CHAPTER 6

FORMULATION OF DEFINITIVE PLAN

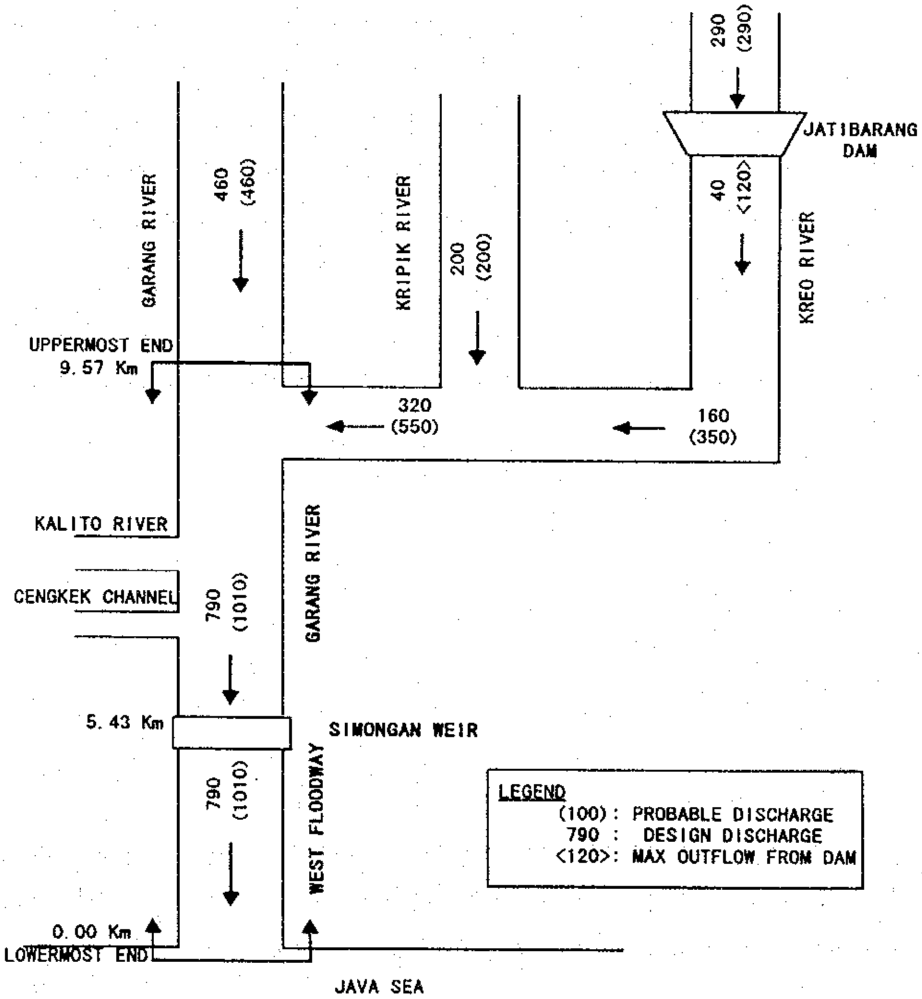
CHAPTER 6 FORMULATION OF DEFINITIVE PLAN

6.1 Function of Jatibarang Multipurpose Dam

Jatibarang Multipurpose Dam aims at providing multipurpose functions of flood control, water supply and hydropower generation. Each purpose is explained hereinafter.

6.1.1 Flood Control Purpose

West Floodway/Garang River Improvement is designed on the scale of a 100-year return period with flood control dam. Jatibarang Multipurpose Dam should have a function to reduce a 100-year probable flood with the peak discharge of 1,010 m^3/s to 790 m^3/s in the downstream from the confluence. Distribution of design flood discharge are as given below:



Distribution of Design Flood Discharge

The peak discharge of a 100-year probable flood is 290 m^3/s at the damsite. The design discharge at Simongan Weir of 790 m^3/s, which corresponds to a 25-year probable flood discharge, was applied to the river improvement of the river stretches from the river mouth up to the confluence with Kreo River.

6.1.2 Water Supply Purpose

The discharge secured by Jatibarang Multipurpose Dam consists of the intake water for municipal water supply of 2.04 m^3/s, which includes the present intake amount at PDAM, and the maintenance flow of 0.65 m^3/s.

The municipal water supply of 2.04 m^3/s includes 0.58 m^3/s for present use and 1.46 m^3/s for newly developed. The maintenance flow of 0.65 m^3/s includes 0.5 m^3/s for Semarang River and 0.15 m^3/s for Left Channel of Simongan Weir.

In the future, the deficit in the water supply from Jatibarang Reservoir would be supplemented by the proposed Mundingan Dam and Inter-basin water transfer project, which were proposed in the Master Plan. After the completion of the above two (2) facilities, the maximum outflow will increase to 6.0 m^3/s. The outlet facilities at Jatibarang Multipurpose Dam are designed to include the planed maximum out flow discharge of 6.0 m^3/s.

6.1.3 Hydropower Generation

The hydropower generation is carried out subordinately using the released water necessary for water supply to Semarang City.

6.2 Jatibarang Multipurpose Dam

6.2.1 Basic Conditions

Reservoir Water Surface

(1) Reservoir Storage Capacity

Storage capacity of the dam reservoir was estimated using the topographic map of 1:2,000, which was prepared in 1997 by the JICA Study Team. The cumulative volumes for each water surface are shown in Fig. 6.2.1.

(2) Reservoir Capacity Allocation

The gross storage capacity of 20,400,000 m^3 of Jatibarang Multipurpose Dam

Reservoir is allocated to (1) sediment capacity (6,800,000 m^3), (2) water use capacity (10,500,000 m^3) and (3) flood control capacity (3,100,000 m^3) as shown in Fig. 6.2.2.

(a) Sediment Capacity

The sediment capacity is allocated to 6,800,000 m^3, which was estimated as shown in Table 6.2.1.

The sediment discharge was estimated at $Qd = 1{,}062$ m^3/year/km^2 as the total of the wash load and the bed load, using the data such as topography, soil type, land use and hydrological conditions. Then, the specific sediment yield was estimated at $Qs = Qd*E/(1\text{-}P) = 2{,}550$ m^3/year/km^2, considering trap efficiency ($E = 0.96$) and porosity ($P = 0.60$). Finally, the sediment capacity was calculated at $Vs = Qs*A*Y = 6{,}800{,}000$ m^3, multiplying the specific sediment yield by the catchment area ($A = 53$ km^2) and the project life ($Y = 50$ years).

For the reference, the values of specific sediment yield adopted at existing nearby dams are shown in Fig. 6.2.3. Compared with them, that of Jatibarang Multipurpose Dam corresponds to a medium level.

The Low Water Surface Elevation 136.0 m has the storage volume equal to the sediment capacity.

(b) Water Use Capacity

The discharge of total 2.69 m^3/s required for the river maintenance and water supply to Semarang is supposed to be secured at the Simongan weir site. For this water use purpose, the storage capacity of 10,500,000 m^3 of Jatibarang Reservoir is needed.

The discharge of 2.69 m^3/s secured by Jatibarang Multipurpose Dam consists of the intake water for municipal water supply of 2.04 m^3/s and the maintenance flow of 0.65 m^3/s.

The municipal and industrial water supply of 2.04 m^3/s includes 0.58 m^3/s for present use and 1.46 m^3/s for newly developed. The maintenance flow of 0.65 m^3/s includes 0.5 m^3/s for Semarang River and 0.15 m^3/s for Left Channel of Simongan Weir.

The storage capacity for the exclusive use of hydropower generation is not allocated. The hydropower generation shall be carried out subordinately using the released water necessary for downstream water use.

The Normal Water Surface EL. 148.9 m has the storage volume of 17,300,000 m^3 equal to the sum of sediment capacity and water use capacity.

(c) Flood Control Capacity

The probable flood discharge of 100-year return period at Simongan Weir site is 1,010 m^3/s. Jatibarang Multipurpose Dam cuts 220 m^3/s of it out and makes the design discharge of the river channel of 790 m^3/s. For this flood control purpose, the storage capacity of 3,100,000 m^3 is allocated including 20 % allowance.

The Surcharge Water Surface Elevation 151.8 m has the storage volume of 20,400,000 m^3 equal to the sum of sediment capacity, water use capacity and flood control capacity.

Design Criteria

(1) Design Discharge

(a) Emergency Spillway

Emergency spillway should be designed to accommodate the Probable Maximum Flood (PMF) for fill type dam (by request letter from Directorate of Technical Guidance on 19 Feb. 1998). The minimum freeboard for a spillway without gate should not be less than 0.75 m, for a spillway with gate 1.25 m.

(b) Stilling Basin for Spillway

The design discharge of energy dissipater of stilling basin should be 100-year probable flood discharge at the damsite. The height of sidewall shall be designed so as not to overflow it when the maximum outflow discharge (Q_d = 1,310 m^3/s) after regulating the design discharge for emergency spillway (PMF) is flowed into the stilling basin.

(c) Diversion Tunnel

Diversion tunnel should be designed referring to the discharge of 25-year probable discharge for a fill type dam.

(d) Discharge Data

Peak discharges of the design flood are summarized in the following table. (refer to Table 6.2.2 and Fig. 6.2.4) The control point for calculation of probable rainfall is the damsite itself, which has the catchment area of 53 km^2.

Structure	Design Flood	Peak Discharge
Emergency Spillway	PMF	1,600 m^3/s
Stilling Basin	100-year flood	340 m^3/s
Diversion Tunnel	25-year	280 m^3/s

(2) Seismic Coefficient

The design seismic coefficient for the dam body stability analysis should be estimated referring to the treatise "Seismic Zone Map and Guidance for Design of Water Resources Structures against Earthquake (by Najoan and others)", which was brought up by Directorate of Technical Guidance.

Now, the seismic coefficient is calculated using the following equation.

$$K = z * a_c * v / g$$

Where,

K : Coefficient of Earthquake (seismic coefficient)

z : Coefficient of Zone (z = 0.8 for Semarang)
(refer to Table 6.2.3 and Fig. 6.2.5)

a_c : Basic Earthquake Acceleration (cm/s^2)
(a_c = 215.81 for 200-year return period, large dam)
(refer to Table 6.2.3)

v : Correction Factor of Ground (v=0.9 for rock, 1.0 for diluvium)
(refer to Table 6.2.3)

g : Gravity Acceleration (g=980 cm/s^2)

Therefore, the earthquake coefficient becomes as follows (refer to Table 6.2.3).

Seismic Coefficient	Type of Structure
K = 0.8 * 215.81 * 0.9 / 980 = **0.16**	Concrete Type Dam Large Concrete Structure
K = 0.8 * 215.81 * 1.0 / 980 = **0.18**	Fill Type Dam

(3) Clearance

Bridges will be designed above an emergency spillway at the dam crest. Clearance of bridge girders above the maximum flow surface should not be less than 0.75 m for a fill type dam, in accordance with Indonesian criteria (by the request letter from Directorate of Technical Guidance of DGWRD on 19 Feb. 1998).

(4) Sediment Deposit in Reservoir

A horizontal deposit surface is assumed for the reservoir capacity allocation of sediment and the stability analysis of the dam. The elevation of the deposit surface corresponds to the Low Water Surface Elevation 136.0 m.

(5) Safety Factor against Sliding

(a) Dam Embankment against Sliding Failure

Safety Factor of stability analysis must satisfy following conditions in accordance with the Indonesian criteria.

Reservoir Water Surface	Earthquake	Safety Factor
Normal Water Surface	100 %	1.20
Normal Water Surface	0 %	1.50
Maximum Water Surface	0 %	1.20
Rapid Drawdown to Low Water Surface	100 %	1.10
Rapid Drawdown to Low Water Surface	0 %	1.25
End of Construction	50 %	1.20

(b) Spillway Concrete Structure against Sliding Failure

Safety Factor by Henny must be larger than 4.0 under the loading conditions as shown below, in accordance with "Manual for River Works in Japan".

Reservoir Water Surface	Earthquake	Safety Factor
Normal Water Surface	100 %	4.0
Surcharge Water Surface	50 %	4.0
Maximum Water Surface	0 %	4.0

Dam Axis

A dam axis must be selected to determine the location of a dam. A topographical map of the damsite and vicinity are shown in Fig. 6.2.6. The dam axis is necessarily confined to a narrow range, because the hill at the left abutment of the dam forms a thin and long ridge shape projecting into the river course. Shown in Fig. 6.2.6 is the appropriate dam axis, which is extended from the center portion of the left thin ridge to the right bank hill in the direction crossing the river at right angle. The sectional configuration along this dam axis shows deep valley shape as shown in Fig. 6.2.7.

6.2.2 Alternative Dam Types to be Studied

(1) Preliminary Screening

Dams are largely classified into two (2) types, namely, concrete dam and fill type dam based on the materials comprising the dam body. Furthermore, the dam types are break down in detail corresponding to the structural characteristics as follows:

Dam Type		Appropriateness for Condition	
		Dam Height = not lower than 75 m	Foundation = Soft Rock
Concrete Dam	Gravity Dam	OK	OK or No
	Arch Dam	OK	No
	Hollow Gravity Dam	OK	No
	Buttress Dam	No	No
Fill Type Dam	Zoned rockfill Dam	OK	OK
	Facing rockfill Dam	OK	OK
	Homogeneous Fill Dam	No	OK

Notes : OK : Applicable, No : Not Applicable
OK or No : Depending on Strength of Foundation

The two (2) obvious conditions of Jatibarang Multipurpose Dam are pointed out as follows:

- The dam height is assumed to be not lower than 75 m.
- The foundation rocks at the Jatibarang damsite consist of soft rocks belonging to Tertiary to Quaternary.

Given these two (2) conditions, four dam types (arch dam, hollow gravity dam, buttress dam, and homogeneous fill dam) are judged as inappropriate dam types by preliminary screening. The reasons for inappropriate dam types are as follows:

(a) Concrete Arch Dam

A concrete arch dam is a structure that transmits the external loads from the reservoir onto the foundation rock on both sides and riverbed by an arch effect. This dam type fits for a narrow and deep valley, and has a possibility to save the concrete volume of the dam body. However, it requires hard foundation rock not only at the riverbed portion but also up to upper portion of the dam abutment on both banks, since the dam body is slender and the stress in the foundation rock becomes large.

The foundation rocks at the Jatibarang damsite consist of soft rocks belonging to Tertiary to Quaternary. This geological condition makes this type inappropriate.

(b) Hollow Gravity Dam

A hollow gravity dam aims at saving concrete volume by setting up the hollow space inside. However, the lowering of efficiency for the construction work becomes remarkable due to the complicated shape of the dam. In addition, there are some problems in the stability of the dam against earthquake in the direction of the dam axis and the countermeasures against floods during construction. Thus, this dam type is seldom adopted recently.

As for Jatibarang Multipurpose Dam, the concrete volume to be reduced by the hollow type is not so much because the damsite forms a deep valley. The possible concrete volume to be saved is larger in case of a wide valley than a narrow valley. Even in case of solid gravity dam, a thick shape of dam body is necessary for the Jatibarang damsite because the foundation rock consists of soft rock. In case of Hollow Gravity Dam, gentler slopes of upstream and downstream faces are required because of lightening the dam own-weight. It will result in unpractical typical section of the dam.

(c) Buttress Dam

A buttress dam sustains the external loads by a concrete plate and transmits the loads onto the foundation rock by the buttress walls. The stress in the thin structural members becomes large and the possible height of the dam is necessarily limited low. It requires about 60 percent less concrete than a concrete gravity dam, but the increased formwork and reinforcement steel required usually offset the savings in concrete. Therefore, although a number of buttress dams were constructed in the 1930's, this dam type is recently excluded from the classification of general dam types.

(d) Homogeneous Fill Dam

A homogeneous dam is constructed entirely or almost entirely of a single embankment material. It has been built since the earliest times and is used today whenever only one type of material is economically available. However, the possible height is limited within 30 m in general, because it is usually composed of impervious or semi-pervious soil with small shear strength. Therefore, this dam type is not applicable for Jatibarang Multipurpose Dam, whose height is more than 75 m.

(2) Applicability of Concrete Gravity Dam

A concrete gravity dam necessarily requires a large cross sectional area because it transmits the hydraulic pressure load of the reservoir on the lower foundation by way of its own weight. Although this type is adapted to site where there is reasonably sound rock foundation, it can be founded on soft rock with sufficient shear strength in accordance with the dam height.

In case of Jatibarang Multipurpose Dam with soft rock foundation, the geological survey reveals that the foundation rocks does not have enough strength for its dam height. The design shear strength of lower pyroclastic rock unit (CM-H class) distributed at the riverbed is expected to be only 50 tf/m^2.

As a result of the shear safety evaluation that is made using Henny's formula, it is found that the dam base have to be widen by means of providing fillet at the upstream face and giving considerably gentle downstream slope. Finally, with the downstream slope of 1:1.2, the upstream slope reached 1:1.4 to secure the total safety factor of greater than 4.

Generally, the upstream and downstream surfaces of a concrete gravity dam are steeper than 1:1.2 and 1:1.0, respectively, without any countermeasure because of good stress distributions in dam body. In addition, the calculated slopes correspond to double to triple compared with the one of a concrete dam constructed on reasonably sound rock foundation. This means that a concrete gravity type dam would be much costly due to its very large volume. A concrete gravity dam is almost twice the cost of a fill type dam at a rough estimate.

In conclusion, from the technical and economical points of view, it is decided that a concrete gravity dam is not applicable.

(3) Alternative Dam Types to be Studied

A fill type dam has a smaller restraint from the strength of foundation rock because it transmits the external loads onto the broader area of the foundation than a concrete gravity dam. The design shear strength of lower pyroclastic rock unit (CM-H class) distributed at the riverbed is expected to be only 50 tf/m^2. Therefore, the zoned rockfill dam and facing rockfill dam have an advantage from the viewpoint of the geological condition.

After exclusion of inappropriate dam types, the following two (2) dam types remain as alternative dam types to be studied.

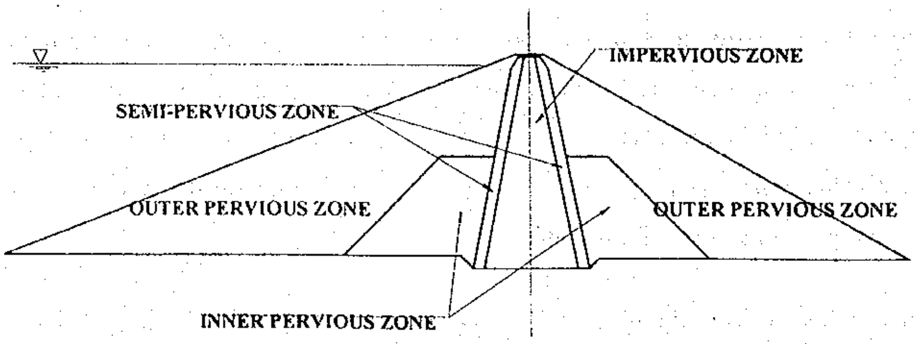
- Zoned Rockfill Dam
- Facing Rockfill Dam

6.2.3 Preliminary Design on Zoned Rockfill Dam

The design features of alternatives discussed hereunder are for the purpose of dam type selection and project evaluation. These preliminary designs are carried out in enough detail to show the basic layouts and sections. However, for any dam types selected, design aspects would be verified analyzed while preparing final design details.

Constitution of Zones

Generally this dam type can be divided into three or more zones as shown below, depending on the range of variation in the character and gradation of the available material. The permeability of each zone increases toward the outer slopes. Impervious zone filled with earth material provides watertightness. Inner and outer pervious zones filled with rock of all sizes support the less stable impervious material and provide the stability of the dam body. Semi-pervious zone of sand-gravel or fine rock is embanked between impervious zone and pervious zone, to be served as a transition and filter zone.



Typical Constitution of Zones in Zoned Rockfill Dam

In this type of dam, the impervious zone is placed in a vertical position near the center of the embankment or sloped upstream. Generally, they are called the center core type and sloping core type respectively.

It is concluded that center core type will be studied due to the following reasons:

① Center core can provide higher pressures which will take place on the contact between the core and the foundation rock, and bring more protection against the possibility of leakage along the contact,

② The embankment volume can be made smaller in center core rockfill dam due to its steeper upstream slope, and

③ It is sometimes speculated that sloping core rockfill dam may be susceptible to cracking owing to differential settlement.

Embankment Material

Generally, it is economical to utilize the available materials near the damsite in their natural states without processing. Based on the principle the investigation was commenced from the areas near the damsite to look for the natural materials of the embankment zones of definite properties.

The investigation results of materials are described in clause 5.3 Construction Material. The conclusion is given hereinafter.

(1) Impervious Material

Earth material for impervious zone must have the required coefficient of permeability with a smaller compressibility after compaction, and must be easy to be compacted, and also must not contain deleterious organic substances.

Borrow Areas A, B and D were investigated to look for the natural materials of impervious zone. (refer to Fig. 5.3.6) Resulting from the test results summarized in Table 5.3.4, the natural materials available in the said Borrow Areas were composed of silt and clay with more than 90 % of fine particles passing a 0.075 mm sieve. In accordance with the Unified Soil Classification System, they can be classified into the CH or MH.

The properties of embankments using the silts and clays of CH or MH group can be predicted as follows:

① They are impervious when compacted with moisture-density control. However, they show a larger compressibility and poor workability due to the deficiency of coarse-grained materials.

② In case of zoned rockfill dam with center core comprised of such silt and clay, it is susceptible to horizontal cracks in the core because rock zone shows smaller compressibility.

③ The construction surface of embankment comprised of such silt and clay is too soft to support the travel of the heavy hauling equipment or any kind of heavy rollers. With average moisture content of several percent above its optimum, heavy hauling equipment may penetrate the construction surface and bog down.

Judging from these predictions of the embankment properties, natural materials investigated so far can not be used without processing.

Accordingly, to improve the larger compressibility and poor workability, it is concluded that the natural material available in the borrow areas should be mixed with coarse-grained materials and used as the impervious material.

Crushed rock and sand manufactured in plant equipment at the quarry will be used as coarse-grained materials. Purchased sand, which are produced around Mt. Merapi and available in Semarang, may substitute for sand manufactured by crushing.

Riverbed and terrace deposits at Borrow Area C are not considered to be used as the coarse-grained materials for the impervious material. The reasons are as follows.

① Although the quality of riverbed deposit is suitable, the available amount of it is less than 10 % of the required quantity because of the very limited distribution.

② It is much costly to use terrace deposit because it is covered by large amount of a silt layer.

(2) Semi-pervious Material

Material for semi-pervious zone shall consist of a well-graded and non-plastic mixture of sand-gravel or fine rock in which the individual particles are hard and durable, free from clay, silt and organic material.

Borrow Area C was investigated to look for the natural materials of semi-pervious zone. (refer to Fig. 5.3.6) Resulting from the test results summarized in Table 5.3.6, riverbed and terrace deposits available in Borrow Area C are composed of sand and gravel. However, they are distributed in the very limited area, and contaminated by 6 to 20 % of silt and clay, and about 10 % of siltstone gravel, which is easily pulverized by slaking.

Judging from the above properties, riverbed and terrace deposits can not be used as the material of the semi-pervious zone. Such contaminated sand and gravel do not have permeability sufficient that seepage water and residual moisture in impervious zone is readily drained from it.

Accordingly, it is decided that mixed material of the crushed rock and sand manufactured by plant equipment at the quarry at Mt. Mergi will be used for the material of the semi-pervious zone. Purchased sand, which are produced around Mt. Merapi and available in Semarang, may substitute for sand manufactured by crushing.

(3) Pervious Material

Rock material for pervious zone, especially filled in outer rock zone, shall consist of a well-graded mixture of hard and durable particles. And it shall be slightly weathered to fresh rock to secure the stability of dam body.

The source of rock has been identified. The nearest source of high-grade rock is andesite that has intruded into the Tertiary formations existing at Mt. Mergi near Mt. Ungaran, where was proposed as the quarry of concrete aggregate (refer to Figs. 5.3.1 and 5.3.2). This andesite rock formed an isolated hill with a sharp peak and located 17 km southeast from the damsite.

Andesite is classified into CL, CM and CH class that are applicable for the pervious zone. The investigation results show that sufficient amount of rock is available. CH and CM class rock can be filled in any pervious zone and the use of CL class rock is limited to be mixed with CH and CM class rock.

Dam Crest Level

The dam crest level of non-overflow portion should not be lower than the highest value determined from among the Normal Water Surface, the Surcharge Water Surface and the Maximum Water Surface, plus freeboard. The wave height due to wind and seismic motion or the minimum freeboard (0.75 m) by Indonesian Criteria, whichever larger, shall be adopted as freeboard.

The wave height (he) due to seismic motion is calculated using Sato's Formula as shown in Fig. 6.2.8.

The wave height (hw) due to wind is calculated using the S.M.B. Method. When the upstream face is inclined as a fill type dam, the run-up height of the wave along the dam is calculated using Saville's Method. Fig. 6.2.8 shows diagram for the run-up wave height obtained by a combined use of the S.M.B. Method and Saville's Method.

The required dam crest level under each water surface for the zoned rockfill dam is calculated as follows:

Reservoir Water Surface		he	hw	Freeboard	Required Dam Crest
Normal Water Surface	EL. 148.9 m	0.8 m	0.5 m	he + hw = 1.3m>0.75m	EL. 148.9 m + 1.3 m = EL. 150.2 m
Surcharge Water Surface	EL. 151.8 m			he/2 + hw = 0.9m>0.75m	EL. 151.8 m + 0.9 m = EL. 152.7 m
Maximum Water Surface	EL. 155.3 m			hw = 0.5m<0.75m	EL. 155.3 m + 0.75 m = EL. 156.1 m

Note : Maximum Water Surface is studied in the following section.

Actual dam crest level is determined adding the thickness of a protection layer which prevents the impervious zone from being eroded. The dam crest level is concluded to become EL. 157.0 m considering the maximum required dam crest of EL. 156.1 m plus 0.9 m thickness of the protection layer covering erodible impervious zone.

Preliminarily Design of Center Core Rockfill Dam

Preliminary design of center core rockfill dam (zoned rockfill dam) is executed considering the requirement to suit the conditions of the site and to utilize available construction materials. The plan and typical cross section are shown in Figs. 6.2.9 and 6.2.10, and main features are given below:

Crest Level	EL. 157.0 m
Foundation Level	EL. 80.0 m
Dam Height	77.0 m
Crest Length	200.0 m
Upstream Slope	1 : 2.6
Downstream Slope	1 : 1.8

The basic concept of design is explained hereunder.

① The narrow crest width leads to poor construction control and local failure of the crest. Considering proper construction of impervious, semi-pervious and pervious zones using heavy equipment, 10.0 m is desirable for the crest width of center core rockfill dam.

② The upstream slope and downstream slope are designed at 1.0 vertical to 2.6 horizontal and 1.0 vertical to 1.8 horizontal, respectively. Stability of these slopes will be confirmed by the stability analysis.

③ The foundation rock under impervious and semi-pervious zones are required to be CM-L to CM-H class rocks except for the both abutments with the low dam height.

④ The foundation rock under pervious zone is required to be non-erodible. Excavation of 1 to 2 m in depth is required to obtain an acceptable foundation rock.

⑤ The excavation of bank slopes toward the both abutments is restricted so as not to be steeper than 1.0 horizontal to 1.0 vertical. Consequently, the possibility of embankment cracking due to differential settlement would be minimized and the extremely low pressure on the contact between the impervious material and the foundation rock should be avoided.

⑥ Since the foundation rock consists of soft rock, the grouting work shall be worked out very carefully. Internal gallery large enough for people to enter and work is provided under impervious zone. By having access to the foundation under the dam when the reservoir is filled, any additional grouting can be conducted. In addition, the use of the grouting gallery can appreciably shorten the time of construction and make the grouting timetable independent of the embankment schedule.

⑦ Width of impervious zone is designed to 4.0 m at the top and inclined shape with 1.0 vertical to 0.2 horizontal on both upstream and downstream sides. The width corresponds to about 45 % of the water head. It was considered that impervious zone with a width of 30 % to 50 % of the water head have proved satisfactory at existing many dams under diverse conditions.

⑧ Semi-pervious zones are provided at both sides of impervious zone. The upstream and downstream semi-pervious zones are designed to prevent the impervious zone from washing out the fine particles and to provide adequate drainage of impervious zone. Considering ease in construction, the horizontal width is fixed at 4.0 m with the same slope of impervious zone.

⑨ Pervious zone is subdivided into two zones, inner pervious zone and outer pervious zone. Both sides of inner zone are arranged at the outer side of semi-pervious zone of both upstream and downstream. Rock materials from the required excavations at the damsite can be filled in the inner zones.

⑩ 1.0 m thick of riprap zone with selected large size rocks is provided to prevent the upstream slope from being eroded. It is placed above the Low Water Surface EL. 136.0 m.

Stability Analysis of Center Core Rockfill Dam

(1) Design Values

The design values of materials to be used in stability analysis are adopted from the limited test results on finer samples (refer to Tables 6.3.7 to 6.3.9). Cohesion and internal friction angle in terms of effective stresses are directly determined from the test results. Wet density and saturated density obtained from the laboratory tests can be converted into the design values considering a content ratio of a gravel coarser than the maximum size (19.0 mm) of samples in the laboratory. (refer to Table 6.2.4)

The rock materials for the pervious zone have high friction angle at low stress levels as shown in Fig. 6.2.11. The upper envelope of the circles on a Mohr diagram is typically concave downward with a slope that is steepest in the lower range of normal stress that decreases gradually with increasing stress.

For the rock materials for outer pervious zone, three (3) fixed values of friction angle depending on stress levels are adopted as shown in Fig. 6.2.11. Internal friction angle of them in inner pervious zone is reduced by about 5 % because the soft rocks from the required excavations, which have less desirable properties and are more erratic, are allowed to be filled in this zone.

For the coarse pervious and semi-pervious materials, the cohesion is assumed to be zero.

The estimated design values are summarized below:

Zone	Wet Density γt (tf/m^3)	Saturated Density γsat (tf/m^3)	Effective Cohesion C' (tf/m^2)	Effective Internal Friction Angle ϕ' (°)
Impervious Zone	2.19	2.11	1	25
Semi-Pervious Zone	2.11	2.27	0	35
Inner Pervious Zone	1.94	2.16	0	43 ($0.0 < \sigma' \leqq 2.6$ kgf/cm^2) 40 ($2.6 < \sigma' \leqq 6.3$ kgf/cm^2) 35 (6.3 $kgf/cm^2 < \sigma'$)
Outer Pervious Zone	1.94	2.16	0	45 ($0.0 < \sigma' \leqq 2.6$ kgf/cm^2) 42 ($2.6 < \sigma' \leqq 6.3$ kgf/cm^2) 37 (6.3 $kgf/cm^2 < \sigma'$)

Notes : σ' = Effective normal stress acting on the failure surface
Refer to Table 6.2.4 and Fig. 6.2.11

(2) Stability Analysis by Surface Sliding Method

The factor of safety against plane surface sliding of the outer pervious zone can be obtained under the assumption that the slope of cohesionless material extends uniformly and semi-infinitely.

The critical safety factor against plane surface sliding is expressed in the following equation. This equation can be applied for the case of the reservoir being full and the upstream slope being fully saturated. In case of the downstream slope, γ sat and γ sub are equal.

$$SF = \frac{\{1 - k * (\gamma \text{ sat} / \gamma \text{ sub}) * \tan\theta\}}{\{\tan\theta + k * (\gamma \text{ sat} / \gamma \text{ sub}) * \tan\phi\}}$$

Where,

SF : critical safety factor

θ : slope gradient (°)

k : horizontal seismic coefficient (0.18)

ϕ : effective internal friction angle (45°)
($0.0 < \sigma' \leqq 2.6$ kgf/cm^2)

γ sat : saturated density (2.16 tf/m^3)

γ sub : submerged density ($2.16 - 1.0 = 1.16$ tf/m^3)

Calculation results are summarized in the following table.

Slopes	tan θ	Reservoir Water Surface	k	Safety Factor	
				Calculated	Required
Up-stream	0.400 (1:2.5)	Normal Water Surface	0.18	1.178	1.20---×
		Normal Water Surface	0	2.500	1.50---OK
		Maximum Water Surface	0	2.500	1.20---OK
		End of Construction	0.09	1.644	1.20---OK
	0.385 (1:2.6)	Normal Water Surface	0.18	1.210	1.20---OK
		Normal Water Surface	0	2.600	1.50---OK
		Maximum Water Surface	0	2.600	1.20---OK
		End of Construction	0.09	1.694	1.20---OK
Down-stream	0.588 (1:1.7)	Normal Water Surface	0.18	1.164	1.20---×
		Normal Water Surface	0	1.700	1.50---OK
		Maximum Water Surface	0	1.700	1.20---OK
		End of Construction	0.09	1.396	1.20---OK
	0.556 (1:1.8)	Normal Water Surface	0.18	1.224	1.20---OK
		Normal Water Surface	0	1.800	1.50---OK
		Maximum Water Surface	0	1.800	1.20---OK
		End of Construction	0.09	1.472	1.20---OK

For 1:2.5 of the upstream slope and 1:1.7 of downstream slope, the safety factors are less than the minimum required value. The designed slopes of the upstream and downstream satisfy the minimum requirement of safety factor.

(3) Stability analysis by Slip Circle Method

The dam embankment shall be safe against sliding failure under the loading conditions. The stability analysis is carried out using the effective stress method. The safety factor against sliding for an assumed circle is examined by the following equation:

$$SF = \frac{\Sigma\{C'*L + (N - U - Ne)*\tan\phi'\}}{\Sigma(T + Te)}$$

Where,

SF : safety factor

N : normal force acting on slip circle (tf/m)

T : tangential force acting on slip circle (tf/m)

U : pore pressure acting on slip circle (tf/m)

Ne : normal force of earthquake load acting on slip circle (tf/m)

Te : tangential force of earthquake load acting on slip circle (tf/m)

φ' : effective internal friction angle on slip circle (°)

C' : effective cohesion on slip circle (tf/m^2)

L : arc length of slip circle (m)

Slope stability analysis of the highest section is performed in two cases, at the Normal Water Surface with 100 % of seismic coefficient for the upstream and downstream slopes, which are the most critical cases evaluated by the plane surface sliding method. The results are given in Figs. 6.2.12 and 6.2.13, and the most critical results are shown below:

Slope	Reservoir Water Surface	Radius of Sliding Circle (m)	Safety Factor	
			Calculated	Required
Upstream	Normal Water Surface	285.004	1.22	1.20---OK
Downstream	Normal Water Surface	106.500	1.23	1.20---OK

It is concluded that designed slopes of the dam embankment satisfy the required safety factor.

Work Volume of Center Core Rockfill Dam

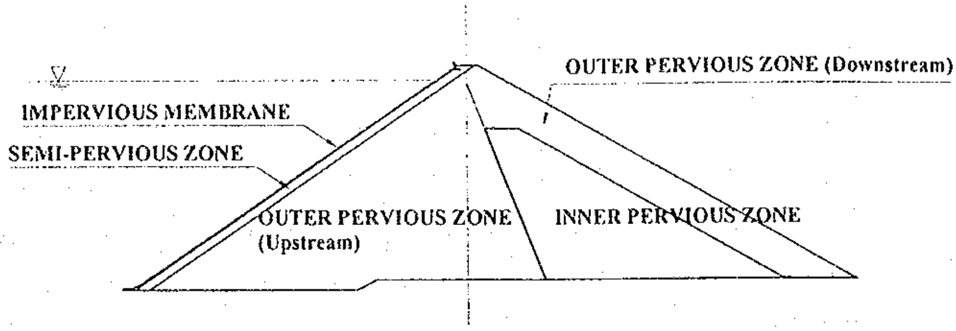
Work volume of the center core rockfill dam is given in the following table.

Item	Zone	Volume (m^3)
Embankment	Impervious Zone	124,000
	Semi-pervious Zone	65,000
	Inner Pervious Zone	105,000
	Outer Pervious Zone	496,000
	Total	790,000
Concrete (for Internal Gallery)		5,000
Excavation		198,000

6.2.4 Preliminary Design on Facing Rockfill Dam

Constitution of Zones

Facing rockfill dam consists of three or more zones as shown below, namely, pervious zone as a major structural element, impervious membrane placed on the upstream slope and semi-pervious zone. Pervious zone mainly constructed with rock materials transfers water loads to foundation rock and provides a larger section of the dam for stability against sliding. Semi-pervious zone of sand-gravel or fine rock is used to provide firm and uniform support for the impervious membrane and to retard extreme water losses from the membrane crack.



Typical Constitution of Zones in Facing Rockfill Dam

Reinforced concrete and asphalt concrete can be used as impervious membranes of manufactured materials. In this report, they are called the concrete face rockfill dam and asphalt face rockfill dam respectively.

It is concluded that concrete face rockfill dam will be studied due to the following reasons:

① Under the strong sunshine in the tropical regions, the temperature of the outer surface of asphalt concrete is extremely high. Asphalt concrete could not withstand such temperature without dangerous softening and downward creep on the slope.

② Deterioration of asphalt concrete is characterized by a loss of plasticity. Intense ultraviolet rays quicken this deterioration and the effective life of asphalt concrete would be much shorter than reinforced concrete.

③ Asphalt concrete face is relatively soft and can be more easily damaged by falling rock or other activities of man and nature than reinforced concrete face.

Embankment Material

(1) Semi-pervious Material

Material for semi-pervious zone shall consist of a well-graded and smaller-sized mixture of sand-gravel or fine rock in which the individual particles are hard and durable, free from organic material.

As mentioned in the study of embankment material for zoned rockfill dam, it is decided that mixed material of the crushed rock and sand manufactured by plant equipment at the quarry at Mt. Mergi will be used in semi-pervious zone. Purchased sand, which are produced around Mt. Merapi and available in Semarang, may substitute for sand manufactured by crushing.

(2) Pervious Material

Rock material for pervious zone shall consist of a well-graded mixture of hard and durable particles. And it shall be slightly weathered to fresh rock to secure the stability of dam body.

As mentioned in the study of embankment material for the zoned rockfill dam, the nearest source of high-grade rock is andesite that has intruded into the Tertiary formations. It is existing at Mt. Mergi, where was proposed as the quarry of concrete aggregate (refer to Figs. 5.3.1 and 5.3.2). This andesite rock formed an isolated hill with a sharp peak and 17 km from the damsite.

Andesite is classified into CL, CM and CH class that are applicable for the pervious zone. The investigation results show that sufficient amount of rock is available. CH and CM class rock can be filled in any pervious zone and the use of CL class rock is limited to be mixed with CH and CM class rock.

Dam Crest Level

The dam crest level of non-overflow portion should not be lower than the greatest value determined from among the Normal Water Surface, the Surcharge Water Surface and the Maximum Water Surface, plus freeboard. The wave height due to wind and seismic motion or the minimum freeboard (0.75 m) by Indonesian Criteria, whichever larger, shall be adopted as freeboard.

The wave height (he) due to seismic motion is calculated using Sato's Formula as shown in Fig. 6.2.8

The wave height (hw) due to wind is calculated using the S.M.B. Method. When the upstream face is inclined as in case of fill type dam, the run-up height of the wave along the dam is calculated using Saville's Method. Fig. 6.2.8 shows diagram for the run-up wave height obtained by a combined use of the S.M.B. Method and Saville's Method.

The required dam crest level under each water surface is calculated as follows:

Reservoir Water Surface		he	hw	Freeboard	Required Dam Crest
Normal Water Surface	EL. 148.9 m	0.8 m	1.8 m	he + hw = 2.6m>0.75m	EL. 148.9 m + 2.6 m = EL. 151.5 m
Surcharge Water Surface	EL. 151.8 m			he/2 + hw = 2.2m>0.75m	EL. 151.8 m + 2.2 m = EL. 153.5 m
Maximum Water Surface	EL. 155.3 m			hw = 1.8m>0.75m	EL. 155.3 m + 1.8 m = EL. 157.1 m

Note : Maximum Water Surface is studied in the following section.

The run-up wave works out to 1.8 m because of the smooth slope. It is more than triple compared with the one of the riprap slope.

To overcome this disadvantage, the concrete parapet wall is provided to throw the wave back toward the reservoir. The top of the parapet wall is set at EL. 157.5 m and 2.2 m of freeboard is provided above the Maximum Water Surface EL. 155.3 m.

Dam crest level is set at EL. 156.5 m considering 0.75 m of the minimum freeboard above the Maximum Water Surface Elevation 155.3 m.

Preliminarily Design of Concrete Face Rockfill Dam

Preliminary design of concrete face rockfill dam (facing rockfill dam) is executed considering the requirement to suit the conditions of the site and to use available construction materials. The plan and typical cross section are shown in Figs. 6.2.14 and 6.2.15, and main features are given below:

Crest Level	EL. 156.5 m
Foundation Level	EL. 80.0 m
Top of Parapet Wall	EL. 157.5 m
Dam Height	76.5 m
Crest Length	200.0 m
Upstream Slope	1 : 1.5
Downstream Slope	1 : 1.8

The basic concept of design is explained hereunder.

① The narrow crest width leads to poor construction control and local failure of the crest. Considering proper construction of semi-pervious and pervious zone using heavy equipment, 8.0 m is sufficient for the crest width of a concrete face rockfill dam. The crest width of concrete face rockfill dam can be narrower than center core rockfill dam because concrete face rockfill dam does not have the protected impervious zone filled with earth material.

② The upstream slope and downstream slope are designed at 1.0 vertical to 1.5

horizontal and 1.0 vertical to 1.8 horizontal respectively. Stability of these slopes will be confirmed by the stability analysis.

③ A concrete parapet wall with conventional cantilever type is provided to protect wave by wind at the dam crest as shown in Fig. 6.2.16. It is 2.62 m in height and projecting 1.0 m above the crest road. The top of wall is set at EL. 157.5 m.

④ The concrete face slab is the impervious element of the dam. It is designed to have a thickness of 35 cm at the top with gradually increasing thickness at lower levels in accordance with the following formula:

$$T = 0.35 + 0.003*H$$

Where,

T : thickness of face slab (m)
H : vertical depth below crest level (m)

⑤ The toe structure is provided as shown in Fig. 6.2.16. The edge of the toe structure that meets the face slab is kept perpendicular to the line of the face slab. The toe structure is joined to the face slab by a flexible but leakproof joint. The topside of joint is sealed with rubber mastic joint filler covered with sheet.

⑥ Since the foundation rock consists of soft rock, the grouting work shall be worked out very carefully. Internal gallery large enough for people to enter and work is provided under the toe structure. By having access to the foundation under the dam when the reservoir is filled, any additional grouting can be conducted.

⑦ The internal gallery is to be constructed in CM-L to CM-H class rocks except for the both abutments with the low dam height.

⑧ The foundation rock at the upstream side with the length almost equivalent to the dam height from the gallery shall be excavated down to the CM-L to CM-H class rock to minimize the foundation settlement by water pressure. The overburden and highly weathered rock of downstream foundation under pervious zone is required to be removed because the foundation rock does not receive water pressure directly. Excavation of 1 to 2 m in depth is required to obtain an acceptable foundation rock.

⑨ Semi-pervious zone is directly under the concrete face slab and provides firm and uniform bedding. It also forms semi-impervious barrier preventing any significant leakage. Its horizontal width is fixed at 6.0 m.

⑩ Transition zone embanked with fine rocks is provided between semi-pervious zone and pervious zone. Its horizontal width is fixed at 6.0 m.

⑪ Pervious zone is subdivided into two zones, inner pervious zone and outer pervious zone. Upstream outer pervious zone has low compressibility, as it would transfer water load to the foundation rock. Inner zone is arranged at the downstream side of upstream outer pervious zone. The rock materials from the required excavations at the damsite can be filled in inner pervious zone.

⑫ Blanket zone filled with impervious material is provided to cover the face slab in the lower elevations (up to EL. 115.0 m). Cracks developed in the face slab could be clogged with fine-grained soil.

⑬ Random fill zone filled with the waste materials is provided to cover blanket zone.

Stability Analysis for Concrete Face Rockfill Dam

(1) Design Values

The design values of materials used in stability analysis are adopted from the limited test results on smaller samples (refer to Tables 5.3.7 to 5.3.9). The explanations of the design values are made in the study of zoned rockfill dam (refer to Table 6.2.4). The estimated design values are summarized below:

Zone	Wet Density γt (tf/m^3)	Saturated Density γsat (tf/m^3)	Effective Cohesion C' (tf/m^2)	Effective Internal Friction Angle ϕ' (°)
Semi-Pervious Zone	2.01	2.21	0	45 ($0.0<\sigma'<2.6$ kgf/cm^2) 42 ($2.6<\sigma'<6.3$ kgf/cm^2) 37 (6.3 $kgf/cm^2<\sigma'$)
Transition Zone	1.94	2.16	0	45 ($0.0<\sigma'<2.6$ kgf/cm^2) 42 ($2.6<\sigma'<6.3$ kgf/cm^2) 37 (6.3 $kgf/cm^2<\sigma'$)
Inner Pervious Zone	1.94	2.16	0	43 ($0.0<\sigma'<2.6$ kgf/cm^2) 40 ($2.6<\sigma'<6.3$ kgf/cm^2) 35 (6.3 $kgf/cm^2<\sigma'$)
Outer Pervious Zone	1.94	2.16	0	45 ($0.0<\sigma'<2.6$ kgf/cm^2) 42 ($2.6<\sigma'<6.3$ kgf/cm^2) 37 (6.3 $kgf/cm^2<\sigma'$)

Note : σ' = Effective normal stress acting on the failure surface
Refer to Table 6.2.4 and Fig. 6.2.11

(2) Stability Analysis by Surface Sliding Method

The factor of safety against plane surface sliding can be obtained under the assumption that the slope of cohesionless material extends uniformly and semi-infinitely. Because the water pressure is resisted on the upstream slope, the upstream slope is safe during the Normal Water Surface and the Maximum Water Surface. Calculation results are summarized in the following table.

Slopes	tan θ	Reservoir Water Surface	k	Safety Factor	
				Calculated	Required
Up-stream	0.714 (1:1.4)	End of Construction	0.09	1.163	1.20---×
	0.667 (1:1.5)	End of Construction	0.09	1.242	1.20---OK
Down-stream	0.588 (1:1.7)	Normal Water Surface	0.18	1.164	1.20---×
		Normal Water Surface	0	1.700	1.50---OK
		Maximum Water Surface	0	1.700	1.20---OK
		End of Construction	0.09	1.396	1.20---OK
	0.556 (1:1.8)	Normal Water Surface	0.18	1.224	1.20---OK
		Normal Water Surface	0	1.800	1.50---OK
		Maximum Water Surface	0	1.800	1.20---OK
		End of Construction	0.09	1.472	1.20---OK

For 1:1.4 of the upstream slope and 1:1.7 of downstream slope, the safety factors are less than the minimum required value. The designed slopes of the upstream and downstream satisfy the minimum requirement of safety factor.

(3) Stability Analysis by Slip Circle Method

The dam embankment shall be safe against sliding failure under the loading conditions. The stability analysis is carried out using the effected stress method.

Slope stability analysis of the maximum section is performed in two cases, at the end of construction with 50 % of seismic coefficient for the upstream and at the Normal Water Surface for the downstream slope, which are the most critical cases evaluated by the plane surface sliding method. The results are given in Figs. 6.2.17 and 6.2.18, and the most critical results are shown below:

Slope	Reservoir Water Surface	Radius of Sliding Circle (m)	Safety Factor	
			Calculated	Required
Upstream	End of Construction	121.487	1.27	1.20---OK
Downstream	Normal Water Surface	166.500	1.22	1.20---OK

It is concluded that designed slopes of dam embankment satisfy the required safety factor.

Work Volume of Concrete Face Rockfill Dam

Work volume of the concrete face rockfill dam (facing rockfill dam) is given in the following table.

Item	Zone	Volume (m^3)
Embankment	Semi-pervious Zone	37,000
	Transition Zone	41,000
	Inner Pervious Zone	120,000
	Outer Pervious Zone	367,000
	Blanket Zone	8,000
	Random Zone	26,000
	Total	599,000
Concrete	Face Slab, Parapet Wall, Toe Structure	7,600
	Internal Gallery	6,500
	Total	14,100
Excavation		194,000

6.2.5 Selection of Dam Type

As shown in the foregoing discussion, two dam types are technically feasible at the selected damsite. Center core rockfill and concrete face rockfill dams can be successfully constructed. The selection of the dam type shall be done based on ease of construction, cost and technical considerations reflecting the foundation condition and ease of maintenance.

(1) Comparative Study on Construction Works

Two (2) types of rockfill dams, (a) center core rockfill dam and (b) concrete face rockfill dam are compared from the viewpoint of construction manners, quality control, safety and construction period based on the preliminary design of both types.

Comparative study focusing on dam construction manners (appurtenant structures such as spillway are not considered in the comparative study) is carried out. The construction plans for two alternatives are studied to clarify their advantages and disadvantages.

(a) Construction Schedule of Center Core Rockfill Dam

The workable days for the impervious zone embankment, that is supposed to be a critical activity, are 172 days a year because the impervious earth material depends on weather in order to control its moisture content. Construction works are executed under the single shift basis for excavation works and two shifts basis for embankment and grouting works.

Center core rockfill dam construction is carried out in accordance with the following flow diagram:

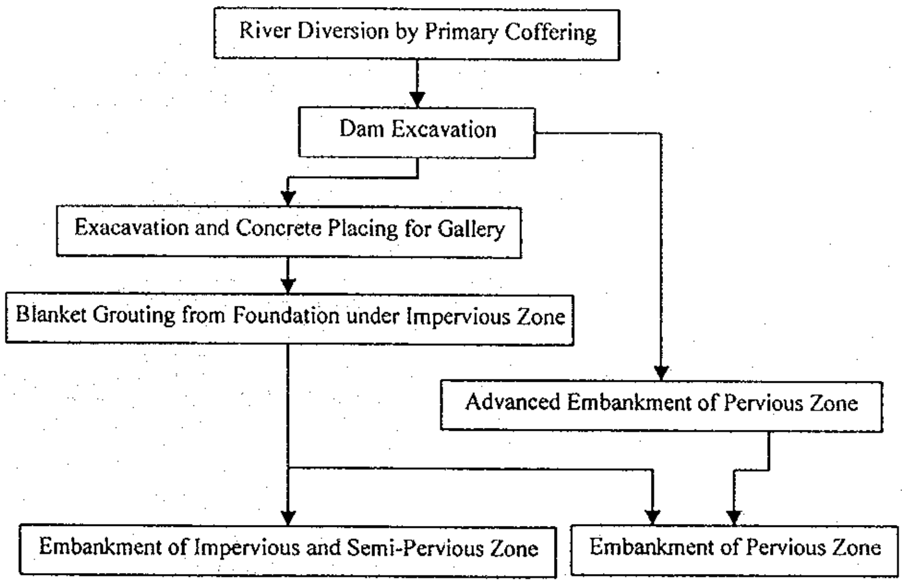


Table 6.2.5 shows the construction schedule for center core rockfill dam. Key dates of events, and the construction period and the average progress required for main activities are also given in the table.

The required duration from the commencement of work to complete the dam construction and to start impounding is 43 months.

(b) Construction Schedule of Concrete Face Rockfill Dam

Concrete face rockfill dam does not have impervious earth zone which is susceptible to weather condition. The workable days for earth and concrete works are 225 days and 252 days a year respectively. Construction works are executed under the single shift basis for excavation work and two shifts' basis for embankment, grouting and concrete facing works.

Concrete face rockfill dam is constructed in accordance with the following flow diagram:

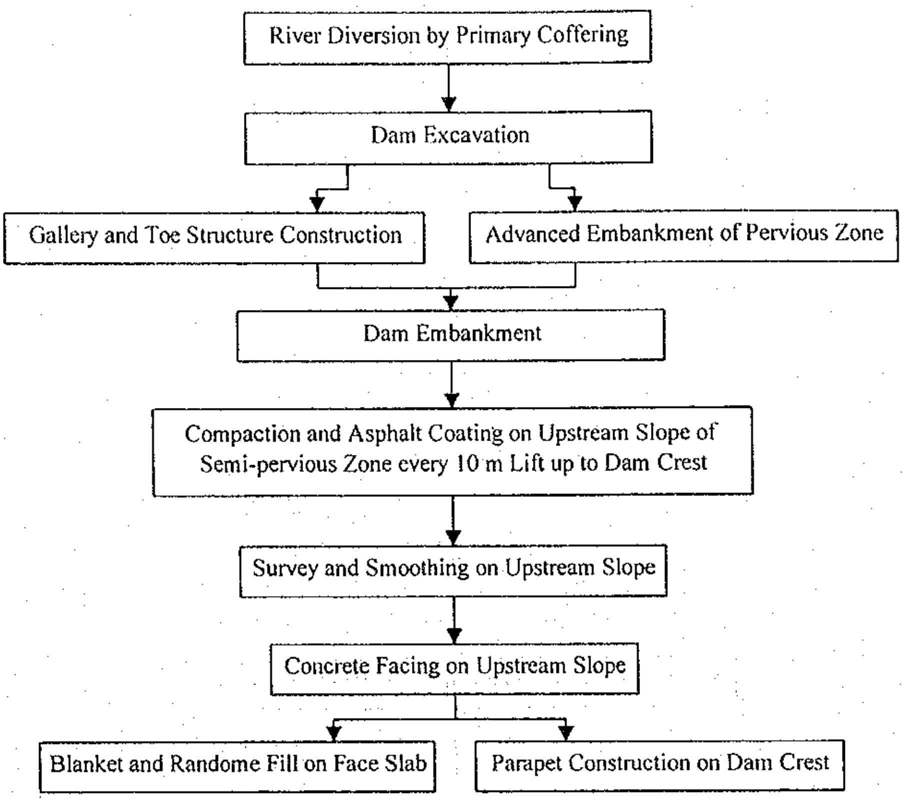


Table 6.2.6 shows the dam construction schedule. Key dates of events, and the construction period and the average progress required for main activities are also given in the table.

The required duration from the commencement of work to complete the dam construction and to start impounding is 43 months.

(c) Comparative Study on Construction Works

Both center core and concrete face rockfill dams are studied from the constructional aspect at various points of view such as construction material, embankment method, construction period and so on. The results are given as shown below:

Item	Center Core Rockfill Dam	Concrete Face Rockfill Dam
Impervious Zone (membrane)	◎ Impervious earth material shall be made by blending fine and coarse materials. The fine material excavated from spillway and main dam can be used. Embankment procedure is same as the other zones.	△ Slab concrete shall be constructed on upstream slope after embankment is completed. Upstream slope shall be compacted and smoothly trimmed very carefully. Bituminous coating shall be placed to protect slope.
Embankment	△ Embankment speed depends on impervious zone which susceptible to weather condition because of its fine-grained soil. The workable days for it are 172 days a year.	◎ Embankment speed is not restricted by material properties and less weather dependent. The workable days for earth and concrete works are 225 days and 252 days a year.
Temporary Facilities	◎ Stockyard and blending areas for impervious and semi-pervious materials are required. Exclusive plant for embankment is not required.	△ Stockyard and blending area for semi-pervious material is required. Slip-form, crane, trolley, guide rail and winch shall be employed for construction of concrete face slab.
Grouting	△ As blanket grouting shall be executed at impervious zone foundation, impervious materials filling shall follow grouting work progress.	◎ As all grouting works are executed in internal gallery, rock embankment can be proceeded irrespective grouting work progress.
Quality Control	○ Quality of impervious material is essential for dam safety. Its moisture content and gradation shall be strictly controlled and its placement and compaction shall be properly executed.	○ Quality of impervious membrane is essential for dam safety. Quality of concrete shall be strictly controlled and concrete shall be properly placed and cured to avoid cracks that will cause water leakage.
Flood Risk	△ Impervious zone will be easily damaged against overtopping.	◎ Impervious zone, which is not resistible against flooding, is not required.
Safety during Construction	◎ As filling area is relatively flat, and slopes are gentle, accidents during construction are hardly occurred.	△ Concrete face slab is constructed on steep upstream slope, there is higher possibility of accident occurrence during construction.
Construction Period	○ 43 months up to commencement of impounding.	○ 43 months up to commencement of impounding.

Note : ◎ : Advantage, ○ : Even, △ : Disadvantage

As shown in the foregoing discussion, it is concluded that center core rockfill dam and concrete face rockfill dam are almost equivalent from constructional aspect.

(2) Cost Comparison

The cost comparison is focusing on excavation for dam, grouting, gallery and embankment works. The costs of spillway, outlet facilities and diversion arrangement being common in both alternatives are not included in the estimate. Common items like cost of preparatory works and indirect cost such as site expense are not included. Detail cost estimates for both types are given in Table 6.2.7. The costs of each item are summarized as below:

Work Item	Center Core Rockfill Dam (Rp. x 10^6)	Concrete Face Rockfill Dam (Rp. x 10^6)
Excavation	1,637	2,033
Grouting	4,120	5,310
Gallery	2,477	3,121
Dam Embankment	27,506	19,172
Impervious Membrane	——	6,684
Total	35,740	36,320

It can be seen that concrete face rockfill dam is just a little more expensive than center core rockfill dam. The concrete face rockfill dam has a saving of about 25 % less embankment than center core rockfill dam, but the expensive impervious membrane, which consists of concrete face slab, toe structure and parapet wall, offsets the savings in embankment.

Generally, the higher the dam, the larger the embankment volume rapidly and the greater the cost advantage of concrete face rockfill dam. However, for not high dam, the expensive cost of the impervious membrane is a disadvantage because the cost of the membrane relative to the total cost is very large. This cost comparison shows clearly that concrete face rockfill type for the Jatibarang damsite does not have the cost advantage in comparison with center core rockfill type due to a small-scale dam.

(3) Conclusion

The foregoing discussions are summarized in the following table.

Item	Center Core Rockfill Dam	Concrete Face Rockfill Dam
Impervious Zone	◎ Relatively easy works	△ Difficult works and costly
Embankment Work	△ Susceptible to weather condition	◎ Less weather dependent
Construction Period	○ 43 months	○ 43 months
Post Construction Settlement under Impervious Zone	◎ Nearly all of the total foundation settlement occurs during construction.	△ It is so great that the face slab can not remain intact.
Repair Against Leakage	○ It is difficult to do anything except control leaks safely.	○ It is only available for inspection and repair of the face slab above EL.115.0 m.
Construction Cost	◎ 35,740 x 10^6 Rp.	△ 36,320 x 10^6 Rp.
Conclusion	◎ The most suitable alternative	△ Not suitable alternative

Note : ◎ : Advantage, ○ : Even, △ : Disadvantage

It is concluded that the center core rockfill type is the most suitable alternative for Jatibarang Multipurpose Dam due to the following reasons:

① Center core rockfill dam and concrete face rockfill dam are almost equivalent from constructional aspect considering their advantages and disadvantages. The construction periods of both types are also equal.

② Concrete face rockfill dam is just a little more expensive than center core rockfill dam. Concrete face rockfill dam has a saving of about 25 % less embankment volume than the center core rockfill dam, but the expensive impervious membrane offsets the savings in embankment

③ In case of the concrete face rockfill dam, there is a possibility of initial leakage. When the reservoir is filled with water for the first time, the excessive leakage will develop through the face slab because of cracks in concrete. As the foundation of Jatibarang Multipurpose Dam consists of soft rock, the post construction settlement of the foundation may be so great that the face slab can not remain intact. The leaks may require that the reservoir be emptied for repairs. The maintenance cost may increase.

④ Although relatively few concrete face rockfill dams have been built in Indonesia, Cirata Dam adopted concrete face rockfill type. The reasons are shown as below:

- Concrete face rockfill dam has the cost advantage in comparison with center core rockfill type due to a large-scale dam of 126 m in height.
- As the foundation consists of fresh and hard rock, the post construction settlement of the foundation is expected to be so small that cracks in concrete which cause the excessive leakage are not developed and the face slab can be used successfully.

6.2.6 Preliminary Design on Appurtenant Structures

Appurtenant Structures necessary for Dam

Spillway, outlet facilities for water use and diversion tunnel shall be designed as main appurtenant structures for Jatibarang Multipurpose Dam. The functions of these structures are described hereunder.

(1) Spillway

Spillways are provided to release surplus or flood water, which can not be contained in the allotted storage capacity of the reservoir. Since Jatibarang Multipurpose Dam is planned to have a flood control function as one of its purposes, the following two (2) features are considered:

① Function to regulate a 100-year probable flood with peak discharge of 290 m^3/s flows through the reservoir, resulting in the river flow discharge of 790 m^3/s at Simongan, by adding the joining flow from Garang and Kripik rivers.

② Sufficient capacity to accommodate the design flood with peak discharge of 1,600 m^3/s (PMF) after regulating the inflow through the reservoir.

(2) Diversion Tunnel

The objectives of the diversion tunnel are to divert the streamflow around or through the damsite during the construction period. They can minimize serious potential flood damage to the work in progress. The tunnel is designed to be capable of managing a 25-year probable flood that has been worked out as 280 m^3/s.

(3) Outlet Facilities for Water Use

The outlet facilities are to assure the reservoir yield, which is required for municipal water supply to Semarang City and river maintenance flow to the area downstream of the dam. The maximum out flow discharge is 2.69 m^3/s which corresponds to the required flow at the Simongan weir site. In the future with proposed Mundingan Dam and Inter-basin water transfer project, there is possibility to increase the maximum outflow to 6.0 m^3/s. The outlet facilities are designed to consider the planed maximum out flow discharge of 6.0 m^3/s. The intake sill should be high enough to prevent sediment deposits from flowing in it, but low enough to fully intake the filling water in the reservoir.

The hydropower generation is carried out subordinately using the released water necessary for downstream water use.

Layout of Appurtenant Structures

Since there is a very limited space for installation of the appurtenant structures due to the narrow valley, it is unreasonable that each structure selects the optimum location to be installed of its own. Accordingly, three (3) alternatives are studied to select the optimum combination of the locations for the structures. The features of the alternatives, which are adopted keeping in view the topographic conditions, are described hereunder.

Alternative L1 --- The layout plan is shown in Fig. 6.2.19.

Spillway is located on the left abutment adjacent to the dam body and can be connected with the downstream river channel smoothly. Side channel spillway with the bathtub type overflow weir is adopted, in which the overflow crests can be equipped on the both sides and the end of the side channel.

The Inclined intake for outlet works is located at the right abutment upstream of the toe of the dam body. A tunnel with about 405 m long is provided to install the outlet pipe. A hydropower station and a valve house can be located on the right side embankment of the spillway-stilling basin.

A diversion tunnel located along the left abutment is arranged to about 441 m long.

Alternative L2 --- The layout plan is shown in Fig. 6.2.20.

Spillway is located on the left side hill away from the dam body and can be connected with the downstream river channel smoothly. Side channel spillway with the T-shape type overflow weir is adopted.

The outlet works are arranged at the same location as Alternative L1.

A diversion tunnel located along the left abutment is arranged to about 474 m long.

Alternative R1 --- The layout plan is shown in Figs. 6.2.21.

Spillway is located on the right abutment adjacent to the dam body. It can not be connected with the downstream river channel smoothly. The side channel spillway is adopted, in which the overflow crest is equipped on only one side of the side channel.

The Inclined intake for outlet works is located at the left abutment. A tunnel with about 293 m long, which is shorter than the others, is provided to install the outlet pipe. It is arranged parallel to the diversion tunnel. A hydropower plant and a valve house can be located on the left side embankment of the spillway-stilling basin.

A diversion tunnel located along the left abutment is arranged to about 437 m long.

The result of the rough cost estimate are given below:

Alternative	Work Item	Work Quantity		Construction Cost (Rp. x 10^6)
Alternative L1	Spillway	Excavation	470,000 m^3	41,000
		Concrete	49,000 m^3	
	Outlet Works	Length of Tunnel	405 m	
	Diversion Tunnel	Length of Tunnel	441 m	
Alternative L2	Spillway	Excavation	700,000 m^3	46,300
		Concrete	57,000 m^3	
	Outlet Works	Length of Tunnel	384 m	
	Diversion Tunnel	Length of Tunnel	474 m	
Alternative R1	Spillway	Excavation	710,000 m^3	44,500
		Concrete	58,000 m^3	
	Outlet Works	Length of Tunnel	293 m	
	Diversion Tunnel	Length of Tunnel	437 m	

From the technical and economical points of view, Alternative L1 is concluded to be the most suitable combination of the locations for the appurtenant structures.

The preliminary design of the appurtenant structures following the selected combination of the locations is described hereinafter.

Spillway

(1) General

The bathtub type of side channel spillway is adopted keeping in view the topographic conditions. This type mainly consists of five (5) portions, namely, overflow weir, side channel, control portion, chute and stilling basin. The plan and profile of the spillway are shown in Figs. 6.2.22 and 6.2.23.

Spillway is founded on more than CL-H crass rock except for the chute with the low wall height. The foundation level of overflow weir and side channel wall is EL. 136.6 m.

(2) Overflow Weir of Service Spillway

The service spillway, which corresponds to the flood control plan for 100-year flood, is an ungated overflow weir having ogee shaped crest. It has the length of 15.0 m and the height of 2.9 m. The crest level is set at EL. 148.9 m.

According to the flood control plan, when a 100-year probable flood with peak discharge of 290 m^3/s flows into the dam reservoir, the peak outflow is regulated at 40 m^3/s. The Surcharge Water Surface is set at EL. 151.8 m, which corresponds to the flood control capacity 3,100,000 m^3 including 20 % allowance. The outflow at the

Surcharge Water Surface EL. 151.8 m is estimated at 150 m^3/s.

(3) Overflow Weir of Emergency Spillway

The emergency spillway is not for the flood control but for the dam safety against excess design flood discharge. The emergency spillway is located at the both sides on the side channel. The overflow weir of the emergency spillway has the length of 60.0 m. The crest level is set at the Surcharge Water Surface Elevation 151.8 m.

The Maximum Water Surface, which corresponds to the design discharge of 1,600 m^3/s (PMF), is set at EL. 155.3 m.

(4) Stilling Basin

The design discharge of stilling basin should be the largest of the following discharges. In this case, the design discharge of stilling basin becomes 340 m^3/s.

(a) Outflow capacity at the Surcharge Water Surface (160 m^3/s)

(b) 100-year probable flood discharge (340 m^3/s)

Regarding the energy dissipater type, a hydraulic jump basin with endsill is employed. The width of apron is determined at 24 m, considering the width of the river channel at the downstream section. The length of the apron and the endsill height are 44.0 m and 6.5 m, respectively.

Diversion Tunnel

(1) General

The layout plan and profile are shown in Figs. 6.2.24 and 6.2.25.

Location of the inlet structure is determined at the concave side on the left bank of the river so that the tunnel length will result shortest. The location of the outlet is designed to easily make the centerline of discharge into the original center of the river flow.

The geological features along the diversion tunnel route are alternating strata that consist of pyroclastic rock unit and sedimentary rock unit from Tertiary to Quaternary (Pleistocene). Tunnel excavation can be made without any blasting operations.

The longitudinal gradient of diversion tunnel is determined straight by the elevation of inlet and outlet structure together with consideration of other conditions such as the height of access road, temporary facilities at the inlet and outlet portal. The elevation of the outlet bed is set at EL 83.8 m considering the riverbed elevation at the downstream. The longitudinal gradient is set at 1/30 considering flow velocity and the elevation of the inlet bed.

(2) Tunnel Cross Section

Generally, three types of the cross sectional shape are employed, namely, standard horseshoe, top-half-round with bottom-half-square and round for tunnels shapes.

Round shape tunnel is considered not suitable because it become unsafe due to unstable flows caused by frequent floods of less than design flow discharge. Although the top-half-round bottom-half-square shaped tunnel has certain advantage in construction stage but it has disadvantage in concentration of stress at corners of tunnel section. Therefore, it is not recommended when the geological condition of the rock at site is not strong. Hence, standard horseshoe (2r) shaped tunnel is recommended.

In the case of open channel flow in the standard horseshoe shaped tunnel, 80 % of the height of tunnel will be used as actual flow section for the design discharge to avoid the creation of self-priming phenomenon as common practice.

Discharge amount is calculated in accordance with the hydraulic formula, design cross section is decided as follows:

Height and width : 5.6 m (r = 2.8 m)

Outlet Facilities

(1) General

The Plan and profile of the outlet facilities are shown in Figs. 6.2.26 and 6.2.27.

The intake structure is located at the right abutment upstream of the toe of the dam body. It is designed to the inclined type with 1.0 vertical to 1.4 horizontal as shown in Fig. 6.2.28. The bulkhead gate is operated when the outlet pipe is necessary to be drained for inspection, maintenance and repair without lowering the reservoir water surface.

The outlet pipe is installed in the tunnel located in the right abutment. A branched pipe is connected with the outlet pipe just before the hydropower station to release the water directly, without passing through the hydropower station.

The minimum outflow from the intake facilities is 0.26 m^3/s which corresponds to the maintenance flow at the damsite.

The maximum outflow from the intake facilities is 2.69 m^3/s which corresponds to the required flow at the Simongan weir site. However, in the future with proposed Mundingan Dam and Inter-basin water transfer project, there is possibility to increase the maximum outflow to 6.0 m^3/s

(2) Outlet Pipe and Control Gate

The diameter of the outlet pipe is determined at $D = (4*Q/3.14/V)^{0.5} = 1.4$ m, using the condition of the discharge $Q = 6.0$ m^3/s and the velocity $V = 4.0$ m/s.

The flows are regulated by a control gate installed at the downstream end of the outlet pipe. Considering the minimum and maximum design discharge of 0.26 m^3/s and 6.0 m^3/s with the Low Water Surface (EL. 136.0 m), control gates of 650 mm diameter and 250 mm diameter are required.

Other Structures

(1) Dam Management Complex and Hydropower Station Complex

For the operation, maintenance and monitoring of Jatibarang Multipurpose Dam and Jatibarang Hydropower Station, operation/management building is planned at the damsite, which consists of office buildings for the dam and the hydropower station and other related buildings.

The dam management complex is located at the left abutment of Jatibarang Multipurpose Dam and the one for the hydropower station complex is located at the immediate downstream of the dam body.

The operation/management office compound for Jatibarang Multipurpose Dam and Jatibarang Hydropower Station consists of ten buildings whose outlines are presented in the table below:

Dam Management Complex

	Building Name	Story	Total Floor Area (m^2)	Unit
1	Dam Administration Building	3	594	1
2	Staff house 1 (Guest house)	1	74	1
3	Staff house 2	1	49	4
4	Mushola	1	72	1

Hydropower Station Complex

	Building Name	Story	Total Floor Area (m^2)	Unit
1	Power Station	2	390	1
2	Garage	1	184	1
3	Guard House	1	14	1

The layout plans of the proposed buildings are shown in Figs. 6.2.29 and 6.2.30.

(2) Preliminary Design of Approach Bridge to Goa Kreo Cave

Since the Normal Water Surface EL. 151.8 m is higher than the existing approach road to Goa Kreo, a new bridge is designed for pedestrian only. The location of the approach bridge to Goa Kreo Cave is shown in Fig. 6.2.31.

After comparison of two types, namely RC girder bridge and suspension bridge, RC girder type was selected from economical reason. (refer to Table 6.2.8)

Preliminary design results are shown in Fig. 6.2.32 and design features are summarized as follows:

(a) Span length 17.0 m × 4 spans

(b) Width; 2.0 m

(c) Clearance above the Surcharge Water Surface (EL. 151.8 m); 0.6 m

(d) Live load; 500 kg/m2 (BINAMARGA standard for a pedestrian bridge)

(e) Foundation Structure; Spread foundation

6.2.7 Summary of Definitive Plan

The plan, longitudinal profile and typical cross section for Jatibarang Multipurpose Dam are shown in Figs. 6.2.33 to 6.2.35. The features are summarized hereinafter.

Dam and Reservoir

(1) Reservoir

Catchment Area : 53.0 km^2

Reservoir Surface Area	:	1.10 km^2
Maximum Water Surface	:	EL. 155.300 m
Surcharge Water Surface	:	EL. 151.800 m
Normal Water Surface	:	EL. 148.900 m
Low Water Surface	:	EL. 136.000 m
Gross Storage Capacity	:	20,400,000 m^3
Effective Storage Capacity	:	13,600,000 m^3
Flood Control Capacity	:	3,100,000 m^3
Water Use Capacity	:	10,500,000 m^3
Sediment Capacity	:	6,800,000 m^3

(2) Dam

Height above Foundation	:	77.0 m
Crest Elevation	:	EL. 157.000 m
Foundation Elevation	:	EL. 80.000 m
Upstream Slope	:	1 : 2.6
Downstream Slope	:	1 : 1.8
Crest Length	:	200.0 m
Crest Width	:	10.0 m

Spillway and Outlet Facilities

(1) Spillway

Design Flood (Inflow)		
Probable Maximum Flood	:	1,600 m^3/s (inflow into the reservoir)
100-year	:	290 m^3/s (inflow into the reservoir)
Design Discharge for Energy Dissipater	:	340 m^3/s
Overflow Crest (Service Spillway)		
Crest Elevation	:	EL. 148.900 m
Crest Width	:	15.0 m
Overflow Crest (Emergency Spillway)		
Crest Elevation	:	EL. 151.800 m
Total Crest Width	:	60.0 m
Total Length of Spillway	:	307 m
Stilling Basin	:	24.0 m wide x 60.0 m long
Spillway Bridge (PC Girder Type)	:	5.0 m wide x 23.94 m long

(2) Outlet Facilities

Maximum Design Discharge	:	6.0 m^3/s

Minimum Design Discharge	:	0.26 m^3/s
Intake Structure	:	Inclined Type
Bulkhead Gate	:	1.4 m height x 2.0 m span
Emergency Gate	:	1.4 m height x 2.0 m span
Steel Outlet Pipe	:	393 m long x 1.4 m dia.
Control Gate	:	650 and 250 mm dia.

Diversion Tunnel

Design Discharge	:	280 m^3/s (25-year return period)
Tunnel Section	:	Horseshoe with 2r (5.8 m)
Longitudinal Gradient	:	1/30
Tunnel Length	:	441 m
Tunnel Inlet Elevation	:	EL. 98.500 m
Crest of Temporary Cofferdam	:	EL. 113.000 m

Dam Management Complex and Hydropower Station Complex

Dam Management Complex		
Administration Building	:	594 m^2
Staff House 1 (Guest House)	:	74 m^2
Staff House 2	:	49 m^2 x 4 unit
Mushola	:	72 m^2
Hydropower Station Complex		
Administration Building	:	390 m^2
Garage	:	184 m^2
Guard House	:	14 m^2

Approach Bridge to Goa Kreo Cave

Span Length	:	17.0 m x 4 spans = 68.0 m
Width	:	2.0 m

Work Quantities

Dam Excavation	:	198,000 m^3
Spillway Excavation	:	465,000 m^3
Blanket Grouting	:	2,500 m
Curtain Grouting	:	17,700 m
Dam Embankment	:	790,000 m^3
Impervious Zone	:	124,000 m^3
Semi-pervious Zone	:	65,000 m^3
Inner Pervious Zone	:	105,000 m^3

Outer Pervious Zone	:	496,000 m^3
Concrete Work	:	57,000 m^3
Spillway	:	49,000 m^3
Outlet Facilities	:	3,000 m^3
Gallery	:	5,000 m^3
Tunnel for Outlet Facilities	:	393 m
Diversion Tunnel	:	441 m

6.3. Hydropower Generation

6.3.1 Present Condition of Electric Power Generation in Central Java Area

Electric power utilities in Indonesia are managed and controlled mainly by the State Electricity Corporation (PLN).

As of 1996/1997, the power generation capability of the Java-Bali Power System of PLN is 11,423MW by the total installed capacity (approximately 71% of the capacity is by all PLN utilities) and about 54,985 GWh is generated annually. The Java-Bali Power System includes the major power stations of Mrica Hydro (184.5 MW), Tambaklorok Steam Oil (300 MW), Tambaklorok Gas and Steam (711 MW) in Central Java.

The major power sources of the Java-Bali Power System are located in West Java, Central Java, East Java and Bali as listed below. In Java Island, a 500 kV trunk transmission line runs from Suralaya in West Java to Surabaya in East Java. In addition, networks of 500 kV and 150 kV transmission lines have been constructed to make practical use of the trunk line.

To cope with the sharply increasing demand for electricity, the government of Indonesia is planning to reinforce the generating capability of PLN and the private sectors all over the country. The power capacity of the Jatibarang Hydropower Station, a component of a multipurpose dam, is only 1.5 MW of installed capacity and 6.0 GWh in annual energy is to be generated, which is regarded to be minor as compared with the power sources.

The generated energy by Jatibarang Hydropower Station will be easily consumed within the electricity supply areas in the vicinity of Semarang City through the proposed low voltage networks of 20 kV transmission line and the existing distribution lines. It is noted that the Jatibarang Hydropower Station will be play an important role in providing a power source for the expanding rural electrification in the areas, whose electrification rate (house-wise) in Java Island is still 26.7%. Furthermore, this will be useful to stabilize the voltage fluctuation in 20kV transmission lines in the areas.

Java-Bali Power System

System	West Java		Central Java		East Java		Bali	
Hydropower (MW)	Saguling	700	Mrica	184.5	Karangkates	105		
	Cirata	800	Garung	26.4	Wlingi	54		
	Jatiluhur	150	Jelok	20.5				
			Wadaslintang	16				
			Wonogiri	12.4				
			Tulis	13				
Steam Oil (MW)	Muara Karang	700	Tambaklorok	300	Gresik	600	Gilimanuk	145
	Tanjung Priok	150			Paiton	800		
	Suralaya	3,200						
Gas Turbine (MW)	Tanjung Priok	267	Tambaklorok	21.4	Grati	342		
	Pulogadung	130						
	Sunyaragi	80						
Gas + Steam (MW)	Muara Tawar	435	Tambaklorok	190+ 521	Grati	527		
Geo-Thermal (MW)	Gunung Salak	165						
	Kamojang	60						

6.3.2 Fundamental Condition of Hydropower Generation

Results of Feasibility Study

(1) Outline of the Planning of Dam Project

The Jatibarang Multipurpose Dam Project, from the viewpoint of the investment effect in terms of EIRR, and less impact on natural and social conditions by this project, has been selected as the first priority project from the development plans which were discussed under the Flood Control and Water Resources Development Master Plan.

The purpose of the project consists of flood control, municipal water supply, and hydropower generation. Among them, hydropower generation is included in one of the purposes in accordance with the policy of the Indonesian government, which makes it a rule to construct a hydropower station in every dam project.

(2) Scale of Hydropower Station

In preparation for planning of the Jatibarang Multipurpose Dam Project, the priority of the purposes of the Project has been given as follows:

First Priority : Flood Control and Water Supply

Second Priority : Hydropower Generation

Among them, alternative plans were prepared for flood control, water supply and hydropower generation. Those plans were compared from the economic viewpoint. As a result, it was found that the benefit from flood control and water supply was enormously greater than that from hydropower generation. Therefore, the plan was so selected that the combination of flood control and water supply purpose and hydropower generation purpose should generate the largest benefit. Since no capacity of reservoir was considered for hydropower generation, hydropower generation was designed to use water released from the dam for water supply. In the Feasibility Study stage, concerning the scale of hydropower station, the maximum discharge was so selected as to be equivalent to the annual water utilization factor of 60 %, and subsequently 1,500 kW was given as the installed capacity.

(3) Staged Development

According to the Water Resources Development Master Plan, the following development of Jatibarang Multipurpose Dam, Mundingan Dam is scheduled to be constructed upstream of the Jatibarang Multipurpose Dam as the second stage. These two dams are planned to be operated under the same system. Furthermore, as the third stage in the future, the surplus water out of the Burorong river basin lying next to the Mundingan river basin is planned to be diverted into the Mundingan reservoir by means of interbasin transfer. The development proceeds as follows:

Stage 1 : Operation with Jatibarang reservoir only

Stage 2 : Operation as series reservoirs with Jatibarang and Mundingan reservoirs

Stage 3 : Stage 2 (above) + receiving water through interbasin transfer from Burorong river basin.

The above mentioned future scheme is also taken into consideration in the Definitive Plan Study.

(4) Economic Evaluation of Hydropower Generation Plan

In the economic evaluation of Jatibarang hydropower generation plan in Feasibility Study, a diesel power station of 1,200 kW is assumed to be the alternative power source, and the corresponding power expenses are regarded as the benefit of hydropower generation. According to the price level as of July 1992, and by applying the 10 % discount rate, EIRR, B/C and NPV (Net Present Value) are computed as follows:

EIRR	5.9 %.
B / C	0.66
NPV	Rp -3,400 $\times 10^6$

The above mentioned way of economic evaluation is also adopted in the Definitive Plan Study.

Hydrological Data

As a result of the study of water supply plan, the water levels and discharges concerning Jatibarang reservoir at Present Stage and Future Stage have been calculated. The data covers 30 years, ranging from 1967 to 1996, and are used on a 5-day basis. In addition, no difference is found in the fluctuation of water level in the reservoir between Future Stage Plan and Present Stage Plan. This means that the operating water level remains unchanged; however, the discharge is changed. (refer to Tables 6.3.1 to 6.3.3)

The drawings prepared in the definitive plan of the dam are used for the definitive plan of the hydropower station.

Location and Layout of Hydropower Station

(1) Location

In consideration of the following advantages, the hydropower station is planned to be located, at the immediate downstream of the dam on the right bank of the energy dissipator.

- The access from the existing public road is easy, in respect to the transportation of construction materials and the operation and maintenance after the completion of the hydropower station.
- The short waterway prevents the occurrence of head losses in the waterway and results in an increase in project output.
- Easy access to the dam control building allows power/control cable to be short and inspection to be readily conducted.

(2) Layout

The water to be used for hydropower generation branches off from the outlet pipe for water supply, passes to turbine, and discharges by way of short tailrace into the downstream end of the energy dissipator for flood control. Therefore, the facilities between intake and branch point of outlet pipe are used both for water supply and hydropower generation.

6.3.3 Hydropower Generation Plan

Criteria for Selecting the Scale of Hydropower Station

The scale of Jatibarang hydropower station, or the maximum plant discharge in this case, is determined from the reservoir discharge operated for water supply purpose.

The maximum plant discharge of 2 ~ 6 m^3/s or the output of 1,000 ~ 3,000 kW have been studied and compared, considering the result of reservoir operation for 30 years, as shown in Tables 6.3.1 to 6.3.3.

Since the scale of hydropower station is small, one unit of turbine is assumed to serve the purpose; however, the plan of installing two units of turbines has been also studied, allowing for the capability of generating energy even when discharge is small.

Various comparisons have been conducted on scales and the number of units, and evaluated from the economic point of view based on cost and benefit. Then the final plan has been selected according to the results of EIRR.

That is, on the assumption that a diesel power station of 1,200 kW is the alternative power source to Jatibarang hydropower station, the cost required for the construction and maintenance/operation of the diesel power station is regarded as the benefit of Jatibarang hydropower station.

For the calculation of EIRR, the time to commit the hydropower station is assumed to be in 2007 at Present stage development plan and in 2012 at future stage, if it is developed, according to the revised schedule.

Alternative Plans

For the planning of hydropower station, several alternative plans are prepared, in which the following four elements are employed as parameters.

- Maximum discharge of hydropower station
- Number of units (one or two)
- Allocation of discharge, in case of two units

- Prospect of Future Stage development

Among these, Future Stage development is studied according to the number of unit, from Case A to Case D as described below:

Case A : One unit (#1 unit) appropriate for the reservoir operation plan under Present Stage development plan is first installed. Subsequently at the time an available discharge increases in execution of Future Stage development, the second unit (#2 unit) will be installed. (Two units plan)

Case B1 : On the premise that Future Stage development will be proceeded. One unit is installed at present in consideration of discharge increment in the future. (One unit plan)

Case B2 : Two units are installed at present on the same assumption as Case B1. (Two units plan)

Therefore, both Case A and Case B are expected to produce Power Output increment at Future Stage as benefit.

Case C : One unit is installed, not considering Future Stage development but considering only reservoir operation plan at Present Stage (One unit plan).

Case D : Two units are installed on the same assumption as above (Two units plan).

Table 6.3.4 shows the features of alternative plans.

Power Output and Optimum Scale

(1) Conditions of the Computation of Power Output

(a) Type of Turbine Unit and Combined Efficiency

The type of hydroturbine to be used can generally be determined by the head and the discharge available for the power plant.

Generally Pelton turbine is used for high head, vertical Francis turbine is applied for large discharge with medium head, horizontal Francis turbine is applied for small discharge with medium head and Propeller turbine is used for low head. The selection of the turbine type according to head and discharge can be referred to Fig.6.3.1.

Horizontal shaft type Francis turbine is selected for the Jatibarang Hydropower Station because the effective head and discharge is in the range of horizontal shaft type Francis turbine.

The combined efficiency of turbine and generator is computed by the following equation, which was derived from field-proven units.

Combined Efficiency $e = 0.0226 \ln (\text{Rated Capacity kW}) + 0.819$

(b) Determination of Normal Effective Head and Reservoir Low Water Level

The net head to be used for the design of the maximum output is defined as the normal effective head. Corresponding to this head, the water level of the reservoir is defined as the normal intake water level for power generation, and the water level of tailrace of the hydropower station is defined as the normal tail water level.

In general, the normal intake water level, in case of reservoir type, is used one third of water depth between low water level (LWL) and the high water level. However, according to the fluctuations of the reservoir water levels under the water supply plan (refer to Table 6.3.5), the number of the days on which the reservoir water level keeps high water level of EL 148.62m accounts for 290 days of a year (79.3 %). Therefore the normal water level (NWL) of EL 148.62m for water supply plan is decided also as the NWL for hydropower generation, instead of one-third water level method mentioned above.

LWL for hydropower generation was decided higher than the design sediment level of the reservoir (EL.136.0m) in order that the turbine unit should be prevented from flowing the sediment from the weir. According to the fluctuations of annual reservoir water level (refer to Table 6.3.5), the number of the days on which the water level is lower than EL138.0m accounts for 1 % of the year and resultant decrease of annual generated energy is negligible.

Therefore, the water level of EL138.0m is adopted for the LWL for hydropower generation. Thus, the following two LWLs are determined.

LWL EL136.0m for Water Supply
LWL EL138.0m for Hydropower Generation

(2) Power Output

The plant size has been studied on 20 alternative cases which range from 951 kW to 2,851 kW of installed capacity. The results of the study are shown in Table 6.3.4. The output in plant size Alt.12, which is planned to be adopted on the basis of the cost evaluation discussed in the next section, is as follows:

One Turbine Unit Plan	1,440 kW
Annual Energy (Present stage development)	5,790 MWh
Annual Energy (Future stage development)	8,307 MWh
Plant factor (Present stage development)	46 %
Plant factor (Future stage development)	66 %

The above annual energy of present and future stage are the average for 30 years, respectively. Table 6.3.6 shows power output of each individual year.

In the Present Stage development for water supply, annual energy production for 30 years varies in the range from 2,640 MWh (1985, the driest year) to 8,779 MWh (1971, the wettest year) with 5,790 MWh on an average.

In the Future Stage development for water supply, on the other hand, annual energy production varies in the range from 6,898 MWh to 9,188 MWh with 8,307 MWh on an average.

Table 6.3.7 shows the variation of output on a 5-day basis for the year of 1990, in which outputs are observed close to the average, and the year of 1985, the driest year.

Power output under a total discharge of 3.0 m^3/s was compared as an example concerning the number of turbine unit, one and two, and also by means of the allocation of different discharges to each turbine. The results are shown in Fig. 6.3.2.

(3) Optimum Scale

The results of the economic evaluation on the alternative plans of 20 cases are shown in Table 6.3.8 and Fig. 6.3.3. The evaluation indexes, B/C, NPV, and EIRR are used in Table 6.3.8. As shown in Fig. 6.3.3 (b), the Alt. 12 (one unit turbine plan), which brings the maximum EIRR, has been selected as optimum scale. The evaluation indexes of the optimum scale are as follows:

Item	Present Stage	Future Stage
EIRR	6.90 %	9.35 %
NPV	Rp. - 1,887 × 10^6	Rp. - 454 × 10^6
B / C	0.739	0.937

Fig. 6.3.3 (c) shows the differences in EIRR between the two cases of two units turbine plan, discharge ratios 2:1 and 1:1 and case of one unit turbine plan, respectively. From this figure also, it can be seen that one unit turbine plan shows the higher EIRR than two units turbine plan.

Therefore, it is concluded that one unit should be installed at Present Stage development, and no further extension is considered in the Future Stage development.

(4) Installed Capacity and Economic Evaluation

The total head for hydropower generation is fixed at 65.99m and as the result, installed capacity of the hydropower station was calculated at 1,500kW, as shown below:

$$
\begin{aligned}
P &= 9.8 \times E_T \times E_G \times He \times Q \\
&= 9.8 \times 0.858 \times 0.951 \times 64.3 \times 3.0 \\
&= 1{,}542 \text{ kW} \\
&\fallingdotseq 1{,}500 \text{ kW}
\end{aligned}
$$

where,

E_T = 0.858	:	efficiency of turbine
E_G = 0.951	:	efficiency of generator
He = NWL.148.9m − TWL.82.91m − h_{loss}	:	Net head
= 64.3m		
h_{loss} = 1.690m	:	Head loss
Q = 3.0m^3/s	:	Discharge

The annual average energy of present and future stage are 6,020 MWh and 8,640 MWh.

6,020 MWh = 5,790 MWh × 1,500/1,440

8,640 MWh = 8,307 MWh × 1,500/1,440

Breakdown of the cost used in the economic evaluation is shown in Tables 6.3.9 to 6.3.12.

Item	Optimum Scale Study	Definitive Plan
Reservoir NWL (EL m)	148.6	148.9
Tailrace WL (EL m)	85.9	82.91
Gross Head (m)	62.7	65.99
Power Output (kW)	1,440	1,500
Annual Energy (MWh)		
Present Stage	5,790	6,020
Future Stage	8,307	8,640
Economic Evaluation		
Present Stage		
EIRR(%)	6.90	7.04
B/C	0.739	0.751
NPV (Rp. $\times 10^6$)	-1,887	-1,845
Future Stage		
EIRR(%)	9.35	9.51
B/C	0.937	0.952
NPV (Rp. $\times 10^6$)	-454	-355

Summary of Definitive Plan

The results of the definitive plan study are summarized below:

Item	Definitive Plan
(1) Hydropower Generation	
Maximum plant discharge (m^3/s)	3.00
Maximum gross head (m)	65.99
Installed capacity (kW)	1,500
Number of generator at future stage	no extension
Annual energy (MWh)	6,020
(2) Dam and Water Level	
Dam height (m)	77.0
Reservoir NWL (EL. m)	148.9
Reservoir LWL (EL. m)	138.0(*1)
Tail water level (EL. m)	82.91
(3) Economic Evaluation	
NPV (Rp$\times 10^6$)	-1,845
B/C	0.75
EIRR (%)	7.0

Note : (*1) LWL for hydropower generation

6.4 Project Evaluation

6.4.1 Flood Control Works

General

In general, a project for public works concerning flooding and/or inundation will be evaluated taking engineering and economic aspects into consideration. The engineering aspects are studied on the technical feasibility of the project from the viewpoint of construction, operation and maintenance.

Economic analysis appraises a project under study in terms of a national and/or a regional social economy by comparing and measuring its economic costs and benefits. In other words, economic analysis evaluates a degree of economic impacts on a project under study that would bring about in the national and/or regional social economy.

Methodology

The Project Evaluation this time is a review of the evaluation executed in the Feasibility Study made by JICA Study Team in 1993, so the way of the project evaluation from the viewpoint of economic aspects in this stage of the Project is made by the same manner applied for the evaluation in the Feasibility Study at that time, namely, the mesh method, reviewing the unit damageable value based on a price index for housing for determining of value of general assets in the flood prone area.

Value of assets and/or damageable value of assets depend surely on a current price level because that they are usually re-evaluated for estimation of amount of damages by using the current price level when they are damaged and/or lost caused by flood and/or inundation. For estimation of existing damages and updated damageable value of properties consisting of buildings and indoor movables, an annual average increase ratio of price index of 6.74 % for the period from 1992 to 1997 in Semarang City as shown in Table 6.4.1 is applied to the standard construction prices of building with the same assumptions used in the said Feasibility Study.

Table 6.4.2 shows a result of estimation of a damages increasing rate using 10-year probable flood. As shown in this table, the increasing rate of damages might be 1.495 comparing the damages in 1992 as a base. This rate can be applied for other scale of floods because of no any changes of such assumptions as standard construction price for residence, industrial and commercial buildings, depreciation rate, rate of tax, building density and so forth as mentioned above.

The economic internal rate of return (EIRR) is calculated and used as an index of economic feasibility. This EIRR is defined by the following formula:

$$\sum_{t=1}^{t=T} \frac{C_t}{(1+R)^t} = \sum_{t=1}^{t=T} \frac{B_t}{(1+R)^t}$$

where, T = the last year of the project life,

C_t = an annual economic cost flow of the project under study in year t,

B_t = an annual benefit flow derived from the project in year t, and

R = the Economic Internal Rate of Return (EIRR).

The project life is assumed at 50 years after completion of the said flood control works.

Flood Damages

Table 6.4.3 shows a result of estimation of probable flood damages by flood scale of each return period by using the said damages increasing rate of 1.495 for general assets consisting of buildings and their indoor movables of residence, industrial and business buildings , and for public facilities and business suspension losses. In this case, rates for applying for estimation of damages of public facilities and business suspension losses are 46.8 % and 6.0 % to the said damages of general assets respectively which are the same rates applied in the Feasibility Study.

Total probable flood damages are Rp. 176,669 million for 10-year flood, Rp. 360,264 million for 25-year, Rp. 545,983 million for 50-year flood, and Rp. 805,102 million for 100-year flood.

The annual average flood damages are estimated based on the said flood damages by each flood scale at Rp. 8,833 million for 10-year flood, Rp. 24,941 million for 25-year flood, Rp. 34,004 million for 50-year flood and Rp. 40,759 million for 100-year flood as shown in Table 6.4.4.

Identification of Economic Benefit

River improvement works for West Floodway/Garang River as Flood Control Works this time is designed to relieve such general assets, industrial and business buildings and their indoor movables, and public facilities and business suspension losses from the damages caused by 25-year flood. Therefore, the above mentioned annual average probable damages caused by 25-year flood, namely the amount of Rp. 24,941 million, are only converted into the benefit due to the said river improvement works when there is no any dam construction works.

A dam constructed in the upper streams of the flood prone area has usually a function of flood control, and the Jatibarang Multipurpose Dam located in the upper streams of flood prone area is one of main components of the Project. According to the design criteria, damages caused by 100-year flood can be eliminated in combination of river improvement and the said dam construction works.

Accordingly, the damages caused by 100-year flood, namely Rp. 40,759 million, can be converted into benefit due to completion of both the said two works.

Identification of Economic Cost

Economic cost of a project is identified as opportunity cost of the project. In this case, if goods and services would be invested in the project under study, they could no longer be utilized for other projects. This implies that the benefits of the other projects could have been created would be sacrificed. These sacrificed benefits of the other projects are called opportunity cost of the project. A project cost consists of foreign currency portion and local currency portion.

Firstly, gross construction costs are separately estimated on river improvement works and dam construction works based on unit prices and work volume as mentioned in previous CHAPTER, and these gross construction costs includes construction base cost, engineering service cost for supervision, cost for administration, value added tax, cost for compensation, physical contingency and price contingency. In this case, the total dam construction cost is allocated to river improvement works based principally on water utility with a rate of 35.40 % according to the design criteria of the dam construction works.

(1) Foreign currency portion

Using the said gross construction cost, an economic cost of the Project is estimated. In this study, the construction base cost includes labour cost, cost for materials, and cost for equipment. For the foreign currency portion, these costs for materials and equipment are estimated in either Cost Insurance Freight (CIF) price or Free on Board

(FOB) price. These international prices are assumed to reflect an economic cost directly.

Value added tax is not included in the foreign currency portion because that the said tax should be paid by local currency based on the taxation regulation in Indonesia.

For economic evaluation of the Project, such transfer cost as contractor's overhead and profit should be deducted, and price contingency should be excluded because that comparison of cost and benefit is made by net present value.

(2) Local currency portion

Because it is presumed that local markets in developing countries are distorted by price controls and other regulations, prices in the domestic markets do not reflect economic scarcity of goods and services. This means that the prices can not be used to identify economic costs of local procurement and have to be converted into economic prices.

In economic analysis of a project, conversion factors are used to convert the costs in domestic markets into economic costs of a project.

Using the export and import statistics, a standard conversion factor (SCF) is estimated. The SCF converts the domestic commodity prices into the economic prices that can be assumed to reflect the economic scarcity of the local costs (refer to Table 6.4.5).

However, the SCF is applied to only tradable goods. The economic cost of non-tradable goods and services have to be separately evaluated. Conversion factors of land, skilled and unskilled labours are respectively estimated.

Economic wage of unskilled labours to be employed for the construction works is assumed to be 90 % of the actual market wage, taking of the employment opportunity of laborers in the study area into consideration.

Economic cost of land compensation including other compensation cost such as the cost for removal of houses is assumed to be 100 % of the financial cost, taking account of the opportunity cost of land use.

(3) Total Economic Cost

The estimated economic cost is shown in Table 6.4.6, and summarized as follows:

Economic Cost for River Improvement Works

(Rp.10^6)

Year	FC portion	LC portion	Total
2000/01	2,889	3,422	6,311
2001/02	17,134	17,984	35,118
2002/03	14,207	24,576	38,783
2003/04	16,839	12,457	29,296
Total	51,069	58,440	109,508

Economic Cost for Jatibarang Multipurpose Dam Construction Works to be Allocated for Flood Control

(Rp.10^6)

Year	FC portion	LC portion	Total
2000/01	644	702	1,346
2001/02	3,156	4,182	7,339
2002/03	4,395	5,362	9,757
2003/04	10,062	9,039	19,101
2004/05	6,816	4,353	11,169
Total	25,074	23,638	48,712

(4) Cost for Operation/Maintenance and Replacement

Financial costs for operation/maintenance (OM cost) and annualized replacement cost (cost for R) are estimated by work items at Rp.480 million per annum for river improvement works and at Rp.570 million for Jatibarang Multipurpose Dam construction works.

From these financial costs, an economic cost in total is estimated at Rp.457 million per annum for river improvement works and at Rp.176 million for Jatibarang Multipurpose Dam construction works to be allocated for flood control by the same manner for estimation of the said economic construction cost. These costs for OM and R will be a burden to the Project until the end of the project life of 50 years after completion of the river improvement works. Detail of calculation process is also shown in Table 6.4.6.

Economic Evaluation of Flood Control Works

The evaluation of the flood control works as a component of the Project is made by using cash flows of the said costs and benefits as shown in Table 6.4.7. The results are also shown in the table and summarized below.

EIRR (%)	19.77%
B/C	1.78
NPV (Rp.10^6)	72,201

In this case, B/C rate is a comparison result of benefit and cost in present value of them, and NPV means net cash balance between benefits and costs also expressed by their present value. For calculation of present value, a discount rate of 12 % is applied as same as that in similar projects in Indonesia.

From the viewpoint of EIRR, the rate is calculated at 19.8 % as indicated above, while the rate of B/C is 1.78. On the other hand, the amount of net cash balance is calculated Rp. 72,201 million as shown in the above table. The annual average benefit is Rp. 40,759 million as shown in Table 6.4.7 in combined case of the river improvement works and the dam construction works.

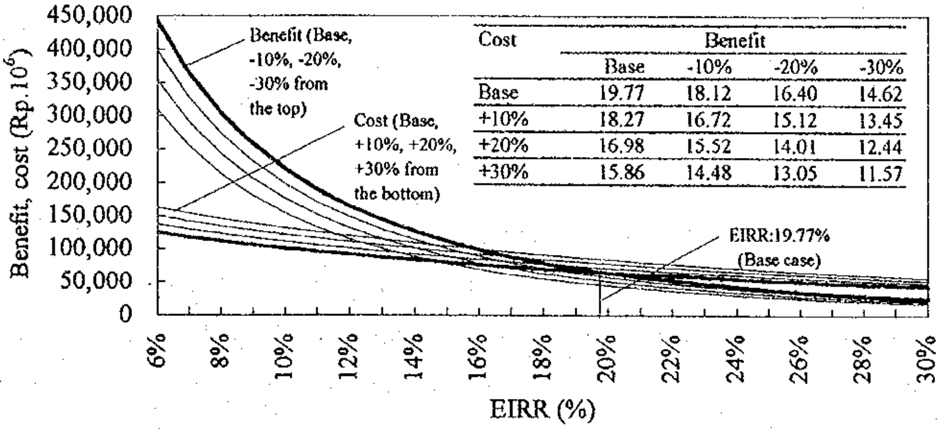
Sensitivity Analysis for Flood Control Works

The economic internal rate of return changes its value depending on the parameters employed for the calculation. Out of these parameters, the construction cost of the Project and its benefit are the most important determinants of the economic analysis.

Therefore, a sensitivity analysis is made for 16 combined cases including base case under the benefit of -10 %, -20 % and -30 %, and the cost of +10, +20 % and +30 % taking into account of fluctuation of the benefit and the cost to be likely to come at present economic situation in Indonesia.

A figure and a table hereunder show the results of sensitivity analysis for economic features.

Sensitivity of EIRR



Cost	Benefit			
	Base	-10%	-20%	-30%
Base	19.77	18.12	16.40	14.62
+10%	18.27	16.72	15.12	13.45
+20%	16.98	15.52	14.01	12.44
+30%	15.86	14.48	13.05	11.57

The EIRR under both the benefit and cost in base case is calculated as 19.77 % as mentioned above. And, nevertheless under the case of the benefit of 30 % decrease and the cost of 20 % increase, the EIRR is calculated as 12.44 % which is higher rate than the used discount rate of 12 % that is suggested by such international financing institutions as the World Bank. Also in the case under the benefit is decreased by 20 % and the cost is increased by 30 %, the EIRR is still keeping more than 12 % as 13.05 % as indicated in the above table. It means that the said flood control works as a component of the Project is economically sound.

Project Justification for Flood Control Works

The EIRR of Flood Control Works indicates 19.8 %, and in the case of (1) 30 % increase in cost with the 20 % decrease in benefit, and/or (2) 20 % increase in cost with 30 % decrease in benefit, it is still keeping more than and/or the same level of the used discount rate as 12 %.

Accordingly, it may say that the said Flood Control Works as a component of the Project has high economic viability, and the above mentioned results indicates that the Flood Control Works surely keeps its viability even if the cost would be increased by 30 % or the benefit would be decreased by 30 %.

6.4.2 Water Resources Development

The cost for the dam construction works should be allocated to each component of the works for project evaluation of each scheme of the functions based on design criteria of water utility. The project evaluation for flood control works mentioned in previous sub-clause has made in accordance with this concept.

As mentioned above, water resources development is one of the main schemes of Jatibarang Multipurpose Dam Construction Works. In this sub-clause, a project evaluation is made within the limit of the said function of water resources development among the several functions of the dam with a reservoir (the Jatibarang Reservoir).

Methodology

The Project Evaluation this time is a review of the evaluation executed in the Feasibility Study made by JICA Study Team in 1993 as mentioned in previous sub-clause, so the way of the project evaluation for water resources development from the viewpoint of economic aspects in this stage of the Project is also made by the same manner applied for the evaluation in the Feasibility Study at that time by using a raw water price reviewing and updating its unit amount based on a result of field survey this time.

According to an information from the Regional State Corporation of Portable Water (PDAM = Perusahaan Daerah Air Minum), the raw water price has become at Rp. 318 per m^3 in 1997 from Rp. 218 per m^3 in 1992 as shown in Table 6.4.8 with an increasing rate of 7.81 % per annum.

Raw water will be able to use from the year 2005/06 just after the completion of the said Jatibarang Multipurpose Dam Construction Works. Therefore, an envisaged amount of raw water price of Rp. 580 per m^3 as of 2005/06 assuming to increase its price with the said past trend of 7.81 % per annum is applied for estimation of the economic benefit on water resources development.

The economic internal rate of return (EIRR) is calculated and used as an index of economic feasibility with the same equation mentioned in 6.4.1 Flood Control Works. The project life is assumed at 50 years after completion of the said works.

Identification of Benefit for Water Resources Development

According to the design criteria of Jatibarang Multipurpose Dam Construction Works, the water volume of 1.46 m^3/sec will be supplied for municipal use after reduction of the maintenance flow of the river. So PDAM might be saved the amount of Rp. 26,700 million per year (= Rp. 580/m^3 x 1.46 m^3/s x 60 seconds x 60 minutes x 24 hours x 365 days).

As mentioned in previous sub-clause , an economic analysis appraises a project under study in terms of a national and/or a regional social economy by comparing and measuring its economic costs and benefits. In other words, economic analysis evaluates a degree of economic impacts on a project under study that would bring about in the national and/or regional social economy.

From the viewpoint mentioned above, the said amount of saving should reflect to a tariff system of potable water which will be connected to customers living in Semarang City, and so it may say an economic benefit for water resources development.

Identification of Economic Cost

In this Project, no any water works, no distribution network, and no house connection works are included in Jatibarang Multipurpose Dam Construction Works with Jatibarang Reservoir. Therefore, only a part of the cost for dam construction works to be allocated for water supply is the cost for economic evaluation for water resources development.

Firstly, gross construction costs for Jatibarang Multipurpose Dam Construction Works are estimated based on unit prices and work volume as mentioned in previous sub-clause, and these gross construction costs includes construction base cost, engineering service cost for supervision, cost for administration, value added tax, cost for compensation, physical contingency and price contingency. In this case, the total dam construction cost is allocated to water resources development based principally on water utility with a rate of 64.44 % according to the design criteria.

(1) Foreign and local currency portions

The calculation method of foreign and local currencys portion is as mentioned in the preceding sub-clause.

(2) Total Economic Cost

The estimated economic cost which is already allocated to water resources development is shown in Table 6.4.9, and summarized as follows:

Economic Cost for Jatibarang Multipurpose Dam Construction Works to be Allocated for Water Resources Development

(Rp.10^6)

Year	FC portion	LC portion	Total
2000/01	1,173	1,278	2,450
2001/02	5,746	7,613	13,359
2002/03	8,001	9,761	17,762
2003/04	18,316	16,454	34,770
2004/05	12,407	7,924	20,331
Total	45,643	43,029	88,672

(3) Cost for Operation/Maintenance and Replacement

Financial costs for operation/maintenance (OM cost) and annualized replacement cost (cost for R) are also estimated by work items at Rp. 570 million for Jatibarang Multipurpose Dam Construction Works. From these financial costs, an economic cost in total is estimated at Rp. 321 million for the works to be allocated for water resources development by the same manner for estimation of the said economic construction cost. These costs for OM and R will be a burden to the Project until the end of the project life of 50 years after completion of the Jatibarang Multipurpose Dam Construction Works. Detail of calculation process is also shown in Table 6.4.9.

Economic Evaluation of Water Resources Development

The evaluation of the water resources development as a scheme of the Project is made by using cash flows of the said costs and benefits as shown in Table 6.4.10. The results are also shown in that table and summarized below.

EIRR (%)	22.14%
B/C	2.08
NPV ($Rp.10^6$)	51,963

In this case, B/C rate is a comparison result of benefit and cost in present value of them, and NPV means net cash balance between benefits and costs also expressed by their present value. For calculation of present value, a discount rate of 12 % is applied as same as that in similar projects in Indonesia.

From the viewpoint of EIRR, the rate is calculated at 22.14 % as indicated above, and the rate of B/C is 2.08. Furthermore, the amount of net cash balance is calculated at Rp. 51,963 million as shown in the above table. The annual average benefit is Rp. 26,700 million as shown in Table 6.4.10.

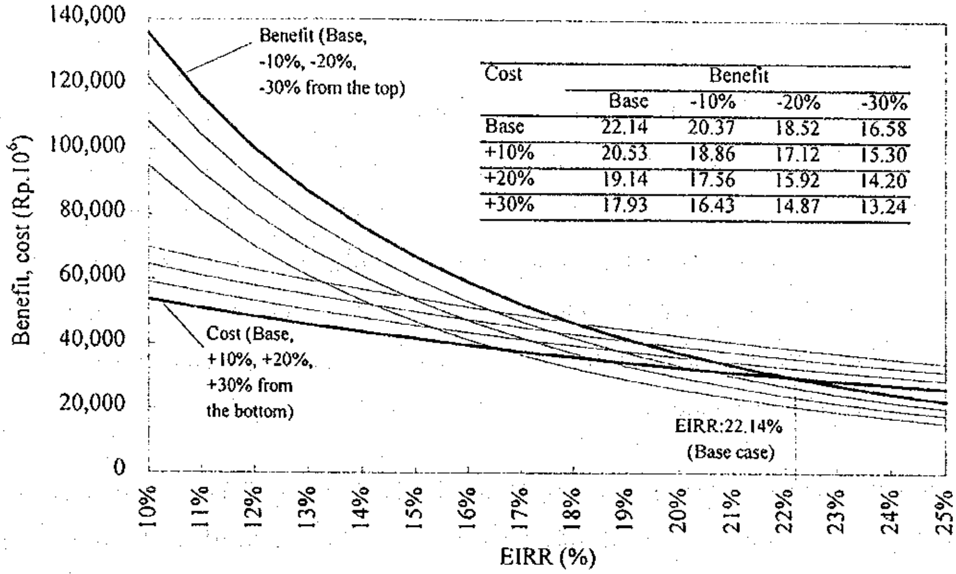
Sensitivity Analysis for Water Resources Development

The economic internal rate of return changes its value depending on the parameters employed for the calculation. Out of these parameters, the construction cost of the Project and its benefit are the most important determinants of the economic analysis.

Therefore, a sensitivity analysis is made for 16 combined cases including base case under the benefit of -10 %, -20 % and -30 %, and the cost of +10, +20 % and +30 % taking into account of fluctuation of the benefit and the cost to be likely to come at present economic situation in Indonesia.

A figure and a table hereunder show the results of sensitivity analysis for economic features.

Sensitivity of EIRR



Cost	Benefit			
	Base	-10%	-20%	-30%
Base	22.14	20.37	18.52	16.58
+10%	20.53	18.86	17.12	15.30
+20%	19.14	17.56	15.92	14.20
+30%	17.93	16.43	14.87	13.24

The EIRR under both the benefit and the cost in base case is calculated as 22.14 % as mentioned above. And, nevertheless under the case of the benefit of 30 % decrease and the cost of 30 % increase, the EIRR is calculated as 13.24 % which is higher rate than the level of 12 %. Usually, a project has viability if it has higher EIRR than the discount rate of 12 % which is suggested by such international financing institutions as the World Bank.

It means that the said water resources development as a scheme of Jatibarang Multipurpose Dam Construction Works is economically sound as a whole.

Project Justification for Water Resources Development

The EIRR of Water Resources Development Plan as a scheme of the Project indicates 22.1 %, and in the case of 30 % increase in cost with 30 % decrease in benefit, the EIRR still keeps the higher rate than the used discount rate of 12 % as 13.2 %.

Accordingly, it may say that the said Water Resources Development Plan as a scheme of Jatibarang Multipurpose Dam Construction Works has quite high economic viability.

6.4.3 Hydropower Generation Works

In this sub-clause, a project evaluation is made within the framework among the several functions of the dam with a reservoir (the Jatibarang Reservoir). However, the hydropower generation works has its own facilities, so the project evaluation should be made in combination of power generation works itself with the dam construction works.

Methodology

The project evaluation this time is a review of the evaluation executed in the Feasibility Study made by JICA Study Team in 1993, so the way of the project evaluation from the viewpoint of economic aspects in this stage of the Project is made by the same manner applied for the evaluation in the Feasibility Study at that time, namely, the method taking a cost for alternative thermal power plant (hypothetical diesel plant was taken as the alternative in the said Feasibility Study) as a benefit, reviewing the unit capacity value (kWh value) and updating based on a price index for fuel.

The economic internal rate of return (EIRR) is calculated and used as an index of economic feasibility with the equation mentioned in 6.4.1 Flood Control Works. The project life is assumed at 50 years after completion of the said works.

Identification of Economic Benefit for Hydropower Generation Works

In the Feasibility Study stage, costs for fuel and lubricant were estimated at US$ 288,500 per annum, and operation and maintenance costs were estimated at US$ 70,200 per annum for alternative thermal power plant. These figures may be applied without any changes. However, annualized construction cost should be corrected from US$ 183,300 per annum in the Feasibility Study to US$ 208,850 per annum by using the discount rate of 12 % with 20 years of economic life of the alternative thermal power plant. Here, the total construction cost was estimated at US$ 1,560,000 in the said Feasibility Study stage. So the annualized capital cost is calculated at the said amount by using a capital recover factor of 0.13388. A formula for estimation of annualized capital cost is as follows:

$$C_T \times \frac{r}{1-(1+r)^{-t}} = C_t$$

Where, C_T = total construction cost (capital cost),

r = discount rate,

t = economic life of facilities in year,

$\frac{r}{1-(1+r)^{-t}}$ = capital recovery factor, and

C_t = annualized capital cost.

So the total annual cost for alternative thermal power plant should be corrected at US$ 567,550 from the amount of US$ 542,000 estimated in the Feasibility Study stage.

On the other hand, annual energy output is 5,790 MWh according to the design criteria of power generation works. Therefore, the unit capacity value may be estimated at US$ 0.098 per kWh (= US$ 567,550/5,790 MWh).

The base cost was estimated at 1992 price level, but the said energy output will be realized in 2005/06. So the unit capacity value which might be converted into economic benefit for hydropower generation works can be calculated at Rp. 575 per kWh in 2005/06 by using the increasing rate of fuel of 5.37 % per annum as shown in Table 6.4.1 with the exchange rate of Rp. 2,971 against US$ 1.00 at the average rate of 1997 as already mentioned.

Namely, the unit capacity value of Rp. 575 per kWh is the envisaged unit economic benefit for power generation works. The annual economic benefit is thus estimated at Rp. 3,328 million (= Rp. 575/kWh x 5,790 MWh).

Identification of Economic Cost

In this case, the total dam construction cost is allocated to power generation works based principally on water utility with a rate of 0.16 % according to the design criteria of the dam construction works.

(1) Foreign and local currency portions

Foreign and local currency portions are calculated as mentioned in 6.4.1 Flood Control Works.

(2) Total Economic Cost

The estimated economic cost is shown in Table 6.4.11, and summarized as follows:

Economic Cost for Hydropower Generation Works

(Rp.10^6)

Year	FC portion	LC portion	Total
2000/01	0	0	0
2001/02	503	140	644
2002/03	3,356	1,319	4,675
2003/04	5,770	2,065	7,835
2004/05	7,542	1,495	9,037
Total	17,171	5,019	22,190

Economic Cost for Jatibarang Multipurpose Dam Construction Works to be Allocated for Hydropower Generation

(Rp.10^6)

Year	FC portion	LC portion	Total
2000/01	3	3	6
2001/02	14	19	33
2002/03	20	24	44
2003/04	45	41	86
2004/05	31	20	51
Total	113	107	220

(3) Cost for Operation/Maintenance and Replacement

Financial costs for operation/maintenance (OM cost) and annualized replacement cost (cost for R) are estimated by work items at Rp. 430 million per annum for hydropower generation works itself and at Rp. 570 million for Jatibarang Multipurpose Dam Construction Works. From these financial costs, an economic cost in total is estimated at Rp. 426 million per annum for hydropower generation works and at Rp. 1 million for Jatibarang Multipurpose Dam Construction Works to be allocated for hydropower generation by the same manner for estimation of the said economic construction cost. These costs for OM and R will be a burden to the Project until the end of the project life of 50 years after completion of the works. Detail of calculation process is also shown in Table 6.4.11.

Economic Evaluation of Hydropower Generation Works

The evaluation of the power generation works as a component of the Project combined with a scheme of Jatibarang Multipurpose Dam Construction Works is made by using cash flows of the said costs and benefits as shown in Table 6.4.12. The results are also shown in that table and summarized below.

EIRR (%)	11.66%
B/C	0.97
NPV (Rp.10^6)	-339

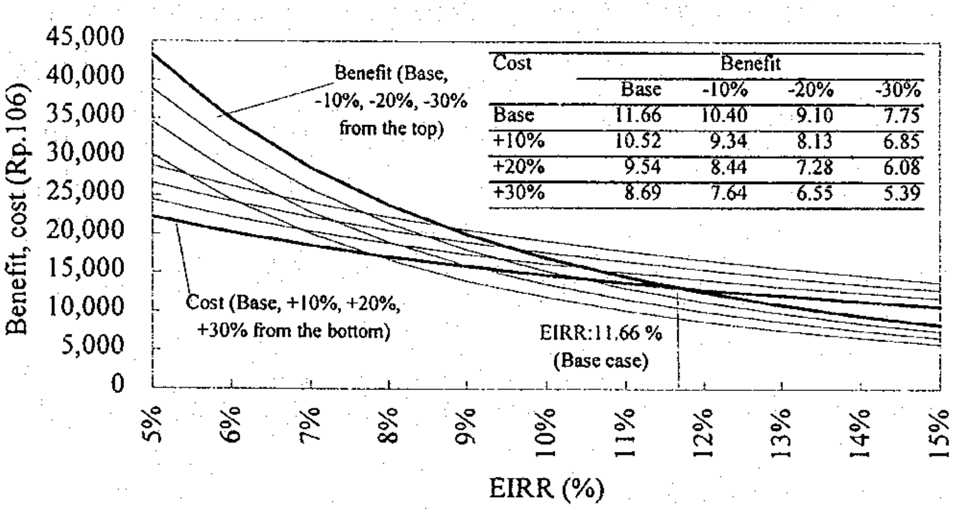
In this case, B/C rate is a comparison result of benefit and cost in present value of them, and NPV means net cash balance between benefits and costs also expressed by their present value. For calculation of present value, a discount rate of 12 % is applied as same as that in similar projects in Indonesia.

From the viewpoint of EIRR, the rate is calculated at 11.7 % as indicated above, and the rate of B/C is 0.97. The amount of net cash balance is calculated at the negative amount of Rp. 339 million as shown in the above table. The annual average benefit is Rp. 3,328 million as shown in Table 6.4.12.

Sensitivity Analysis for the Hydropower Generation Works

A sensitivity analysis is made for 16 combined cases including base case under the benefit of -10 %, -20 % and -30 %, and the cost of +10, +20 % and +30 % taking into account of fluctuation of the benefit and the cost to be likely to come at present economic situation in Indonesia. A figure and a table hereunder show the results of sensitivity analysis for economic features.

Sensitivity of EIRR



Cost	Benefit			
	Base	-10%	-20%	-30%
Base	11.66	10.40	9.10	7.75
+10%	10.52	9.34	8.13	6.85
+20%	9.54	8.44	7.28	6.08
+30%	8.69	7.64	6.55	5.39

The EIRR under both the benefit and the cost in base case is calculated as 11.66 % which is slightly lower than 12 % as mentioned above. And, nevertheless under the case of the benefit of 30 % decrease and the cost of 30 % increase, the EIRR is resulted at 5.39 % which is still higher rate than 5 % that is the minimum rate to hurdle for project formation from the viewpoint of basic human needs suggested by such international financing institutions as the World Bank.

Project Justification for Hydropower Generation Works

The EIRR of Hydropower Generation Works indicates 11.7 %, and in the case of 30 % increase in cost with 30 % decrease in benefit, it is still keeping more than 5 %. Accordingly, it may say that the said Hydropower Generation Works as a component of the Project combined with a scheme of Jatibarang Multipurpose Dam Construction Works has a reliability from the viewpoint of basic human needs.

6.4.4 Jatibarang Multipurpose Dam Construction Works

Identification of Benefit for Jatibarang Multipurpose Dam Construction Works

For project evaluation of the said Jatibarang Multipurpose Dam Construction Works as a whole, the benefits from each scheme should be divided because that those benefits are corresponding to the total cost of each scheme in combination with each component belonging to the dam construction works.

It is assumed that the benefits for dam construction works are corresponding to share rates of the costs allocated to each component as shown in the following table:

Benefits Belonging to the Jatibarang Multipurpose Dam Construction Works

Construction scheme	Component	Economic cost (Rp.10^6)	Share rate of cost (%)	Benefit in total (Rp.10^6)	Benefit to be shared (Rp.10^6)
Flood control works	Own facilities	109,508	69.21		28,209
	Belonging to dam	48,712	30.79		12,550
	Total	158,220	100.00	40,759	40,759
Water resources	Belonging to dam	88,672	100.00	26,700	26,700
Hydropower	Own facilities	22,190	99.02		3,295
	Belonging to dam	220	0.98		33
	Total	22,410	100.00	3,328	3,328
Total benefit belonging to the Jatibarang Multipurpose Dam Construction Works					39,283

Identification of Economic Cost

Economic cost for the dam construction works is estimated by the same way as mentioned in previous clauses for economic evaluation of each component.

The estimated economic cost for the dam construction works is shown in Table 6.4.13, and summarized below:

Economic Cost for Jatibarang Multipurpose Dam Construction Works

(Rp.10^6)

Year	FC portion	LC portion	Total
2000/01	2,148	2,464	4,612
2001/02	10,611	14,160	24,771
2002/03	15,194	18,690	33,884
2003/04	31,666	30,971	62,637
2004/05	21,551	15,537	37,089
Total	81,170	81,823	162,993

Financial costs for operation/maintenance (OM cost) and annualized replacement cost (cost for R) are estimated by work items at Rp. 570 million for Jatibarang Multipurpose Dam construction works. From this financial cost, an economic cost in total is estimated at Rp. 497 million. Detail of calculation process is also shown in Table 6.4.13.

Economic Evaluation of Jatibarang Multipurpose Dam Construction Works

The evaluation of the said Jatibarang Multipurpose Dam Construction Works is made by using cash flows of the said costs and benefits as shown in Table 6.4.14. The results are also shown in that table and summarized below.

EIRR (%)	18.53%
B/C	1.66
NPV (Rp.10^6)	58,938

In this case, B/C rate is a comparison result of benefit and cost in present value of them, and NPV means net cash balance between benefits and costs also expressed by their present value. For calculation of present value, a discount rate of 12 % is applied as same as that in similar projects in Indonesia.

From the viewpoint of EIRR, the rate is calculated at 18.5 % as indicated above, and the rate of B/C is 1.66. Furthermore, the amount of net cash balance is also calculated at Rp. 58,938 million as shown in the above table. The annual average benefit is Rp. 39,283 million as shown in Table 6.4.14.

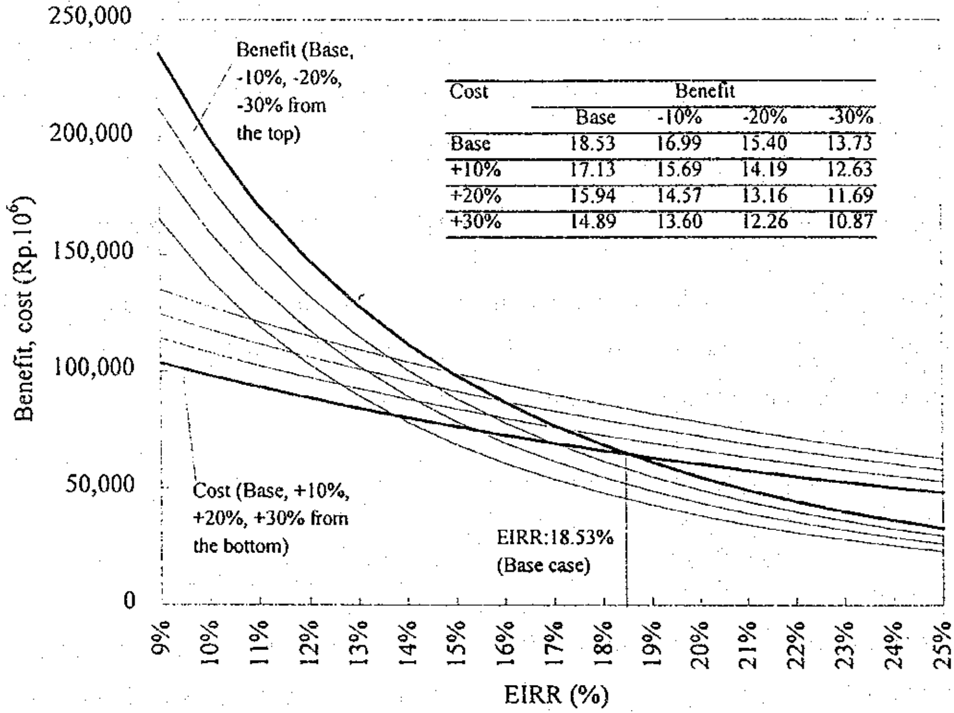
Sensitivity Analysis for Jatibarang Multipurpose Dam Construction Works

The economic internal rate of return changes its value depending on the parameters employed for the calculation. Out of these parameters, the construction cost of the Project and its benefit are the most important determinants of the economic analysis.

Therefore, a sensitivity analysis is made for 16 combined cases including base case under the benefit of -10 %, -20 % and -30 %, and the cost of +10, +20 % and +30 % taking into account of fluctuation of the benefit and the cost to be likely to come at present economic situation in Indonesia.

A figure and a table hereunder show the results of sensitivity analysis for economic features.

Sensitivity of EIRR



Cost	Benefit			
	Base	-10%	-20%	-30%
Base	18.53	16.99	15.40	13.73
+10%	17.13	15.69	14.19	12.63
+20%	15.94	14.57	13.16	11.69
+30%	14.89	13.60	12.26	10.87

The EIRR under both the benefit and the cost in base case is calculated as 18.53 % as mentioned above. And, nevertheless under the case of the benefit of 20 % decrease and the cost of 20 % increase , the EIRR still indicates higher rate than 12 % as 13.16 %. Usually, a project has a viability if it has higher EIRR than the discount rate of 12 % which is suggested by such international financing institutions as the World Bank.

<u>Project Justification for Jatibarang Multipurpose Dam Construction Works</u>

The EIRR of Jatibarang Multipurpose Dam Construction Works indicates 18.5 %, and in the case of (1) 30 % increase in cost with the base case of benefit, (2) base case of the cost with 30 % decrease in benefit and/or (3) 20 % increase in cost with 20 % decrease in benefit, it is still keeping higher rate than 12 %. Accordingly, it may say that Jatibarang Multipurpose Dam Construction Works has quite high economic viability.

6.4.5 Overall Project Evaluation

The Project as a whole consists of Jatibarang Multipurpose Dam Construction Works with components of Flood Control Works, Water Resources Development and Hydropower Generation Works, and Urban Drainage System Improvement Works. In this sub-clause, an overall Project evaluation is made using total costs and total benefits of those components.

Identification of Benefit for Overall Project

Economic benefits for each component are already estimated as mentioned in the previous sub-clauses. For evaluation of the overall Project, all those benefits should be combined. Following table shows a summary of them:

Summary of Benefits of Each Component by Year

(Rp.10^6)

Component	2004/05	2005/06 and after
Flood Control Works	24,941	40,759
Water Resources Development	0	26,700
Hydropower Generation Works	0	3,328
Drainage System Improvement Works	16,101	16,101
Overall Project (total of the above)	41,042	86,888

Identification of Economic Cost for Overall Project

By the same reason of the benefit, all costs should be combined for the evaluation of the overall Project as shown in the following table:

Summary of Economic Cost of Each Component by Year

(Rp.10^6)

Year	Flood Control Works*		Water Resources Development*		Hydropower Generation Works*		Urban Drainage Improvement		Overall Project (total)	
	(FC)	(LC)	(FC)	(LC)	(FC)	(LC)	(FC)	(LC)	(FC)	(LC)
2000/01	3,533	4,124	1,173	1,278	3	3	1,869	4,079	6,578	9,484
2001/02	20,290	22,166	5,746	7,613	517	159	11,920	20,160	38,473	50,098
2002/03	18,602	29,938	8,011	9,761	3,376	1,343	14,624	20,844	44,603	61,886
2003/04	26,901	21,496	18,316	16,454	5,815	2,106	1,789	7,063	52,821	47,119
2004/05	6,816	4,353	12,407	7,924	7,573	1,515	-	-	26,796	13,792

(Note) * : Total amount of its own facilities of each component and cost of dam to be allocated to each component.

Financial costs for operation/maintenance and annualized replacement cost (cost for OMR) are also already estimated and summarized as below:

Summary of Economic OMR Cost

(Rp.10^6)

Component	OMR cost	
	2004/05	2005/06 and after
Flood Control Works*	457	633
Water Resources Development*	0	321
Hydropower Generation Works*	0	427
Drainage System Improvement Works	759	759
Overall Project (total of the above)	1,216	2,140

(Note) * : Total amount of its own facilities of each component and OMR cost of dam to be allocated to each component.

Economic Evaluation of Overall Project

The evaluation of the overall Project is made by using cash flows of the said costs and benefits as shown in Table 6.4.15. The results are also shown in that table and summarized below.

EIRR (%)	18.81%
B/C	1.68
NPV (Rp.10^6)	139,235

In this case, B/C rate is a comparison result of benefit and cost in present value of them, and NPV means net cash balance between benefits and costs also expressed by their present value. For calculation of present value, a discount rate of 12 % is applied as same as that in similar projects in Indonesia.

From the viewpoint of EIRR, the rate is resulted at 18.8 % as indicated above, and the rate of B/C has shown as 1.68. The amount of net cash balance has resulted at Rp.139,235 million as also shown in the above table.

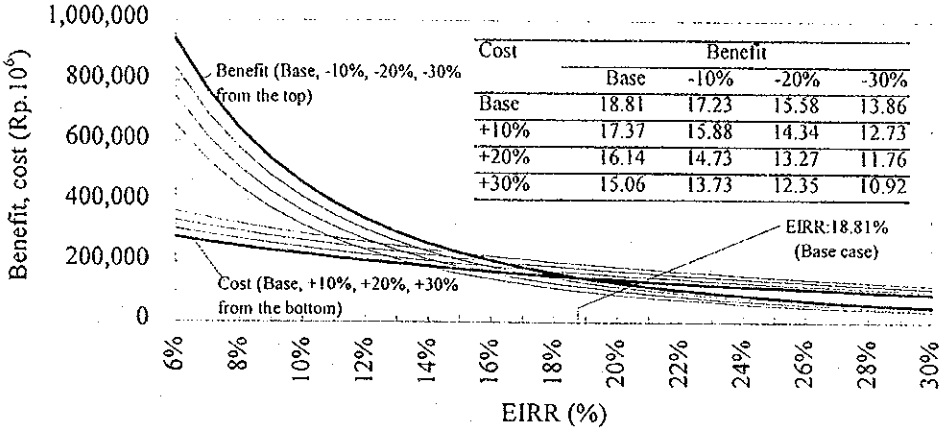
Sensitivity Analysis for Overall Project

The economic internal rate of return changes its value depending on the parameters employed for the calculation. Out of these parameters, the construction cost of the Project and its benefit are the most important determinants of the economic analysis.

Therefore, a sensitivity analysis is made for 16 combined cases including base case under the benefit of -10 %, -20 % and -30 %, and the cost of +10, +20 % and +30 % taking into account of fluctuation of the benefit and the cost to be likely to come at present economic situation in Indonesia.

A figure and a table hereunder show the results of sensitivity analysis for economic features.

Sensitivity of EIRR



Cost	Benefit: Base	Benefit: -10%	Benefit: -20%	Benefit: -30%
Base	18.81	17.23	15.58	13.86
+10%	17.37	15.88	14.34	12.73
+20%	16.14	14.73	13.27	11.76
+30%	15.06	13.73	12.35	10.92

The EIRR under both the benefit and the cost in base case is calculated as 18.81 % as mentioned above. And, nevertheless under the case of the benefit of 20 % decrease and the cost of 20 % increase , the EIRR still indicates higher rates than 12 % as 13.27 %. Usually, a project has a viability if it has higher EIRR than the discount rate of 12 % which is suggested by such international financing institutions as the World Bank.

Project Justification for Overall Project

The EIRR of overall Project indicates 18.8 %, and in the case of (1) 30 % increase in cost with the base case of benefit, (2) base case of the cost with 30 % decrease in benefit and/or (3) 20 % increase in cost with 20 % decrease in benefit, it is still keeping quite higher rate than 12 %. Accordingly, it may be said that overall Project has also quite high economic viability as a whole.