		1,596
Reinforcement bar	ton	12,000		16		192
Others	L. S.	—		20%		404
7. Powerhouse						5,380
Excavation	m3	80		3,000		240
Concrete	m3	2,000		1,200		2,400
Reinforcement bar	ton	12,000		79		948
Others	L. S.	—		50%		1,792
8. Tailrace channel						—
Excavation	m3	—	—	—	—	—
Concrete	m3	—	—	—	—	—
Others	L. S.	—	—	15%	30%	—
9. Tailrace						640
Excavation	m3	80		1,100		88
Concrete	m3	2,100		170		357
Reinforcement bar	ton	12,000		6		72
Others	L. S.	—		25%		123
10. Miscellaneous Works	L. S.	—		5%		2,670
Sub total	—	—		—		55,100

Table A-5-3 (2) Hydraulic Equipment Cost (Run-of-river Type)

(10^3 Monetary Unit)

Work Items	Unit	Unit Price	Quantity	Total Amount
1. Weir				240
Gate	ton	80,000	3	240
2. Intake				400
Gate	ton	80,000	4	320
Screen	ton	40,000	2	80
3. Settling basin				400
Gate	ton	80,000	3	240
Screen	ton	40,000	4	160
4. Penstock and Spillway	ton	50,000	43	2,150
5. Outlet Gate	ton	—	—	—
6. Others	L. S.	—	20%	610
Sub total				3,800

(4) Economic evaluation

Economic evaluation is conducted according to Section 6.3.

Coal fired thermal power is used for economic evaluation as alternative thermal power plant.

1) Annual benefit of hydro power

Ph = 700kW (Firm output)

E = 21.3×10^6kWh (annual energy generation)

$b_1 = Ct \times \alpha = 26{,}000\times(0.11+0.03) = 3{,}640$ monetary unit/kW

where,

Ct : Unit construction cost of coal fired thermal power plant (monetary unit/kW)

A : Annual cost factor (discount rate 10%)

$b_2 = 860\div0.38\div5{,}800\times1.096 = 0.428$ monetary unit/kWh

Thermal efficiency 38%, calorific value 5,800 kcal/kg,

fuel unit price 1.096 monetary unit/kg

$B = B_1+B_2 = 700\times3{,}640+21.3\times106\times0.428 = 11.7\times10^6$ monetary unit

2) Annual cost of hydro power

Ch = 138×106 monetary unit

$$C = Ch\times \alpha = 138\times10^6\times0.11 = 15.2\times10^6 \text{ monetary unit}$$

where,

Ch : Hydro power construction cost (monetary unit)

α : Annual cost of hydro power

3) Calculation of B/C

$$B/C = 11.7\times10^6 / (15.2\times106) = 0.77$$

2. Reservoir Type

The main study points are described below in accordance with the study procedure given in Chapter 5 and 6.

(1) Project study

1) Study of dam site, waterway route and power generation type

The power generation type at site B is studied from available topographic maps, and either reservoir type or pondage type is selected from the following reasons.

- A dam in 100 m height provides a storage capacity of approx. 280×10^6 m^3
- By classification of method to gain head, dam and waterway type of development is selected for the following reason.
- An additional head of approx. 50m can be gained by increasing the waterway length to some extent.

2) Catchment area

Area of 1,050 km^2 is measured from the topographic map.

3) Mean flow and inflow at the dam site

The annual mean flow and annual total inflow at the dam site are obtained from runoff data of the flow gauging station by ratio of catchment area.

$$Q_{ave} = 58.48 \text{ m}^3/\text{sec}$$

$$\Sigma Q_i = 58.48\times86{,}400\times365 = 1{,}844\times10^6 \text{ m}^3$$

where,

Q_{ave} : Annual mean flow at dam site (m^3/sec)

ΣQ_i : Annual total inflow at dam site (m^3)

4) Reservoir area and storage capacity curve

The storage capacity curve shown in Figure A-5-2 is prepared from topographical map.

5) Sediment volume and sedimentation level

Sediment volume deposited in the reservoir is calculated at 12.5×10^6 m^3 for 100 years from the specific sediment yield of 119 m^3/ km^2/year. Assuming the HWL of the reservoir of about 500m, the gross storage capacity (Vg) is about 280×10^6 m^3. The value of ($Vg/\Sigma Q_i$ = 0.5) gives the sediment trap efficiency of the reservoir referring to Figure 9-50, then sediment volume of

10×10^6 m^3 is calculated.

$$Vs = 119 \times 1{,}050 \times 100 \times 0.8 = 10.0 \times 10^6\ m^3$$

The sedimentation level derived from the storage capacity curve is EL. 428 m.

6) Lower limit of low water level

Low water level is tentatively set at EL. 439 m on the basis of the sedimentation level of EL. 428 m and the inner diameter of tunnel of 6.2 m (tentative value).

The basis for setting this value is as follow.

- ➢ Maximum plant discharge is set at 234 m^3/sec (tentative) on the basis of the following equation.

  $Q_{max} = Q_{ave}/0.25 = 58.42/0.25 = 234\ m^3/sec$

- ➢ Two headrace tunnels are adopted and the inner diameter (D) of each tunnel is 6.2 m from Figure 6-2 for design discharge of 117 m^3/sec

  $LWL = ELc+1+D \times 1.5 = 428+1+6.2 \times 1.5 \div 439\ m$

7) Temporary setting of high water level (HWL) and gross storage capacity (Vg)

Judging from the topographic and geologic features, compensation, etc., the high water level is set at EL. 500 m. The gross storage capacity (Vg) is 276×10^6 m^3 from the storage capacity curve.

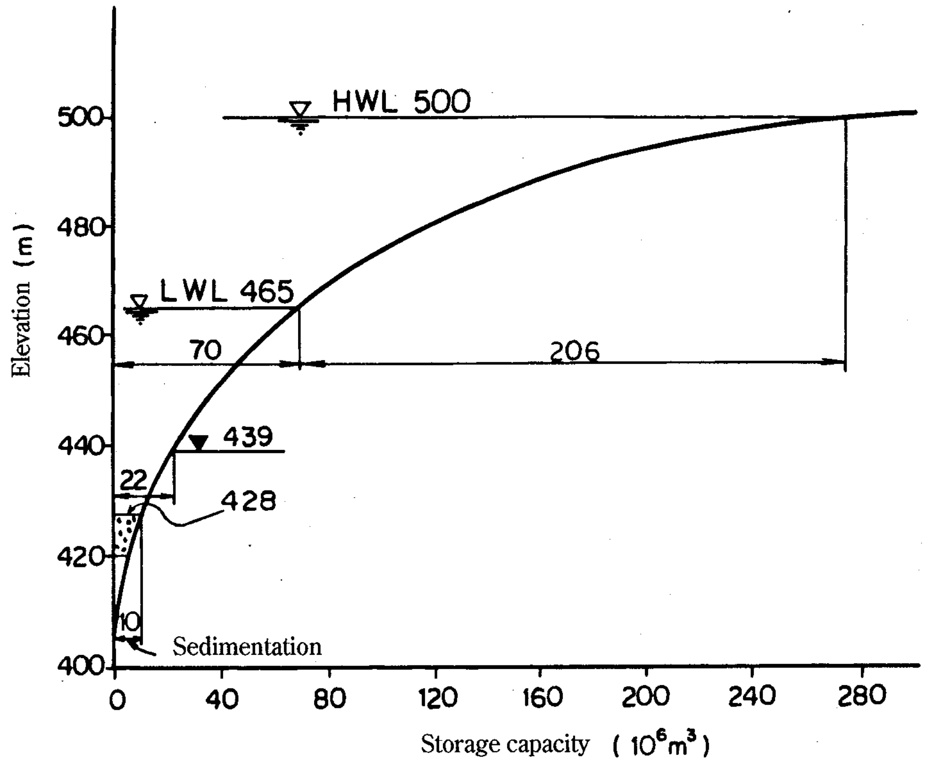

Figure A-5-2 Storage Capacity Curve

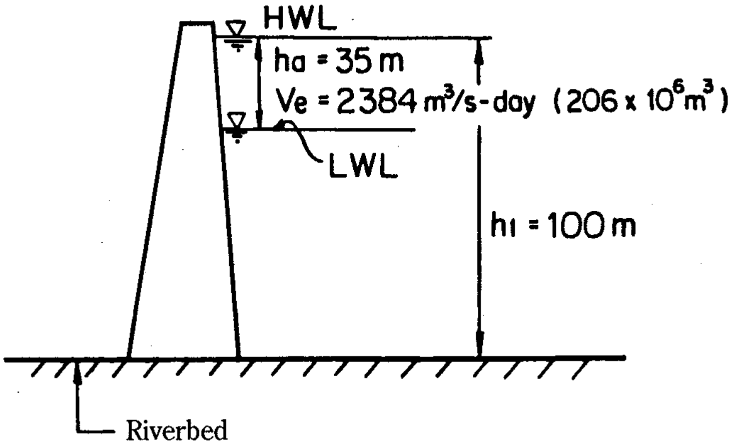

Figure A-5-3 Schematic of Reservoir Capacity

8) Determination of effective storage capacity, low water level and high water level

Justification of low water level of EL. 439 m is checked.

(a) Available drawdown, effective storage capacity and regulating capability factor of reservoir (RCF)

Available drawdown	: ha = 500 - 439 = 61 m
Effective storage capacity	: Ve = (272 - 22) $\times 10^3$ = 256×10^3 m^3

$$\text{RCF} = \frac{Ve}{\Sigma Q_i}\times 100 = \frac{256\times10^6}{1{,}844\times10^6}\times 100 = 14\ \% > 5\%$$

The RCF of 14 % indicates reservoir type of development is applicable.

(b) Estimate of head fluctuation rate

Head fluctuation rate:

$$\frac{\text{LWL - TWL}}{\text{HWL - TWL}} = \frac{439-350}{500-350} = 0.59<0.7$$

The head fluctuation rate is less than 0.7 which is the limit of the operating range of the Francis turbine, therefore the low water level is reset.

(c) The low water level is reset at EL. 465 m.

Head fluctuation rate:

$$\frac{\text{LWL - TWL}}{\text{HWL - TWL}} = \frac{465-350}{500-350} = 0.77>0.7$$

The low water level of 465 m is adopted because it is in the allowable operating range of the Francis turbine. Whether the low water level is set at EL. 439 m and Kaplan turbine is adopted or not may be studied separately, however it is omitted here because of the reconnaissance level of this study.

(d) High water level of EL. 500 m and low water level of EL. 465 m is determined from the above, and the effective storage capacity is determined to be 206×10^6 m^3.

9) Determination of power generation type

$$RCF = \frac{206\times10^6}{1,844\times10^6}\times100 = 11\%>5\%$$

As the RCF value is more than 5%, a reservoir type is selected.

10) Preparation of mass curve

Table A-5-4 is an example of mass curve calculation. Figure A-5-4 is an example of the mass curve.

11) Calculation of firm discharge (Q_f)

Firm discharge is calculated by using Figure A-5-4.

S_1 = 4,887 (m^3/sec-day)
S_2 = 0 (m^3/sec-day)
Ve = 206 (10^6 m^3) = 2,384 (m^3/sec-day)
n = 243 (day)
Q_{ave}= 58.48 (m^3/sec)

$$Qf = \frac{S_2 + Ve - S_1}{n} + Qave = 48.18 \text{ (m3/sec)}$$

The firm discharge is obtained from the mass curve by using one year runoff record so that the study procedure can be easily understood. In the actual study, however, it should be obtained from long-term runoff data.

Table A-5-4 Mass Curve Calculation

month	(1) day	(2) Inflow (m^3/s−day)	(3) = Σ (2) (m^3/s−day)	(4) = Σ Qave × (1) (m^3/s−day)	(5) = (3) − (4) cumulative flow (m^3/s−day)
4	30	3,067.38	3,067.38	1,754.40	1,312.98
5	31	4,250.12	7,317.50	3,567.28	3,750.22
6	30	2,649.58	9,967.08	5,321.48	4,645.40
7	31	2,054.64	12,029.72	7,134.56	4,887.16
8	31	680.44	12,702.16	8,947.44	3,754.72
9	30	723.20	13,425.36	10,701.84	2,723.52
10	31	1,779.96	15,205.32	12,514.72	2,690.60
11	30	1,939.12	17,144.44	14,269.12	2,875.34
12	31	1,487.62	18,632.06	16,082.00	2,550.06
1	31	886.66	19,518.72	17,894.88	1,703.84
2	28	618.08	20,136.80	19,532.32	604.48
3	31	1,207.68	21,344.48	21,345.20	− 0.72
Total	365	21,344.48			
Average		Qave 58.48			

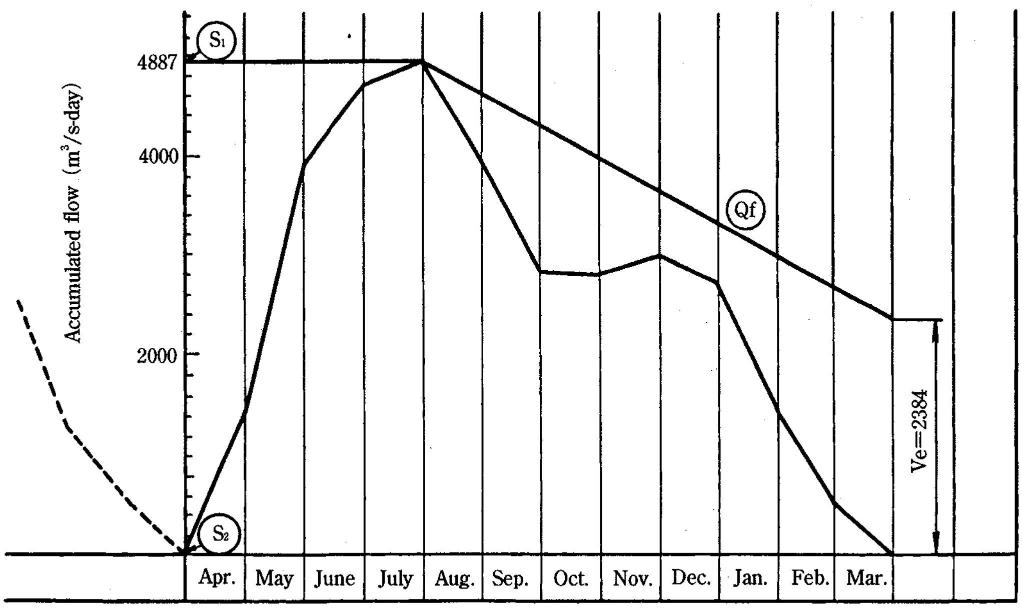

Figure A-5-4 Mass Curve (Differential Mass Curve)

12) Determination of maximum plant discharge

Peak duration hours is assumed as 5 hours to determine the maximum plant discharge.

$$Q_{max} = \frac{Q_f \times 24}{5} = 230 \text{ m}^3\text{/sec}$$

13) Normal water level and tailwater level

Water level of 488 m is the normal water level and the tailwater level is 350 m which is the riverbed elevation at the powerhouse site. NWL = 500 - (500 - 465)/3 = 488 m

14) Waterway profile

Refer to Figure A-5-5.

15) Calculation of head loss and normal effective head

$$Hg = 500 - 35/3 - 350 = 138$$

Head loss ($H\ell$) is obtained as shown in 5.3.4(15).

$$H\ell = a \times L_1 + b \times L_2 + c \times L_3 + \Delta h$$
$$= 850/700 + 155/200 + 0.6 = 2.6 \text{ m}$$

As there is no tailrace, L3 = 0 and other loss (Δh) = 0.6 m.

Normal effective head (Hes) is obtained by the following equation.

$$Hes = Hg - H\ell = 138 - 2.6 = 135.4$$

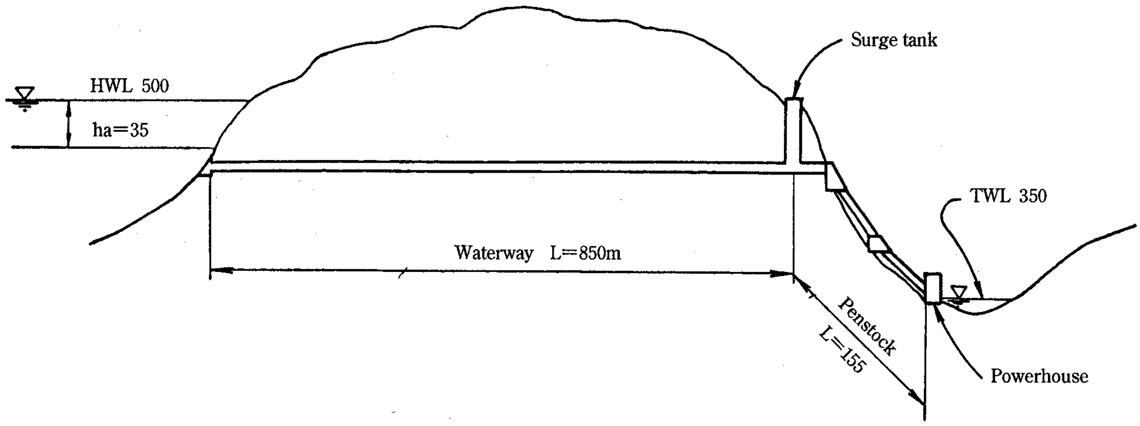

Figure A-5-5 Waterway Profile

16) Selection of turbine type and combined efficiency of turbine and generator

Francis turbine is selected from Figure 12-16, and the combined efficiency for calculation of the maximum output is 89%. (calculated from Table 5-2.)

17) Calculation of maximum output and firm peak output

$$P_{max} = 9.8 \times 0.89 \times 230 \times 135.4 = 270{,}000\text{kW}$$

The firm peak output is the same value.

18) Annual energy generation

(a) Approximate calculation by annual total plant discharge for generation

It is judged in Figure A-5-4 that all inflow of 21,344 m^3/sec-day to the reservoir can be used for generation without spill. The annual energy generation by using annual plant discharge (annual available discharge) is approximately calculated by the following equation.

$$E = 9.8 \times 0.89 \times 21{,}344 \times 135.4 \times 24/10^6 = 605 \times 10^6\text{kWh}$$

(b) Calculation of monthly energy generation by mass curve

Table A-5-5 shows the example of monthly energy calculation by using the mass curve.

Column (1) : Inflow by month

Column (2) : Plant discharge for generation by month

Reservoir is operated to supply firm discharge of 48.18 m^3/sec by drawing down the reservoir in the dry season from August to March as shown in Figure A-5-6.

The tangent line (Q) of the mass curve is operation line from early April when the reservoir is empty to the end of June when it is full to prevent spilling in the flood season as much as possible. The maximum value of Q is the maximum plant discharge, because overflow occurs in case the Q value exceeds the maximum plant discharge of 230 m^3/sec.

$$Q = (3{,}067+4{,}250+2{,}650 - 2{,}384)/91 = 83.33\ m^3/sec < 230\ m^3/sec$$

In July, the reservoir is at the high water level and reservoir is operated so that the outflow is equal to the inflow.

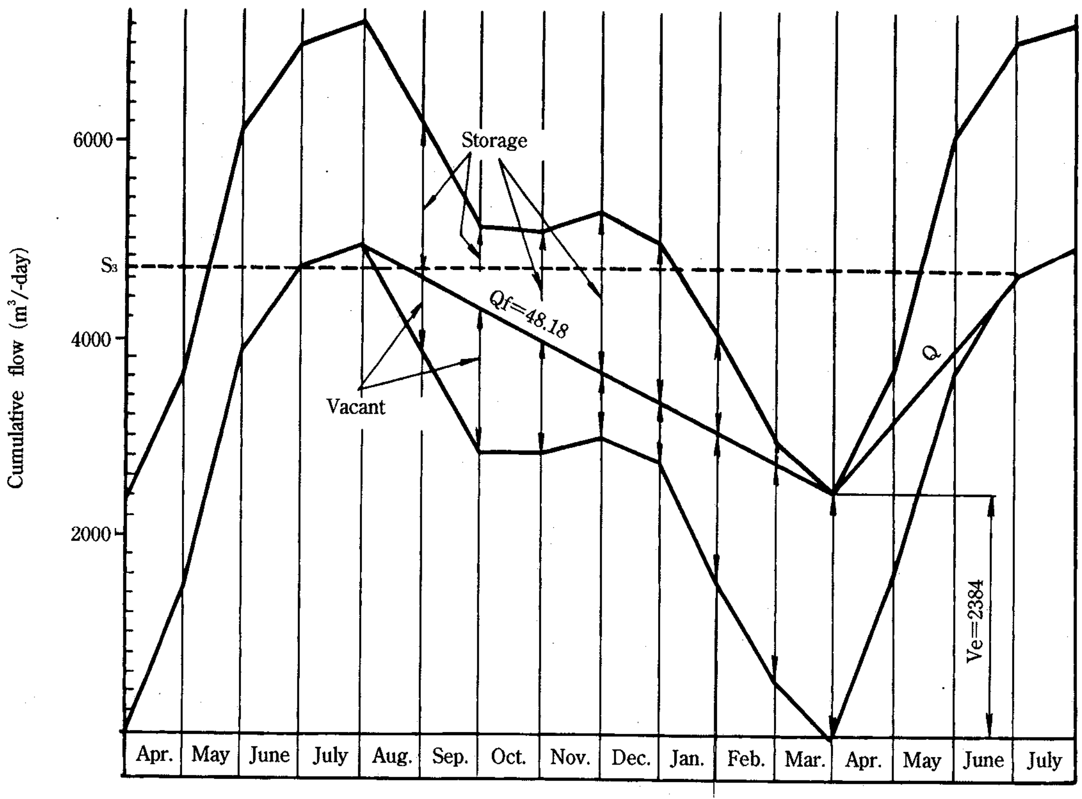

Figure A-5-6 Reservoir Operation Using Mass Curve

Column (3) : Overflow

Because of the above reservoir operation, no overflow occurs.

Column (4) : Storage

Storage at the end of the month is obtained.

Column (5) : Water Level at month's end

The water level corresponding to the storage in (4) is obtained from Figure A-5-7, and this storage level is taken as the month's end storage level. Figure A-5-7 shows the storage volume in m^3/sec-day between the low water level and high water level shown in Figure A-5-2.

Column (6) : Mean water level

The mean water level of this month is obtained from the mean value of column (5) water level at month's end and the beginning of the month water level (month end water level of the preceding month) .

Column (7) : Effective head

The gross head of each month is obtained from the difference between the mean water level of each month and the tailwater level of 350 m and effective head is calculated from head loss of 2.6 m.

Column (8) : Head fluctuation rate

The head fluctuation rate is obtained from the standard effective head and effective head of each month.

Columns (9) (10) : Combined efficiency

The variable head efficiency (column (9)) corresponding to head fluctuation rate is obtained from Figure 5-28, Chapter 5. The efficiency of the turbine and generator of the month is calculated by the following equation. The value 0.89 corresponds to the value Q/ Q_{max} = 100% in Figure 5-16.

Combined efficiency of this month = 0.89 × variable head efficiency of this month

Column (11) : Calculation of electric energy

Electric energy by month (Ei) is obtained by the following equation.

E = 9.8×(10)×(2)×(7)×24/106 (10^6kWh)

The annual energy generation is 609×10^6kWh

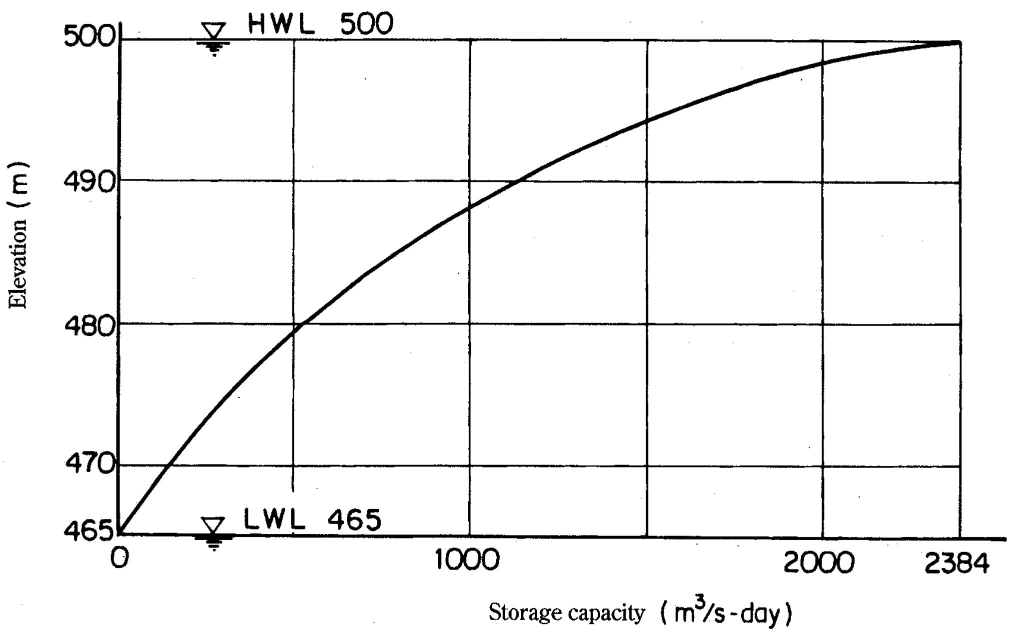

Figure A-5-7 Storage Capacity Curve between High and Low Water Levels

Table A-5-5 Calculation of Energy Generation by Mass Curve

Month	Number of days	(1) Inflow (m3/s-day)	(2) Plant discharge (m3/s-day)	(3) Spill (m3/s-day)	(4) Storage =(4)+(1)−(2)−(3) (m3/s-day)	(5) Reservoir water level end of Month (m)	(6) Average of (5) (m)	(7) Effective head =(6)−350−2.6 (m)	(8) Rate of head fluctuation =(7)/135.4 (m)	(9) Variable head efficiency	(10) Combined efficiency 0.89×(9)	(11) Energy generation 9.8×(10)×(2)×(7)×24/10⁶ (10^6kWh)
(7)					2,384.00	500						
8	31	680.44	(Note 1) 1,493.58	0	1,571.86	495.0	497.5	144.9	1.07	1.0	0.89	45.3
9	30	723.20	1,445.40	0	849.66	485.9	490.5	137.9	1.02	1.0	0.89	41.7
10	31	1,779.96	1,493.58	0	1,136.04	490.0	488.0	135.4	1.00	1.0	0.89	42.3
11	30	1,939.12	1,445.40	0	1,629.76	495.5	492.8	140.2	1.04	1.0	0.89	42.4
12	31	1,487.62	1,493.58	0	1,623.80	495.5	495.6	143.0	1.06	1.0	0.89	44.7
1	31	886.66	1,493.58	0	1,016.88	488.4	492.0	139.4	1.03	1.0	0.89	43.6
2	28	618.08	1,349.04	0	285.92	474.1	481.3	128.7	0.95	1.0	0.89	36.3
3	31	1,207.68	1,493.58	0	0.72(0)	465. 0	469.6	117.0	0.86	0.99	0.89	36.1
4	30	3,067.38	(Note 1) 2,499.90	0	567.48	480.8	472.9	120.3	0.89	0.99	0.89	62.2
5	31	4,250.12	2,583.23	0	2,234.37	499.5	490.2	137.6	1.02	1.0	0.89	74.4
6	30	2,649.58	2,499.90	0	2,384.00	500.0	499.8	147.2	1.09	1.0	0.89	77.0
7	31	2,054.64	2,054.64	0	2,384.00	500.0	500.0	147.4	1.09	1.0	0.89	63.4
Total		21,344.48	21,345.41	0	—	—	489.2	136.6	—	—	—	609.4

(Note 1) 48.18 × 31 = 1,493.58
83.33 × 30 = 2,499.90
(Note 2) TWL 350m, Head loss 2.6 m
(Note 3) Normal effective head 135.4 m

(2) Calculation of quantities of works

1) Dam

Concrete gravity dam is selected.

Dam height (Hd) = 100 m
Crest length (L) = 350 m
Riverbed width (B) = 70 m
Design flood discharge (Q_f) = 4,600 m^3/sec

The design flood discharge is calculated by using the simplified equation described in 6.2.2(1). From the annual rainfall of 1,350 mm of this area, the regional coefficient of T district is (a) = 34.

$A = 1{,}050\ km^2$, $q = 4.4\ m^3/sec/\ km^2$, $Q_f = 4.4 \times 1{,}050 = 4{,}600\ m^3/sec$

Quantities

$Hd^2 \times L = 3{,}500 \times 10^3 > 100 \times 10^3$
$Ve = 10.0 \times Hd \times L = 350{,}000\ m^3$
$Vc = 22.4 Hd^2 \times L = 735{,}000\ m^3$ (B/L = 0.2)
$Wg = 0.13 \times Q_f = 600$ tons

2) Intake

Pressure type

$Q = 230\ m^3/sec$

When two tunnels are constructed, the design discharge per tunnel is 115 m^3/sec Therefore, the inner diameter of the tunnel is D = 6.2 m as shown in Figure 6-2.

$n = 2$, $Q = 230\ m^3/sec$, $D = 6.2$ m, $ha = 35$ m

Quantities

$Ve = 130 \times [\{(ha+D) \times Q\}^{1/2} \times n^{1/3}]^{1.27} = 58{,}200\ m^3$
$Vc = 56.5 \times [\{(ha+D) \times Q\}^{1/2} \times n^{1/3}]^{1.23} = 20{,}900\ m^3$
$Wr = 0.04 \times Vc = 840$ tons
$Wg = 0.9 \times (ha+D)^{1/9} \times Q = 310$ tons
$Ws = 0.5 \times (ha+D)^{1/9} \times Q = 170$ tons

3) Head race

Circular pressure tunnel

D = 6.2 m so R 3.1 m, L = 850 m, n = 2

Quantities

to = 0.55 m as shown in Figure 6-5.
$Ve = 3.2 \times (R+to)^2 \times L \times n = 72{,}000\ m^3$
$Vc = \{3.2 \times (R+to)^2 - \pi R^2\} \times L \times n = 21{,}200\ m^3$

$Wr = 0.04 \times Vc = 850$ tons

4) Surge tank

$Q = 230\ m^3/sec$, $q = Q/n = 115\ m^3/sec$, $ha = 35\ m$, $L = 850m$, $n = 2$

Quantities

$Ve = 38 \times q \times (ha+L)^{1/4} \times n = 47{,}600\ m^3$
$Vc = 11 \times q \times (ha+L)^{1/4} \times n = 13{,}800\ m^3$
$Wr = 0.05 \times Vc = 690$ tons

5) Penstock

Exposed type is selected.

$q = 115\ m^3/sec$, $n = 2$, $H = 500 - 350 = 150m$

Steel pipe inner diameter Dm = 5.1 m (when $q = 115m^3/sec$ as indicated in Figure 6-3)
Steel pipe length L = 155 m
Design head H = 500 - 350 = 150 m

Quantities

$Ve = (20.3 \times D_m^2 - 49.5 \times D_m + 41.3) \times n^{1/3} \times L = 61{,}900\ m^3$
$Vc = (0.5 \times D_m^2 + 2.5 \times D_m) \times n^{1/3} \times L = 5{,}000\ m^3$
$Wr = 0.018 \times Vc = 90$ tons
$tp_1 = 0.0313 \times H \times D_m + 2 = 26$
$Wp = 7.85 \times \pi \times 5.1 \times 26 \times 150 \times 1.15 \times 2 = 1{,}130$ tons

6) Powerhouse

$Q = 230\ m^3/sec$ $He = 135.4m$, $n = 4$

Quantities

$Ve = 97.8 \times (Q \times He^{2/3} \times n^{1/2})^{0.727} = 91{,}100\ m^3$
$Vc = 28.1 \times (Q \times He^{2/3} \times n^{1/2})^{0.795} = 49{,}600\ m^3$
$Wr = 0.05 \times Vc = 2{,}480$ tons

7) Tailrace

Non-pressure type

$Q = 230\ m^3/sec$, $D = 8.0m$ ($R = 4.0m$ extrapolated from Figure 6-2)

Quantities

$Ve = 395 \times (R \times Q)^{0.479} = 10{,}400\ m^3$
$Vc = 40.4(R \times Q)^{0.684} = 4{,}300\ m^3$
$Wr = 0.278 \times Vc^{0.61} = 46$ tons

(3) Total construction cost

Total construction cost is calculated on the above quantities and unit prices. It is estimated to be

$4,529\times10^6$ monetary unit. Calculation conditions are described below, and detailed examples of the calculation are shown in Table A-5-6 and Table A-5-7.

- Cost of preparatory works is calculated at 2% of the civil works cost.
- Electrical equipment cost of the subject country is plotted on the bilateral logarithmic coordinate paper with $P/He^{1/2}$ (P: output (kW), He: effective head (m) as parameter and the cost against $P/He^{1/2} = 23,200$ of this project is calculated.
- The mean interest rate of local and foreign currencies is used..
- Unit prices were taken from past experience.
- The construction period is 4 years.

If this example is a hydropower potential study, transmission line cost is not included. As a result of the study, if the project is recognized as promising, site reconnaissance is conducted and the transmission line or distribution line cost is calculated. In this example, the installed capacity is 270MW and the distance from the powerhouse to the substation (or existing transmission line) is assumed to be 100km, two circuits of 140kV is selected from Figure 6-6 (b) Chapter 6, The transmission line cost is estimated by multiplying the unit price to the line length.

(4) Economic assessment

Economic viability is assessed using the kW value and kWh value obtained in 1. (4).

1) Annual benefit

$$Ph = 270,000 \text{ kW}$$
$$E = 605\times10^6 \text{kWh}$$
$$B = B_1+B_2 = 270,000\times3,640+605\times10^6\times0.428 = 1,240\times10^6 \text{ monetary unit}$$

2) Annual cost

$$C = 4,529\times10^6\times0.11 = 498\times10^6 \text{ monetary unit}$$

3) Benefit cost ratio

$$B/C = 1,240/498 = 2.5$$

Accordingly, the economic viability of this project is excellent.

Table A-5-6 Total Construction Cost (Reservoir Type)

Unit: 10^3 monetary unit

Description	Estimated Cost	Note
1. Preparation works & Land aquisition Cost		
(1) Access Road	45,700	
(2) Compensation & Resettlement		
(3) Camp and Facilities		(3 Civil work) × 0.02
Sub total	45,700	
2. Environmental Mitigation Cost	68,500	(3 Civil work) × 0.03
3. Civil Works		
(1) Care of River	33,000	
(2) Dam	1,647,800	
(3) Spillway	—	
(4) Intake	73,300	
(5) Headrace	113,000	
(6) Surge Tank	63,700	
(7) Penstock	19,900	
(8) Powerhouse	211,800	
(9) Tailrace channel	—	
(10) Tailrace	13,100	
(11) Miscellaneous Works	108,800	((1)～(10)) × 0.05
Sub total	2,284,400	
4. Hydraulic Equipment		
(1) Gate & Screen	95,500	
(2) Penstock	67,800	((1)～(2)) × 0.05
Sub total	163,300	
5. Electro-mechanical Equipment	650,000	
Direct Cost	3,211,900	1 + 2 + 3 + 4 + 5
6. Administration & Engineering fee	481,800	(Direct Cost) × 0.15
7. Contingency	321,200	(Direct Cost) × 0.1
7'. Cost allocation of dam	—	
Total	4,014,900	i = 8%, T = 4 years
8. Interest during Construction	514,100	(Total) × 0.4 × i × T
Total Cost	4,529,000	

Table A-5-7 (1) Civil Works Cost (Reservoir Type)

(10^3Monetary Unit)

Work Items	Unit	Unit Price	Quantity	Total Amount
(1.) Concrete gravity dam				1,680,800
(1) 1 Care of river	L. S.	—	1	33,000
(1) 2 Concrete dam				1,647,800
①Excavation	m^3	80	35,000	28,000
②Concrete	m^3	2,000	735,000	1,470,000
③Others	L. S.	—	1	149,800
(2.) Intake				73,300
①Excavation	m^3	80	58,400	4,672
②Concrete	m^3	2,100	20,900	43,890
③Reinforcement bar	ton	12,000	840	10,080
④Others	L. S.	—	1	14,658
(3.) Headrace				113,000
①Excavation	m^3	600	72,500	43,500
②Concrete	m^3	2,100	21,200	44,520
③Reinforcement bar	ton	12,000	850	10,200
④Others	L. S.	—	1	14,780
(4.) Surge tank				63,700
①Excavation	m^3	80	47,600	3,808
②Concrete	m^3	2,100	13,800	28,980
③Reinforcement bar	ton	12,000	690	8,280
④Others	L. S.	—	1	22,630
(5.) Penstock				19,900
①Excavation	m^3	80	61,900	4,952
②Concrete	m^3	2,100	5,000	10,500
③Reinforcement bar	ton	12,000	90	1,080
④Others	L. S.	—	1	3,368
(6.) Powerhouse				211,800
①Excavation	m^3	80	91,100	7,288
②Concrete	m^3	2,100	49,600	104,160
③Reinforcement bar	ton	12,000	2,480	29,760
④Others	L. S.	—	1	70,592
(7.) Tailrace				13,100
①Excavation	m^3	80	10,400	832
②Concrete	m^3	2,100	4,300	9,030
③Reinforcement bar	ton	12,000	46	552
④Others	L. S.	—	1	2,686
(8.) Miscellaneous work	L. S.	—	1	108,800
Sub total				2,284,400

Table A-5-7 (2) Hydraulic Equipment Cost (Reservoir Type)

(10^3 Monetary Unit)

Work Items	Unit	Unit price	Quantity	Total Amount
1. Dam and spillway				48,000
Gate	ton	80,000	600	48,000
2. Intake				31,600
Gate	ton	80,000	310	24,800
Screen	ton	40,000	170	6,800
3. Penstock	ton	50,000	1,130	56,500
4. Tailrace	ton	—	—	0
5. Others	L.S.	—	20%	27,220
Sub Total				163,300

3. Pondage Type

The main points of the study are described below by following the study procedure described in 5.3.5. The procedure of construction cost estimate and economic evaluation is the same as for the reservoir type and therefore they are omitted.

(1) Planning of project

1) Selection of dam site, powerhouse site and waterway route

The power generation type for Site C is studied using topographic map. As the topography is suitable for a pond or reservoir, the regulating capability factor of runoff (RCF) studied in 2. above gives a value less than 5% and a pondage type is studied. As additional head can be gained with a relatively short headrace, a dam and waterway type is selected for the project.

The dam site, powerhouse site and waterway route are selected according to 5.3.4 (2). (Detail is omitted).

2) Catchment area

It is measured to be 860 km^2 from topographic maps.

3) Calculation of flow at the dam site

The annual mean flow and annual total inflow at the dam site are 55.3 m^3/sec and $1,744 \times 10^6$ m^3 respectively.

4) Storage capacity curve

The storage capacity curve is shown in Figure A-5-9. A curve of reservoir area-elevation is omitted.

5) Sediment volume and sedimentation level

Sedimentation for catchment area of 860 km^2 at the intake site is calculated. The specific sediment volume of a dam is 30 m^3/km^2/year assumed from existing dams nearby, therefore, sedimentation for 100 years is expressed by the following equation;

$$Vs = 30 \times 860 \times 100 = 2.58 \times 10^6 \ m^3$$

Sedimentation level is taken as ELs = EL. 516 m from Figure A-5-9.

6) Preparation of flow duration curve and determination of firm discharge

A flow duration curve prepared from daily flow data at the dam site is shown in Figure A-5-8. Firm discharge is taken as 90% runoff from Figure A-5-8 giving a value of 18.50 m^3/sec corresponding to 90%. The value of 90% discharge is adopted here instead of 95% because of local power requirement.

7) Determination of maximum plant discharge (Q_{max})

The project is planned to generate for peak duration hours of 5 hours. The firm plant discharge is determined to correspond to 5 hours.

$$Q_{max} = Q_f \times 24/5 = 89.0 \ m^3/sec$$

8) Lower limit of low water level

From Figure 5-22, the inner diameter of the pressure tunnel is D = 5.6 m for Q_{max} = 89.0 m^3/sec.

The lower limit of low water level is as follows;

$$LWL = ELs + 1.0 + 5.6 \times 1.5 \doteqdot 526m$$

9) Determination of effective storage capacity, low water level and high water level

The firm discharge is regulated daily, and the regulating capacity to enable 5 hours peaking operation at maximum plant discharge is:

$$\begin{aligned} Ve &= (Q_{max} - Q_f) \times 5 \times 3{,}600 \\ &= (89.0 - 18.5) \times 5 \times 3{,}600 = 1.3 \times 10^6 \ m^3 \end{aligned}$$

The high water level is 534m from low water level EL. 526 m considering the above water volume. The validity of the low water level is checked to ensure the operating range of the turbine.

Calculation of head fluctuation rate (k)

$$NWL = HWL - ha/3 = 534 - (534 - 526)/3 = 531 \ m$$
$$Hg = NWL - TWL = 531 - 400 = 131 \ m$$

$$k = \frac{LWL - TWL}{Hg} = \frac{526 - 400}{131} = 0.96 > 0.7$$

The head fluctuation rate calculated by the above equation exceeds 0.7 which is within the operating range for a Francis turbine. The study is continued for low water level of EL. 526 m and high water level of EL. 534 m.

10) Waterway profile

A profile of the decided waterway route is shown in Figure A-5-10.

11) Head loss and effective head

Effective head is calculated using the following equation for the dam and waterway type of development.

NWL = HWL - ha/3 = 534 - 8/3 = 531.3

Hg = NWL - TWL = 531.3 - 400 = 131.3

Q = 89.0 m^3/sec, L = 700m for headrace length, D = 5.6 m for headrace (Figure 6-2, Chapter 6) ; penstock L = 185 m, D = 4.7 m (Figure 6-3)

$H\ell$ = 700/700+185/200+0.6 = 2.5m

Hes = Hg - $H\ell$ = 131.3－2.5 = 128.8 m

12) Selection of turbine type and combined efficiency

Maximum plant discharge of 89.0 m^3/sec and effective head of 128.5 m are obtained, and arrive at an output of about 100MW. As indicated in Figure 12-16, Chapter 12, a vertical Francis turbine is used. The combined efficiency of turbine generator is 0.89 as indicated in Table 5-2, Chapter 5.

13) Maximum output and firm peak output

$$P = 9.8\times Q_{max}\times Hes\times \eta = 9.8\times 89.0\times 128.8\times 0.89 = 100{,}000 \text{ kW}$$

Firm peak output (Pf) is the same as maximum output because firm discharge is regulated for 5 hours peaking operation.

14) Annual energy generation

Calculation procedure is shown in Table A-5-8.

$$E = 409\times 10^6 \text{kWh}$$

15) Plant factor

$$Pf = \frac{409\times 10^6}{100{,}000\times 8{,}760}\times 100 = 47\%$$

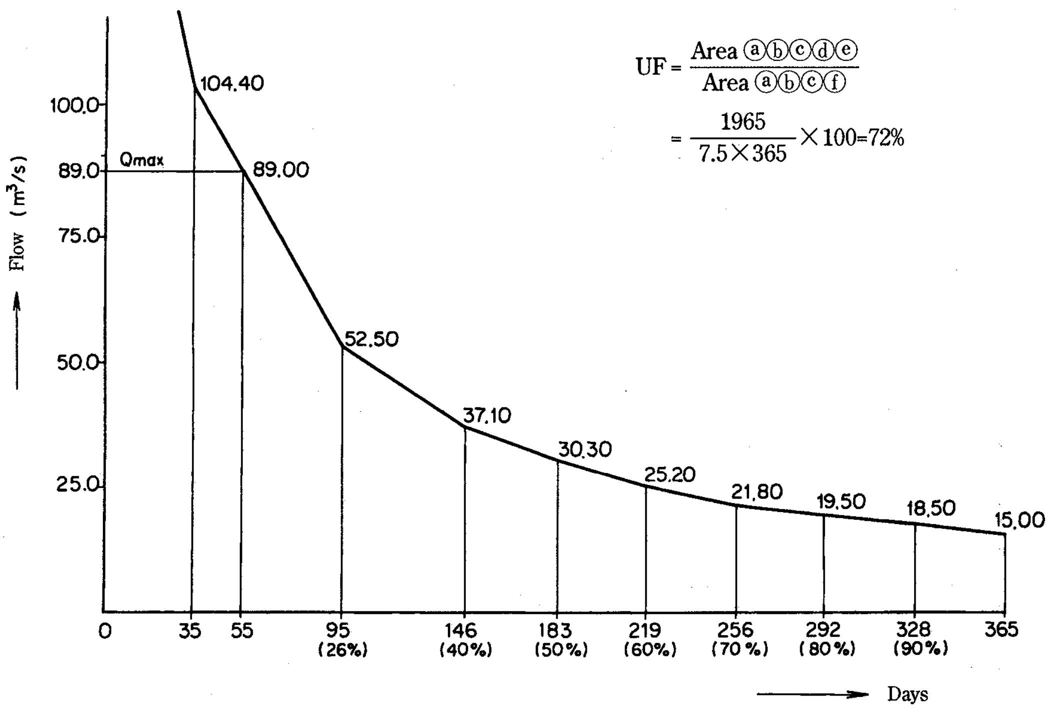

Figure A-5-8 Flow Duration Curve

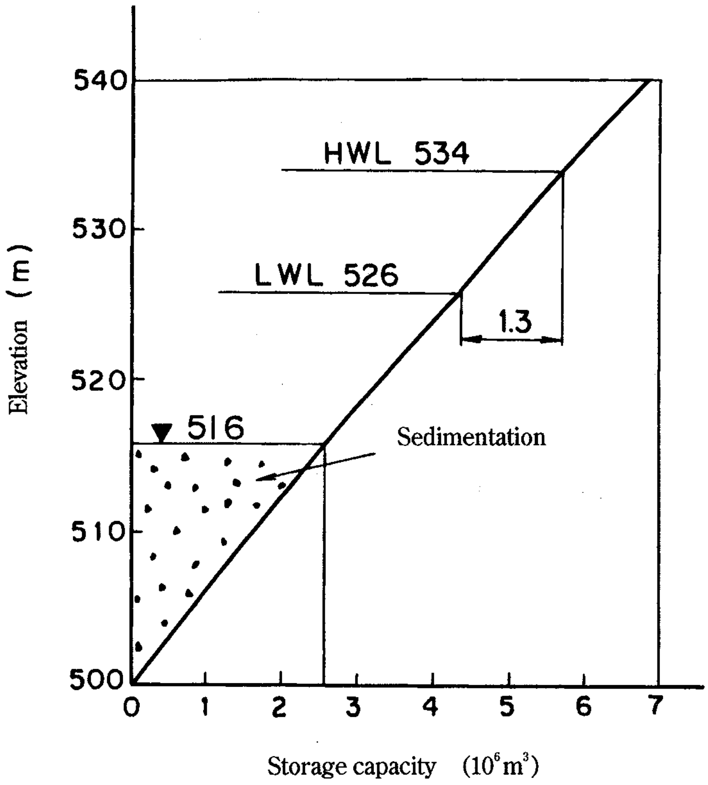

Figure A-5-9 Storage Capacity Curve

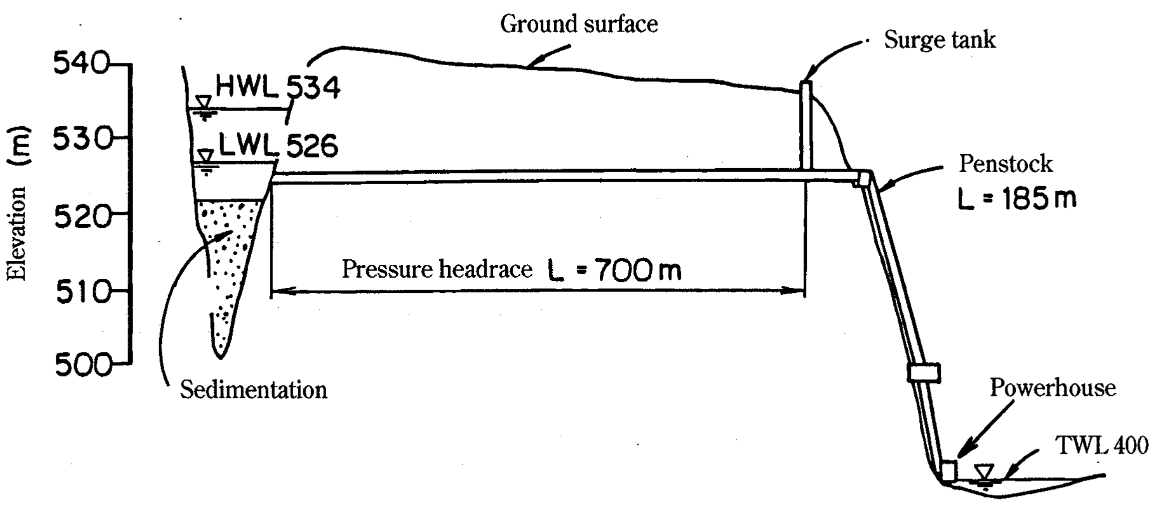

Figure A-5-10 Water Way Profile

Table A-5-8 Energy Calculations (Pondage Type)

(1) Days	(2) Difference of (1)	(3) (m^3/s)	(4) Average of (3) m^3/s	(5) ζ	(6) Q ($10^6 m^3$)	(7) E (10^6kWh)
0		89.00				
55	55	89.00	89.00	0.89	422.9	132.0
95	40	52.50	70.75	0.89	244.5	76.3
146	51	37.10	44.80	0.89	197.4	61.6
183	37	30.30	33.70	0.89	107.7	33.6
219	36	25.20	27.75	0.89	86.3	26.9
256	37	21.80	23.50	0.89	75.1	23.4
292	36	19.50	20.65	0.89	64.2	20.0
328	36	18.50	19.00	0.89	59.1	18.4
365	37	15.00	16.75	0.89	53.6	16.7
					1,310.8	408.9

Note:

Francis turbine ζ: Combined efficiency of turbine generator (See Chapter 5, Table 5-2)

Peaking operation: $\zeta = 0.89$

(6) = (2) × (4) × 24 × 3,600

(7) = 9.8 × (5) × He × (6)/3,600

He = 128.8 m

4. Pumped Storage Type

Study of pumped storage type is in accordance with 5.4. Construction cost estimate and economic evaluation are conducted in the same way as that for a reservoir type explained in 2. above and therefore is omitted here.

(1) Planning of project

1) Selection of dam site

The upper and lower dam sites where high head can be secured are selected using 1:50,000 maps. As shown in Figure A-5-11, the ratio L/H is calculated as follows.

L = 1,800m

H = 840 - 350 = 490 m

L/H = 3.7

where,

L: Horizontal length of upper and lower dam sites (m)

H: Difference of elevation (m) of riverbed between upper and lower dam sites

As L/H is equal almost 4, the study mentioned below is conducted.

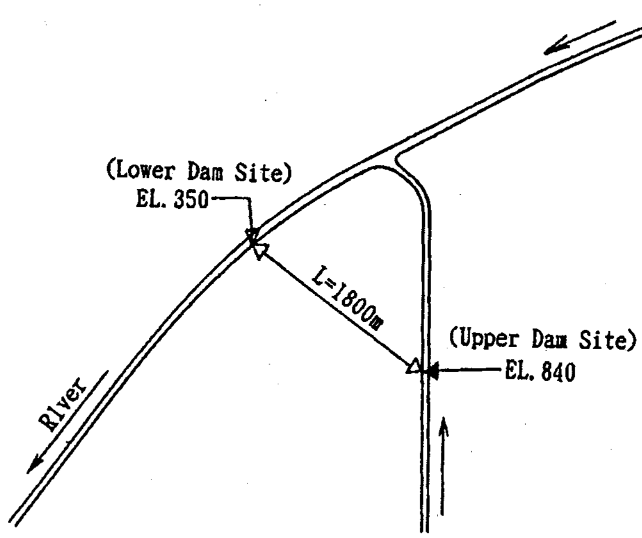

Figure A-5-11 Plan of Project Area

2) Catchment area

The catchment areas of the upper and lower dams are measured. The areas are 31 km^2 for the upper dam and 113 km^2 for the lower dam.

3) Storage capacity curve

The storage capacity curves of both dam sites are prepared from topographic maps

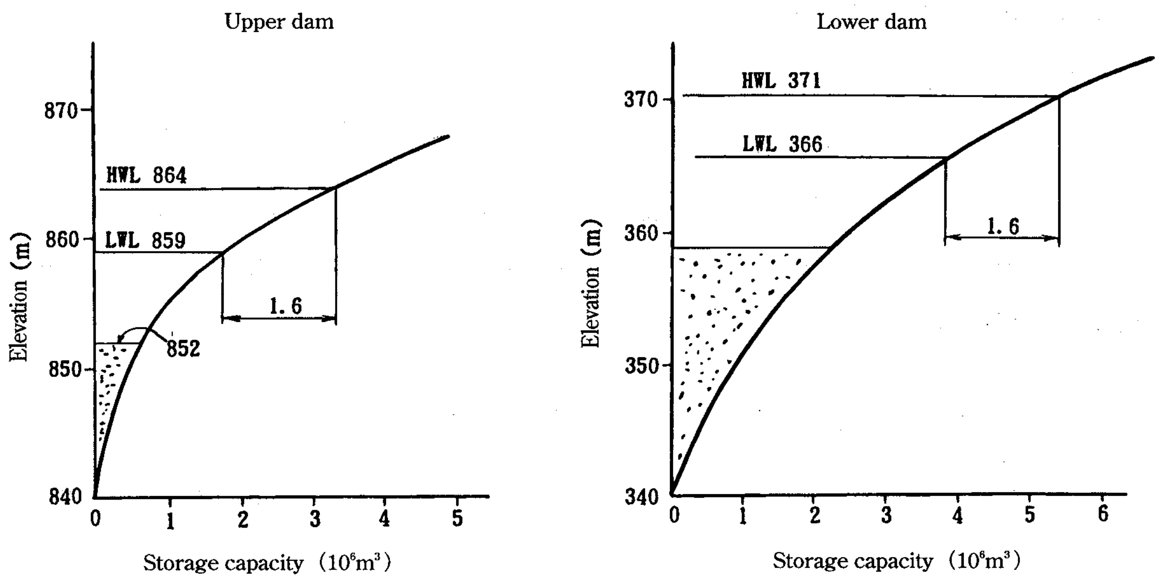

Figure A-5-12 Storage Capacity Curves

4) Tentative setting of maximum plant discharge and effective storage capacity

Required output is determined taking into account the power demand of the power system, and maximum plant discharge is then calculated. In this example, the maximum output is determined to be 300MW, and the maximum plant discharge is computed from the difference of 490 m in elevation between both dam sites.

$$Q_{max} = \frac{300,000}{8.5 \times 490} = 72 \text{ m}^3/\text{sec}$$

Effective storage capacity is calculated to be 1.6×10^6 m^3 for maximum plant discharge of 72 m^3/sec and peak duration hours of 6 hours.

$$Ve = 72\times6\times3,600 = 1.6\times10^6 \text{ m}^3$$

5) Sedimentation

Specific sediment volume is calculated referring to the records of existing dams. In this example, specific sediment volume is assumed to be 200 m^3/ km^2/year. The sediment volume for 100 years is calculated as follows.

Upper dam:

$$Vsu = 200\times31\times100 = 0.6\times10^6 \text{ m}^3$$

Lower dam:

$$Vsd = 200\times113\times100 = 2.2\times10^6 \text{ m}^3$$

6) High water level and low water level

(a) Upper dam

The sedimentation level (ELs) as shown in Figure A-5-12 is EL. 852m for sediment volume of 0.6×10^6 m^3. The inner diameter 3.9m of headrace tunnel is selected for maximum plant discharge of 72 m^3/sec and for a velocity of 6 m/sec. Low water level (LWL) is set at EL. 859m based on the concept of Figure 5-21, Chapter 5.

$$LWL = ELs+1+1.5D = 1+3.9 \times 1.5 = 859 \text{ m}$$

The high water level (HWL) is set at EL. 864 m from the storage capacity curve to achieve an effective storage capacity of 1.6×10^6 m^3

(b) Lower dam

As shown in Figure A-5-12, the sedimentation level (ELs) is EL. 359 m for sediment volume of 2.2 $\times10^6$ m^3. The inner diameter of the tunnel is 3.9 m which is the value obtained above. The LWL is EL. 366m based on the concept in Figure 5-21, Chapter 5.

$$LWL = ELs+1+1.5D = 359+1+3.9 \times 1.5 = 366 \text{ m}$$

The HWL is at EL. 371 m to achieve an effective storage capacity of 1.6×10^6 m^3 from the storage capacity curve.

7) Setting of normal water level and tailwater level

Normal water level:

$$NWL = 864 - (864 - 859)/3 = 862.3 \text{ m}$$

Normal tailwater level:

$$TWL = 366 + (371 - 366) \times 2/3 = 369.3 \text{ m}$$

8) Waterway profile

Turbine center is set as follows.

Maximum pumping head = HWL of upper pond - LWL of lower pond + head loss

= 864 -366 = 498 m (head loss ignored)

Draft head is 58 m as shown in Figure 5-37, Chapter 5.

Elevation of turbine center = LWL of lower pond - draft head = 366 - 58 = 308 m

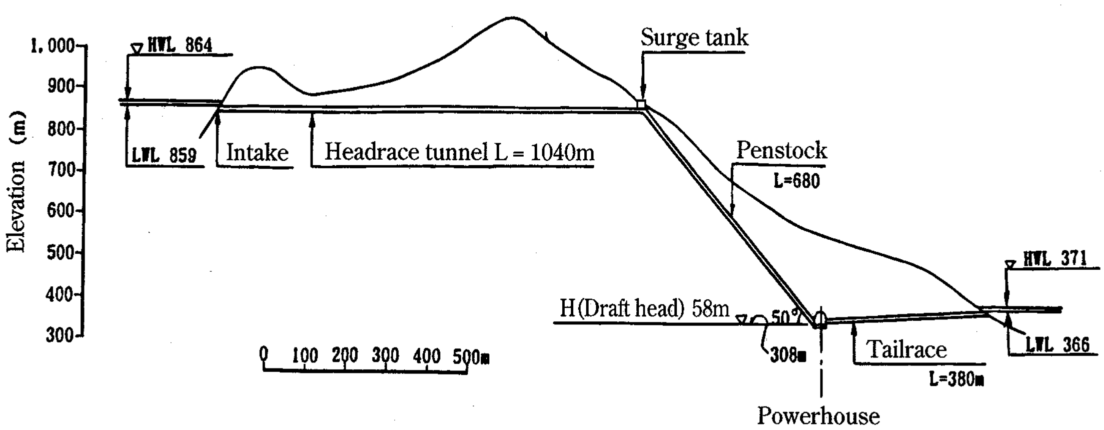

Figure A-5-13 Waterway Profile

9) Head loss and effective head

$$Hg = NWL - TWL = 862.3 - 369.3 = 493.0 \text{ m}$$

$$H\ell = 1{,}040/300 + 680/100 + 380/300 + 2 = 14 \text{ m}$$

$$Hes = 493.0 - 14 = 479 \text{ m}$$

10) Maximum plant discharge

$$Q_{max} = \frac{300{,}000}{8.5 \times 479} = 74 \text{ m}^3/\text{sec}$$

11) Annual energy generation

Annual operating hours is assumed to be 800 hours.

$$E = P_{max} \times T = 300{,}000(\text{kW}) \times 800(\text{hr}) = 24 \times 10^6 \text{ kWh}$$

12) Pumping energy

Pumping energy is calculated as follows.

$$Ep = E/0.7 = 34 \times 10^6 \text{ kWh}$$

13) Project outline

The project features are determined from the above as follows.

- Maximum output: 300MW
- Maximum plant discharge: 74 m^3/sec
- Effective head: 479 m
- Upper dam: High water level EL. 864m, low water level EL. 859 m, effective storage capacity 1.6×10^6 m^3
- Lower dam: High water level EL. 371 m, low water level EL. 366 m, effective storage capacity 1.6×10^6 m^3

(2) Construction cost estimate, and economic evaluation

As the details of the plan are determined, the construction cost is calculated based on 6.2.4, and the economic evaluation is made based on 6.3. The methods is not explained (refer to reservoir type.).

A-9-1 Study on Period Setting of Runoff Record for Hydropower Planning

1. Outline

In order to evaluate hydropower resources properly, standpoints to utilize hydropower resources effectively and to minimize uncertainty of investment recovery are important. The standpoints relate to the period setting of runoff record. As a result of the study mentioned bellow, it can be said that the period of 20 to 30 years is recommendable for hydropower planning.

- Period of runoff record should be able to express hydrological characteristics on average for entire service life of the project.
- Period of runoff record should be able to keep the intrinsic uncertainty in hydropower projects within acceptable range.

2. Period to be able to express long term hydrological characteristics on average

Annual rainfall (precipitation) records from 1901 to 1998 are used for analysis. These records were taken 28 meteorological stations which are located in monsoon area in Asia and Africa as shown in Figure A-9-1.

(1) Analysis on Periodic component

The periodic component is analyzed by a spectrum analysis. Since the periodic component is recognized for only 4 stations, the period setting of runoff record is not affected by this periodic component.

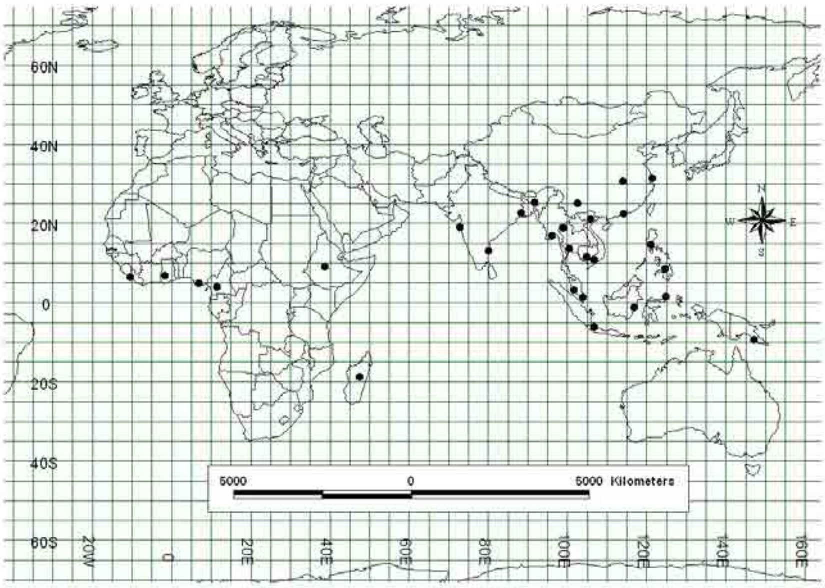

Figure A-9-1 Location of meteorological stations

(2) Analysis on moving average

A characteristic of moving average is that the value of moving average is getting close to average of the sample in response to a length of the setting period of the moving average. The period to be able to express river flow on average for long period is studied by utilizing the characteristic.

Figure A-9-2 shows a schematic figure to express a relation among average(μ) of the samples, 95% confidence interval (μ±Δ)and N-year moving average. Since an example in Figure A-9-2 shows a case of 10-year moving average (dotted line).

In case, the values of N-year moving average are in the range of 95 % confidence interval, it can be said that the period of N-year expresses the average rainfall.

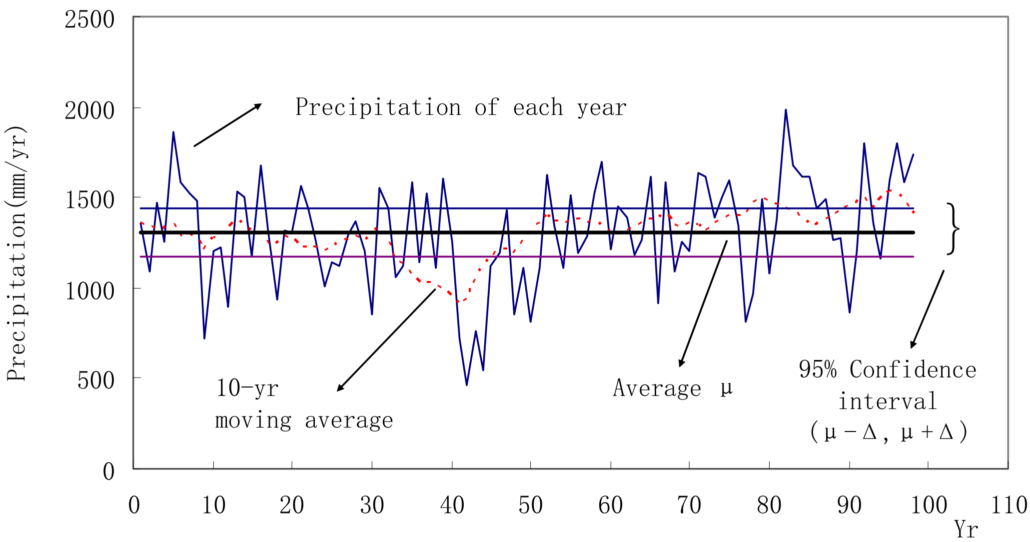

Figure A-9-2 Schematic figure on moving average

Period of N-year is calculated by the following formula using standard deviation, required interval （Δ） from 98 samples.

$$N = \lambda^2{}_{\beta \bullet t}\left(\frac{s}{\Delta}\right)^2$$

where,

N	: number of years required for moving average
$\overline{X}$	: mean of sample (grouping of 98 years' annual rainfall data)
s	: standard deviation (grouping of 98 years' annual rainfall data)
Δ	: interval from average value (Δ=k $\overline{X}$), 2 cases of k=5% and K=10%
$\lambda_{\beta \bullet t}$	: value of t-distribution obtained from β=5%and number of sample size.

Figure A-9-3 shows the result of an analysis for k=5% and K=10%. All 28 stations can satisfy the condition of a 20 year period in the case of K=10%. However in the case of 5% values of N-years scatter from about 10 years to 70 years. Taking into account the acceptable value of k=10% for a feasibility study, runoff records of 20 years can express meaningful period for hydropower projects.

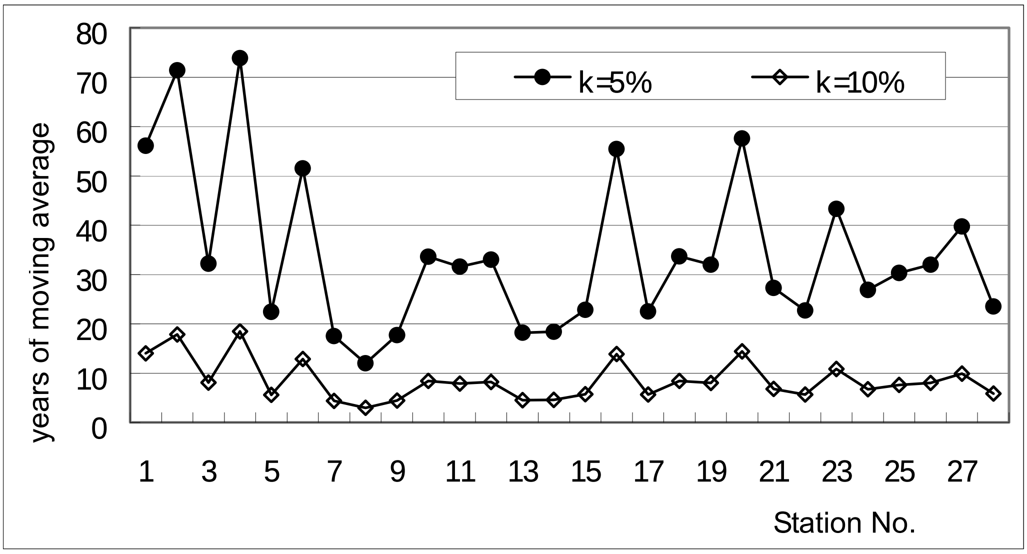

Figure A-9-3 Characteristics of rainfall data expressed by moving average

3. Period to keep uncertainty of investment recovery within an acceptable range

Runoff records having more than 50 years records and simulated data by random number are used for the study. The former records are taken from 7 gaging stations in Japan and the latter data are generated by positive correlation between rainfall and river flow.

(1) Analysis of energy decreasing ratio caused by the length of runoff records

Benefit of hydropower projects for economic evaluation is estimated from power (kW) and energy (kWh). Considering a reservoir type hydropower, energy (kWh) is used to evaluate the hydrological uncertainty. Shortage of expected energy at the operation stage against the expected energy at the planning stage becomes a risk of the project. Therefore energy decreasing ratio (EDR) is defined here as follows (See Figure A-9-4).

EDR＝（ET－E50）/ET

where,

EDR : energy decreasing ratio

ET : annual energy production calculated from T-years runoff record at planning stage

E50 : annual energy production calculated from 50 years runoff record at operation stage

Since the energy production depends on water volume used for power generation, the water volume by using flow duration curve of a 50 year record is used to calculate EDR.

Ri(n,T)＝Area[KHCDE(n,T)]÷Area[AOBDE(n,T)]

where,

Flow duration curve: (1) 50 years records, (2) T-years records, (3) extended records to 50 years from T-years

Qt : maximum plant discharge

Ri(n,T) : EDR(%) for start of runoff record is n-year, period of the record is T-year

Area[AOBDE(n,T)]: annual water volume (m^3/sec×year)corresponding to expected value at planning stage

Area[KHCDE(n,T)]: Shortage of annual energy (m^3/sec×year)

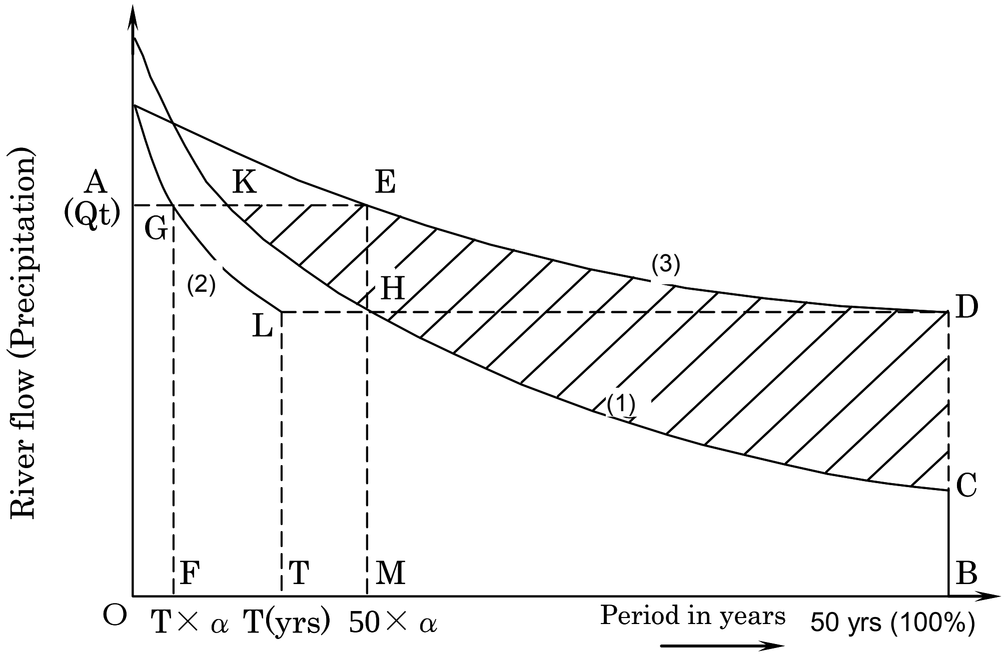

Figure A-9-4 Schematic figure for EDR calculation

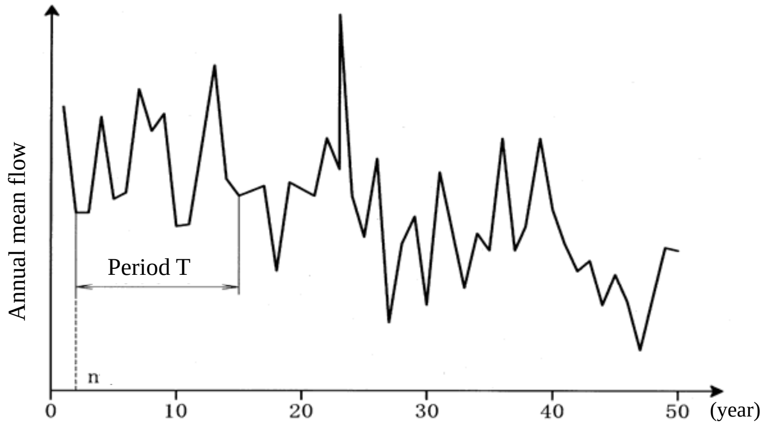

Figure A-9-5 Schematic figure for period of T-years and start of the record

(2) Analysis on the maximum decreasing ratio

The flow duration curves of seven gaging stations are made from observed daily data for 50 years which corresponds to the period used for economic analysis of hydropower projects.

- Period of T-years is fixed and starting year is changed as shown in Figure A-9-1-5, and 50 values of EDR are calculated. The maximum value from 50 EDRs is defined as Max.EDR for T years.
- In the same way, Max.EDR for other T values is calculated by changing the period of T.
- Acceptable range of the Max.EDR is assumed 10%, taking into account the accuracy

required for a feasibility study.

The result of the analysis for actual gaging stations is shown in Figure A-9-1-6(left), and for simulated data groups is shown in Figure A-9-1-6(right). It could be said that the required period is about 20 to 30 years.

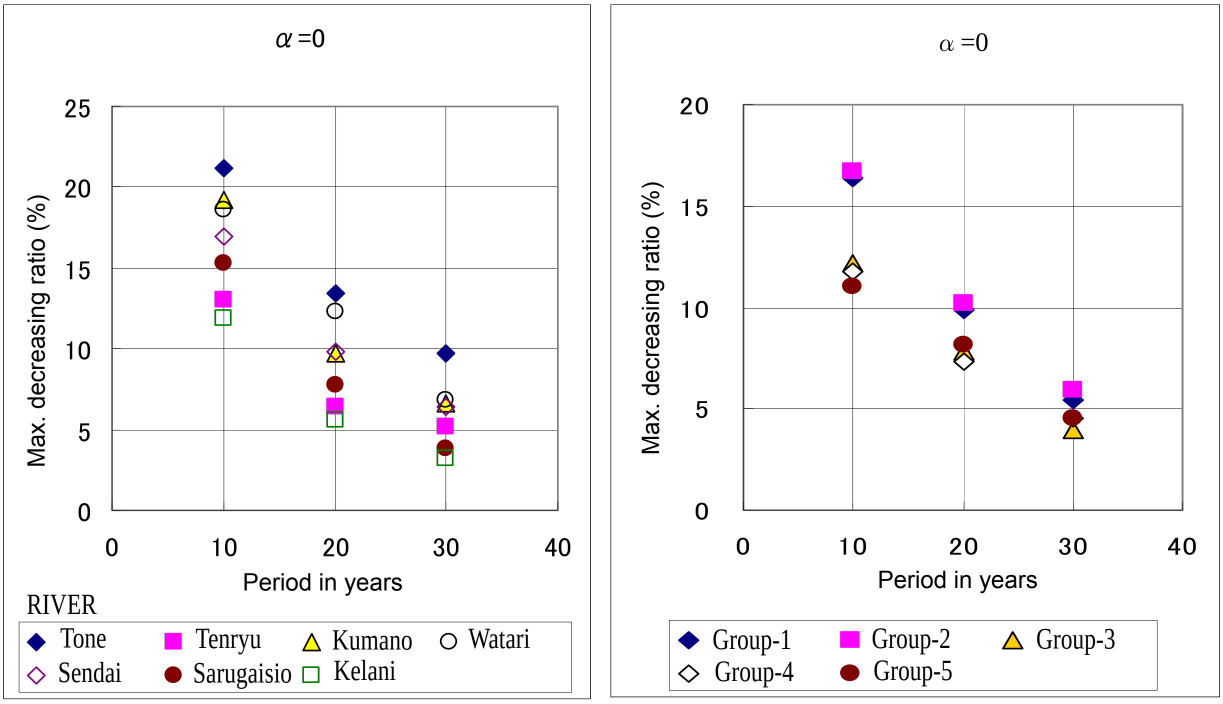

Figure A-9-6 Max.EDR obtained from river flow

4. Period setting of river flow data for hydropower planning

The following is summarized under the condition of an assumed acceptable accuracy of 10% for feasibility study.

- All the station can satisfy the required condition for 20 year moving average.
- Maximum EDR is in the range of 20 to 30 years for almost all stations.

The period of 20-30 years is the most preferable for hydropower projects in developing countries, although that of more than 10 years is used in Japan.

Source: Study on evaluation of hydropower resources in developing countries(in Japanese), J.TANI, Tokyo Institute of Technology, 2005

A-11-1(1): Structure of Zoned Fill Dam (Japan)

Dam	Completion year	Dam height (m)	Crest length (m)	Crest width (m)	Core width (m)		Filter min. witdth (m)		B/H	Extra embankment (m)	Slope gradient		Remarks
					Max.	Min.	Upstream	Downstream			Upstream	Downstream	
Miboro Dam	1960	131.0	405.0	12.0	101.05	5.0	2.00	2.00	0.77	2.7	2.5	1.75	Inclined core
Makio Dam	1961	106.0	264.0	10.0	25.0	6.0	4.00 (Equal width)	4.00 (Equal width)	0.24	2.0	3.0	2.25	Center core
Kuzuryu Dam	1968	128.0	355.0	12.0	55.0	4.0	2.00	2.00	0.43	1.15	2.6～3.0	1.8	Inclined core
Kisenyama Dam	1969	95.0	270.0	11.0	51.0	4.0	Fine 1.50 Coarse 2.00	Fine 1.50 Coarse 2.00	0.54	2.0	2.5 3.0	2.2	Center core
Shimokotori Dam	1973	119.0	289.2	11.0	49.7	3.0	Fine 1.00 Coarse 2.00	Fine 1.00 Coarse 2.00	0.42	2.0	2.3 2.4	1.83～ 1.85	〃
Niikappu Dam	1973	102.8	326.0	11.0	45.12	4.0	2.00	2.00	0.44	2.0	2.3	1.9	〃
Iwaya Dam	1976	127.5	366.0	10.0	42.25	4.0	10.25 (Equal width)	1.50	0.33	2.0	2.5	2.0	Inclined core
Takase Dam	1978	176.5	362.0	14.0	94.0	6.0	2.00 Transition	Fine 2.00 Coarse 2.00	0.53	3.0	2.6	2.1	Center core
Nanakura Dam	1978	125.0	340.0	12.0	62.0	6.0	Fine 3.00 Coarse 2.00	Fine 3.00 Coarse 2.00	0.50	1.5	2.7	2.0	〃
Seto Dam	1978	110.5	342.8	11.0	37.15	4.0	2.00	2.00	0.34	1.0	2.5	2.0	〃
Miho Dam	1978	95.0	590.0	15.0	39.25	6.0	3.00	3.00	0.41	2.0	3.4	2.8	Inclined core
Tedorigawa Dam	1978	153.0	420.0	12.0	67.5	6.0	6.00 (Equal width)	6.00 (Equal width)	0.44	4.0	2.6	1.85	Center core
Tokachi Dam	1984	84.3	443.0	12.0	46.0	6.0	6.00 (Equal width)	6.00 (Equal width)	0.55	2.0	2.6	2.0	〃
Takami Dam	1983	120.0	435.0	11.0	46.6	4.0	2.00	2.00	0.39	1.2	2.65	1.95	〃

A-11-1(2): Structure of Zoned Fill Dam (Outside Japan)

Dam	Country	Completion year	Dam Height H: (m)	Crest Length (m)	Core Width (m)		Filter Min. Width(m)		B/H	Extra Embankmrnt (m)	Slope Gradient		Remarks
					Max:B	Min	Upstream	Downstream			Upstream	Downstream	
Wadi Arab Dam	Jordan	1986	82.5	434	51.6	4.0	3.0	6.0	0.63	1.5	2.5	2.3	Center Core
Sutami Dam	Indonesia	1972	97.0	823	50.0	5.0	1.0	1.0	0.52	–	2.2	2.0	Center Core
Wonorejo Dam	Indonesia	1991	97.0	500	40.0	5.0	1.0	1.0	0.41	–	2.4	2.0	Center Core
Soyan Gang Dam	Korea	1973	123.0	530	64.6	5.0	13.0	13.0	0.52	2.0	2.3	2.0	Center Core
Imha Dam	Korea	1992	73.0	515	28.0	4.0	4.0	4.0	0.38	1.0	2.2	1.8	Center Core
Kulekhani Dam	Nepal		147.0	397	41.7	6.0	2.0	2.0	0.28	1.7	2.0	1.8	Center Core
Samanalawewa Dam	Sri Lanka		100.0	480	64.0	4.0	2.0	5.0	0.64	–	2.1	1.8	Center Core
Tsengwen Dam	Taiwan		133.0	405	124.0	4.0	5.0	8.0	0.93	–	3.0	2.5	
Saguling Dam	Indonesia		97.5	301.4	47.0	5.0	3.0	3.0	0.48	2.0	2.6	1.9	Center Core
Srinagarind Dam	Thai land	1978	135.0	610	53.3	6.0	2.0	2.0	0.39	–	2.0	1.8	Center Core
Hasan Ugurln Dam	Turkey	1983	175.0	405	69.0	6.0	2.5	2.5	0.39	2.0	2.5	1.8	Inelined Core
Altinkaya Dam	Turkey	1988	195.0	619	87.8	7.0	5.0	5.0	0.45	2.0	2.2	1.9	Center Core

A-11-2 Fill Dams with Concrete Facing Membrane

Dam	Country	Height (m)	Crest length (m)	Thickness of concrete membrane		Slope gradient	
				Max. (cm)	Min. (cm)	Upstream	Downstream
McKay	US	48.8	822.9	32	20	1 : 1. 75	1 : 2. 00
Don Martin	Mexico	30.5	1,219.2	30	20	1 : 1. 75	1 : 2. 00
Bucks Creek	US	39	304.8	—	—	1 : 1. 40	1 : 1. 50
Dix River	US	83.8	310.9	46	20	1 : 1. 2～1.0	1 : 1. 40
Salt Springs	US	100	396.2	91	30	1 : 1. 4～1.1	1 : 1. 40
Lower Bear River No.1	US	74.7	292.6	91	30	1 : 1. 30	1 : 1. 40
Lemolo No.1	US	36.6	236.2	46	30	1 : 1. 30	1 : 1. 30
Paladela	Portugal	110	—	110	30	1 : 1. 30	1 : 1. 30
Wishong	US	90.2	1,015.0	87	30	1 : 1. 3～1.0	1 : 1. 40
Courtright	US	96.6	274.3	91	30	1 : 1. 3～1.0	1 : 1. 40
Pinzanes	Mexico	55.3	—	55	30	1 : 1. 20	1 : 1. 30
Ishibachi	Japan	53	345	60	30	1 : 1. 4～1.2	1 : 1. 50
Nozori	Japan	44	152.5	60	40	1 : 1. 30	1 : 1. 30
Quoich	Britain	38.4	320	38	30	1 : 1. 30	1 : 1. 40
Grandes-Patures	France	20	240	150	30	1 : 0. 40	1 : 1. 00
Greziolles	France	30	126	20	50	1 : 0. 99～0.84	1 : 1. 00
Sassiere	France	30	300	30	20	1 : 1. 40	1 : 1. 40
Bakhadda	Algeria	60	220	70	30	1 : 1. 30	1 : 2. 50
New Exchequer	US	149	378	—	70	1 : 1. 30	1 : 1. 40
Cabin Creek Upper	US	62	454	—	—	1 : 1. 10	1 : 1. 75
Fades	France	68	235	—	—	1 : 1. 30	1 : 1. 30
Rama	Yugoslavia	103	230	—	—	1 : 1. 10	1 : 1. 30
Kangaroo Creek	Australia	59	178	—	—	1 : 1. 30	1 : 1. 30
Pindari	Australia	45	487	—	—	1 : 1. 30	1 : 1. 30
Cethana	Australia	110	213	30	—	1 : 1. 30	1 : 1. 30
Alto Anchicaya	Colombia	140	240	30	30	1 : 1. 40	1 : 1. 40
Chuza (Golillas)	Colombia	135	106	—	30	1 : 1. 60	1 : 1. 60
Yacambu	Venezuela	158	107	—	—	1 : 1. 50	1 : 1. 50
Foz do Areia	Brazil	153	830	—	—	1 : 1. 40	1 : 1. 40
Khao Laem	Thailand	141	980	30	30	1 : 1. 60	1 : 1. 60
Machintosh	Australia	175	877	—	—	1 : 1. 30	1 : 1. 30
Murchison	Australia	89	210	—	—	1 : 1. 30	1 : 1. 30
Sugarloaf	Australia	85	1,000	—	—	1 : 1. 50	1 : 1. 70
Cirata	Indonesia	126	453	72	45	1 : 1. 50	1 : 1. 50

A-11-3 Fill Dams with Asphalt Facing Membrane

Name	Completion year	Country	Storage capacity (10^6m^3)	Dam height (m)	Watertight facing surface area (10^3m^2)	Upstream slope gradient	Thickness of asphalt membrane (cm)	Middle drain layer thickness (cm)
Zoccolo	1965	Italy	33	66	42	1 : 2.5,1 : 2	20	—
Silvergrund	1964	East Germany	0.19	12	2.0	1 : 1. 7	26	10
Kessenhamm	1964	West Germany	0.3	18	3.8	1 : 2	14	—
Kruth-Wildenstein	1964	France	21	35	13	1 : 1. 5	29.5, asphalt 7	—
Ulmbach	1965	West Germany	0.8	20	6.3	1 : 1. 8	15	—
Moravka	64/66	Czechoslovakia	11	39	25	1 : 1. 75	28	—
Ohra	1966	East Germany	20	59	22	1 : 2	27	10
Innerste	1966	West Germany	20	35	39	1 : 1. 75	31	10
Ste-Cecile d'Andorge	1966	France	21	45	8.0	1 : 1. 7	22	—
Upper Blue River	1966	US	2.6	22	6.2	1 : 1. 7	25	—
Homestake	1967	US	55	69	52	1 : 1. 6	35～17.5	—
Magosawa	1967	Japan	6.9	13	8.0	1 : 3, 1 : 2	16	—
Ronkhausen	1967	West Germany	1.3	27	9.0	1 : 1. 8	11	—
Nagold	1967	West Germany	5.5	31	8.0	1 : 2	25	7
Kindaruma	1967	Kenya	17	28	15	1 : 1. 7	28	8
Trapan	1967	France	1.3	24	6.0	1 : 2. 5	28	—
Otsumata	1968	Japan	1.8	52	11	1 : 1. 7	30	8
Villarino	1968	Spain	2,475	23	51	1 : 1. 75	17	6
Salagou	1969	France	170	52	20	1 : 1. 5	32	—
Pedu	1969	Malaysia	1,047.8	61	15	1 : 1. 7	15	—
Manzanares el Real	1969	Spain	40	40	32	1 : 1. 75	27	8
Legadadi	1969	Ethiopia	40	25	13	1 : 1. 55	15	—
Grane	1969	West Germany	45	67	39	1 : 1. 75	20	—
Alesani	1969	France	11	65	13	1 : 1. 7, 1 : 1. 6	22	—
Dungonnel	1969	Northern Ireland	1.1	17	4.2	1 : 1. 7	31	13
Coo-Trois Ponts	1969	Belgium	8	20	9.0	1 : 2	13～17	—
Coo-Trois Ponts	1969	Belgium		25	12	1 : 2	17～21	6
Gijon	1969	Spain	2.5	15	15	1 : 2. 35～1 : 1. 95	30	12
Abono	1969	Spain		17	15	1 : 2. 35	29	10
Diga di Saretto	1969	Italy	0.25	13	1.5	1 : 2		—
Ninokura	1971	Japan	2.8	37	7.0	1 : 2	27	10
Poza Honda	1970	Ecuador	98	40	25	1 : 2. 5	28	8
Ry de Rome	1970	Belgium	0.8	22	4.0	1 : 1. 85	31	8
Nidda	1970	West Germany	7	33	16	1 : 1. 6	12.5	—
Ponte Liscione	1970	Italy		60	50	1 : 2	34	10
Obernau	1971	West Germany	15	60	28	1 : 1. 93	26	8
Vallon d'Ol	1971	France	2.8	45	16	1 : 2	22	—
Schombach	1971	East Germany		14	30	1 : 2. 5	28	8
Miyama	1974	Japan	25.8	76	41	1 : 1. 9	42	15
Tataragi	1974	Japan	19.44	65	30	1 : 1. 80	33	8

A-15-1: Screening Format of JICA

Name of Proposed Project:

Project Executing Organization, Project Proponent. or Investment Company:

Name, Address, Organization, and Contact Point of a Responsible Officer:

Name:

Address:

Organization:

Tel:

Fax:

E-Mail:

Date:

Signature:

Check Items

Please write “to be advised (TBA)” when the details of a project are yet to be determined.

Question 1: Address of project site

Question 2: Scale and contents of the project (approximate area, facilities area, production, electricity generated, etc.)

2-1. Project profile (scale and contents)

2-2. How was the necessity of the project confirmed?

Is the project consistent with the higher program/policy?

□YES: Please describe the higher program/policy.

( )

□NO

2-3. Did the proponent consider alternatives before this request?

□YES: Please describe outline of the alternatives

( )

□NO

2-4. Did the proponent implement meetings with the related stakeholders before this request?

□Implemented □Not implemented

<u>If implemented</u>, please mark the following stakeholders.

□Administrative body
□Local residents
□NGO
□Others（ ）

2-2 Does the project include any of the following items?
Yes No

If yes, please mark the items included in the project.
Involuntary resettlement (scale: households persons)
Groundwater pumping (scale: m3/year)
Land reclamation, land development, and/or land-clearing (scale: hectors)
Logging (scale: hectors)

Question 3:
Is the project a new one or an ongoing one? In the case of an ongoing project, have you received strong complaints or other comments from local residents?
New Ongoing (with complaints) Ongoing (without complaints)
Other ( )

Question 4:
Is an Environmental Impact Assessment (EIA), including an Initial Environmental Examination (IEE) Is, required for the project according to a law or guidelines of a host country? If yes, is EIA implemented or planned? If necessary, please fill in the reason why EIA is required.
☐Necessity (□Implemented □Ongoing/planning)
(Reason why EIA is required:)
☐Not necessary
☐Other (please explain)

Question 5:
In the case that steps were taken for an EIA, was the EIA approved by the relevant laws of the

host country? If yes, please note the date of approval and the competent authority.

Approved without a supplementary condition	Approved with a supplementary condition	Under appraisal

(Date of approval: Competent authority:)

Under implementation

Appraisal process not yet started

Other ( )

Question 6:

If the project requires a certificate regarding the environment and society other than an EIA, please indicate the title of said certificate. Was it approved?

Already certified

Title of the certificate: ( )

Requires a certificate but not yet approved

Not required

Other ( )

Question 7:

Are any of the following areas present either inside or surrounding the project site?

Yes No

If yes, please mark the corresponding items.

National parks, protection areas designated by the government (coastline, wetlands, reserved area for ethnic or indigenous people, cultural heritage)

Primeval forests, tropical natural forests

Ecologically important habitats (coral reefs, mangrove wetlands, tidal flats, etc.)

Habitats of endangered species for which protection is required under local laws and/or international treaties

Areas that run the risk of a large scale increase in soil salinity or soil erosion

Remarkable desertification areas

Areas with special values from an archaeological, historical, and/or cultural points of view

Habitats of minorities, indigenous people, or nomadic people with a traditional lifestyle, or areas with special social value

Question 8:

Does the project include any of the following items?

Yes No

If yes, please mark the appropriate items.

Involuntary resettlement (scale: households persons)

Groundwater pumping (scale: m3/year)

Land reclamation, land development, and/or land-clearing (scale: hectors)

Logging (scale: hectors)

Question 9:

Please mark related environmental and social impacts, and describe their outlines.

☐Air pollution
☐Water pollution
☐Soil pollution
☐Waste
☐Noise and vibrations
☐Ground subsidence
☐Offensive odors
☐Geographical features
☐Bottom sediment
☐Biota and ecosystems
☐Water usage
☐Accidents
☐Global warming

☐Involuntary resettlement
☐Local economies, such as employment, livelihood, etc.
☐Land use and utilization of local resources
☐Social institutions such as social infrastructure and local decision-making institutions
☐Existing social infrastructures and services
☐Poor, indigenous, or ethnic people
☐Misdistribution of benefits and damages
☐Local conflicts of interest
☐Limitation of accessibility to information, meetings, etc. on a specific person or group
☐Gender
☐Children's rights
☐Cultural heritage
☐Infectious diseases such as HIV/AIDS
☐Other ( )

Outline of related impact:

Question 10:

In the case of a loan project such as a two-step loan or a sector loan, can sub-projects be specified at the present time?

☐Yes ☐No

Question 11:

Regarding information disclosure and meetings with stakeholders, if JICA's environmental and social considerations are required, does the proponent agree to information disclosure and meetings with stakeholders through these guidelines?

☐Yes ☐No

A-15-2: JICA Check list for Hydropower (JICA Web site)

Category	Environmental Item	Main Check Items	Yes: Y No: N	Confirmation of Environmental Considerations (Reasons, Mitigation Measures)
1 Permits and Explanation	(1) EIA and Environmental Permits	(a) Have EIA reports been already prepared in official process? (b) Have EIA reports been approved by authorities of the host country's government? (c) Have EIA reports been unconditionally approved? If conditions are imposed on the approval of EIA reports, are the conditions satisfied? (d) In addition to the above approvals, have other required environmental permits been obtained from the appropriate regulatory authorities of the host country's government?	(a) (b) (c) (d)	(a) (b) (c) (d)
	(2) Explanation to the Local Stakeholders	(a) Have contents of the project and the potential impacts been adequately explained to the Local stakeholders based on appropriate procedures, including information disclosure? Is understanding obtained from the Local stakeholders? (b) Have the comment from the stakeholders (such as local residents) been reflected to the project design?	(a) (b)	(a) (b)
	(3) Examination of Alternatives	(a) Have alternative plans of the project been examined with social and environmental considerations?	(a)	(a)

Category	Environmental Item	Main Check Items	Yes: Y No: N	Confirmation of Environmental Considerations (Reasons, Mitigation Measures)
2 Pollution Control	(1) Water Quality	(a) Does the water quality of dam pond/reservoir comply with the country's ambient water quality standards? Is there a possibility that proliferation of phytoplankton and zooplankton will occur? (b) Does the quality of water discharged from the dam pond/reservoir comply with the country's ambient water quality standards? (c) Are adequate measures, such as clearance of woody vegetation from the inundation zone prior to flooding planned to prevent water quality degradation in the dam pond/reservoir? (d) Is there a possibility that reduced the river flow downstream will cause water quality degradation resulting in areas that do not comply with the country's ambient water quality standards? (e) Is the discharge of water from the lower portion of the dam pond/reservoir (the water temperature of the lower portion is generally lower than the water temperature of the upper portion) planned by considering the impacts to downstream areas?	(a) (b) (c) (d) (e)	(a) (b) (c) (d) (e)
	(2) Wastes	(a) Are earth and sand generated by excavation properly treated and disposed of in accordance with the country's regulations?	(a)	(a)
3 Natural Environment	(1) Protected Areas	(a) Is the project site located in protected areas designated by the country's laws or international treaties and conventions? Is there a possibility that the project will affect the protected areas?	(a)	(a)
	(2) Ecosystem	(a) Does the project site encompass primeval forests, tropical rain forests, ecologically valuable habitats (e.g., coral reefs, mangroves, or tidal flats)? (b) Does the project site encompass the protected habitats of endangered species designated by the country's laws or international treaties and conventions? (c) Is there a possibility that the project will adversely affect downstream aquatic organisms, animals, plants, and ecosystems? Are adequate protection measures taken to reduce the impacts on the ecosystem? (d) Is there a possibility that installation of structures, such as dams will block the movement of the migratory fish species (such as salmon, trout and eel those move between rivers and sea for spawning)? Are adequate measures taken to reduce the impacts on these species?	(a) (b) (c) (d)	(a) (b) (c) (d)

Category	Environmental Item	Main Check Items	Yes: Y No: N	Confirmation of Environmental Considerations (Reasons, Mitigation Measures)
	(3) Hydrology	(a) Is there a possibility that hydrologic changes due to the installation of structures, such as weirs will adversely affect the surface and groundwater flows (especially in "run of the river generation" projects)?	(a)	(a)
	(4) Topography and Geology	(a) Is there a possibility that reductions in sediment loads downstream due to settling of suspended particles in the reservoir will cause impacts, such as scouring of the downstream riverbeds and soil erosion? Is there a possibility that sedimentation of the reservoir will cause loss of the storage capacity, water logging upstream, and formation of sediment deposits at the reservoir entrance? Are the possibilities of the impacts studied, and adequate prevention measures taken? (b) Is there a possibility that the project will cause a large-scale alteration of the topographic features and geologic structures in the surrounding areas (especially in run of the river generation projects and geothermal power generation projects)?	(a) (b)	(a) (b)

Category	Environmental Item	Main Check Items	Yes: Y No: N	Confirmation of Environmental Considerations (Reasons, Mitigation Measures)
4 Social Environment	(1) Resettlement	(a) Is involuntary resettlement caused by project implementation? If involuntary resettlement is caused, are efforts made to minimize the impacts caused by the resettlement? (b) Is adequate explanation on compensation and resettlement assistance given to affected people prior to resettlement? (c) Is the resettlement plan, including compensation with full replacement costs, restoration of livelihoods and living standards developed based on socioeconomic studies on resettlement? (d) Are the compensations going to be paid prior to the resettlement? (e) Are the compensation policies prepared in document? (f) Does the resettlement plan pay particular attention to vulnerable groups or people, including women, children, the elderly, people below the poverty line, ethnic minorities, and indigenous peoples? (g) Are agreements with the affected people obtained prior to resettlement? (h) Is the organizational framework established to properly implement resettlement? Are the capacity and budget secured to implement the plan? (i) Are any plans developed to monitor the impacts of resettlement? (j) Is the grievance redress mechanism established?	(a) (b) (c) (d) (e) (f) (g) (h) (i) (j)	(a) (b) (c) (d) (e) (f) (g) (h) (i) (j)

Category	Environmental Item	Main Check Items	Yes: Y No: N	Confirmation of Environmental Considerations (Reasons, Mitigation Measures)
	(2) Living and Livelihood	(a) Is there any possibility that the project will adversely affect the living conditions of inhabitants? Are adequate measures considered to reduce the impacts, if necessary? (b) Is there any possibility that the project causes the change of land uses in the neighboring areas to affect adversely livelihood of local people? (c) Is there any possibility that the project facilities adversely affect the traffic systems? (d) Is there any possibility that diseases, including infectious diseases, such as HIV, will be brought due to the immigration of workers associated with the project? Are adequate considerations given to public health, if necessary? (e) Is the minimum flow required for maintaining downstream water uses secured? (f) Is there any possibility that reductions in water flow downstream or seawater intrusion will have impacts on downstream water and land uses? (g) Is there any possibility that water-borne or water-related diseases (e.g., schistosomiasis, malaria, filariasis) will be introduced? (h) Is there any possibility that fishery rights, water usage rights, and common usage rights, etc. would be restricted?	(a) (b) (c) (d) (e) (f) (g) (h)	(a) (b) (c) (d) (e) (f) (g) (h)
4 Social Environment	(3) Heritage	(a) Is there a possibility that the project will damage the local archeological, historical, cultural, and religious heritage? Are adequate measures considered to protect these sites in accordance with the country’s laws?	(a)	(a)
	(4) Landscape	(a) Is there a possibility that the project will adversely affect the local landscape? Are necessary measures taken?	(a)	(a)
	(5) Ethnic Minorities and Indigenous Peoples	(a) Are considerations given to reduce impacts on the culture and lifestyle of ethnic minorities and indigenous peoples? (b) Are all of the rights of ethnic minorities and indigenous peoples in relation to land and resources to be respected?	(a) (b)	(a) (b)

Category	Environmental Item	Main Check Items	Yes: Y No: N	Confirmation of Environmental Considerations (Reasons, Mitigation Measures)
	(6) Working Conditions	(a) Is the project proponent not violating any laws and ordinances associated with the working conditions of the country which the project proponent should observe in the project? (b) Are tangible safety considerations in place for individuals involved in the project, such as the installation of safety equipment which prevents industrial accidents, and management of hazardous materials? (c) Are intangible measures being planned and implemented for individuals involved in the project, such as the establishment of a safety and health program, and safety training (including traffic safety and public health) for workers etc.? (d) Are appropriate measures taken to ensure that security guards involved in the project not to violate safety of other individuals involved, or local residents?	(a) (b) (c) (d)	(a) (b) (c) (d)
5 Others	(1) Impacts during Construction	(a) Are adequate measures considered to reduce impacts during construction (e.g., noise, vibrations, turbid water, dust, exhaust gases, and wastes)? (b) If construction activities adversely affect the natural environment (ecosystem), are adequate measures considered to reduce the impacts? (c) If construction activities adversely affect the social environment, are adequate measures considered to reduce the impacts?	(a) (b) (c)	(a) (b) (c)
	(2) Accident Prevention Measures	(a) Is a warning system established to alert the inhabitants to water discharge from the dam?	(a)	(a)
5 Others	(3) Monitoring	(a) Does the proponent develop and implement monitoring program for the environmental items that are considered to have potential impacts? (b) What are the items, methods and frequencies of the monitoring program? (c) Does the proponent establish an adequate monitoring framework (organization, personnel, equipment, and adequate budget to sustain the monitoring framework)? (d) Are any regulatory requirements pertaining to the monitoring report system identified, such as the format and frequency of reports from the proponent to the regulatory authorities?	(a) (b) (c) (d)	(a) (b) (c) (d)

Category	Environmental Item	Main Check Items	Yes: Y No: N	Confirmation of Environmental Considerations (Reasons, Mitigation Measures)
6 Note	Reference to Checklist of Other Sectors	(a) Where necessary, pertinent items described in the Forestry Projects checklist should also be checked (e.g., projects in the mountains including large areas of deforestation). (b) In the case of dams and reservoirs, such as irrigation, water supply, and industrial water purposes, where necessary, pertinent items described in the Agriculture and Water Supply checklists should also be checked. (c) Where necessary, pertinent items described in the Power Transmission and Distribution Lines checklist should also be checked (e.g., projects including installation of electric transmission lines and/or electric distribution facilities).	(a) (b) (c)	(a) (b) (c)
	Note on Using Environmental Checklist	(a) If necessary, the impacts to transboundary or global issues should be confirmed (e.g., the project includes factors that may cause problems, such as transboundary waste treatment, acid rain, destruction of the ozone layer, or global warming).	(a)	(a)

1) Regarding the term “Country’s Standards” mentioned in the above table, in the event that environmental standards in the country where the project is located diverge significantly from international standards, appropriate environmental considerations are requested to be made.
In cases where local environmental regulations are yet to be established in some areas, considerations should be made based on comparisons with appropriate standards of other countries (including Japan's experience).
2) Environmental checklist provides general environmental items to be checked. It may be necessary to add or delete an item taking into account the characteristics of the project and the particular circumstances of the country and locality in which it is located.