3.4 Domestic and Industrial Water

3.4.1 General

1. Objectives of Study

Objectives of the Study on the domestic and industrial water supply are as follows;

- Objective area ; area within the Brantas river basin plus area outside of the basin but included in the Surabaya Metropolitan area

- to examine the present water supply conditions in the objective area

- to clarify the on-going water supply projects

- to estimate potential water demands in accordance with the Government sectoral policy, etc.

2. Objective Area

In the objective area, the water use conditions are different from area to area. Therefore, the objective area is classified as follows;

- Kota. Surabaya

- Other area within the Surabaya Metropolitan area (hereinafter referred to as " other SMA "). According to definition given by the Surabaya Urban Development Planning Study hereinafter referred to "Urban Study" (Ref. No. MW-06), other SMA consists of Kec. Gresik, Cerme, Menganti and Driyorejo in Kab. Gresik, Kec. Sidoarjo, Wonoayu, Krian and Taman in Kab. Sidoarjo, and Kec. Kamal in Kab. Bangkalan.

- Large urban ; urban area in Kota. Kediri, Blitar, Malang and Mojokerto

- Urban ; urban area in Kab. Trenggalek, Tulungagung , Blitar, Kediri, Malang, Mojokerto, Jombang, Nganjuk and Sidoarerjo.

- Rural ; rural area in the objective area

3. Classification of Water Demands

Water demands are classified as follows;

- Domestic Domestic demand is water to be delivered to people at their place of residence.

- Industrial — Industrial water is that to be used by processing or manufacturing establisments. The port is considered as an industrial user. The industrial water demands are further classified into two; Authorized ones which have licence to take water, and future ones which have no licence at present.

- Commercial — Commercial demands are water to be furnished to non-industrial business.

- Social — Social users consist of public facilities, government officers, schools, hospitals, religious installations and millitary facilities.

In case of demand estimation for Kota such as Surabaya and other SMA, the above classifications are followed. However, for the other areas, the industrial, commercial and social demands are treated under one category as "non-domestic".

3.4.2 Present Water Supply Conditions

1. Access to Drinking Water

According to the 1980 Census, conditions of access to drinking water in the Brantas basin are as follows;

Drinking Water Sources of Household in the Basin

Source of Water	Urban		Rural		Total	
	Nos. (10^3)	%	Nos. (10^3)	%	Nos. (10^3)	%
Pipe	351,141	50.6	61,032	3.3	412,173	16.3
Pump	27,251	3.9	15,683	0.9	42,934	1.7
Well	296,009	42.6	1,345,726	73.6	1,641,735	65.1
Spring	12,247	1.8	312,107	17.1	324,354	12.9
River	1,791	0.3	82,383	4.5	84,174	3.3
Rain	40	0	2,238	0.1	2,278	0.1
Others	3,790	0.5	9,646	0.5	13,436	0.5
Not stated	2,059	0.3	913	0	2,972	0.1
Total	694,328	100.0	1,829,728	100.0	2,524,056	100.0

Note ; Breakdown to Kabupatens and Kotamadya refer Table MW 1 and 2 in ANNEX MW

Source: 1980 Census

As shown in the above, majority of the population in the basin still relies on the non-piped water which is easily subject to contamination.

2. Existing Water Supply Capacity in Municipalities

According to the address given by Proyek Air Bersih, East Java, the existing water supply capacities in the municipalities in the basin are reported as follows;

Kotamadya	Surabaya	3,200 ℓ/sec
	Malang	763 ℓ/sec
	Mojokerto	60 ℓ/sec
	Kediri	100 ℓ/sec
	Blitar	60 ℓ/sec
Kabupaten	Sidoarjo	80 ℓ/sec
	Jombang	36 ℓ/sec
	Nganjuk	40 ℓ/sec

The existing facilities of Surabaya and other municipalities are as shown in Table MW-3 in ANNEX MW.

3. Production and Supply

In PDAM Surabaya, the water production and supply in 1982 are reported as 89.8 million m^3 and 60.5 million m^3 respectively as shown in Table 3.4.1. Of the total supply, the category-wise supply is reported as follows (see Table 3.4.2)

Domestic	56.6 %
Industry	6.4 %
Commercial	11.3 %
Government	18.9 %
Social	1.2 %
Other enterprice	5.5 %

Although data of other municipalities are not available,majority of supply in other municipalities is considered to be used for domestic purpose.

4. Per Capita Supply

In case of PDAM Surabaya, the average supply per residential connection is 1.16 m^3 / day as shown in Table 3.4.2. If one connection is used by ten persons, the per capita supply to the residential connection becomes 116 ℓ/day. However, if the total domestic supply is divided by the total population in Kota Surabaya, the per capita supply becomes 49 ℓ/day. Similar story is applicable to all other PDAMs; per capita supply to residential connection is not so low, but the per capita supply divided by total population is still low.

5. Water Quality

Among the water sources, the groundwater and spring water developed and under-development have good qualities as shown in some samples presented in Table MW-4. On the contrary the surface water is generally of high turbidity, and sometimes highly contaminated. The qualities of raw water in Surabaya river at the Ngagel plant intake are reported as follows;

TYPICAL QUALITY OF KALI SURABAYA AT NGAGEL INTAKE

(Data Source : PDAM 1982/83)

		Dry Season	Wet Season
Turbidity	NTU	4.5 - 35	5 - 500
pH		7.6 - 7.8	7.5 - 7.7
Alkalinity as $CaCO_3$	mg/ℓ	160 - 185	130 - 200
Total hardness as $CaCO_3$	mg/ℓ	130 - 160	120 - 150
Dissolved oxygen	mg/ℓ	0.8 - 6.0	0.7 - 6.0
Permanganate Value	mg/ℓ O_2	5.8 - 14.0	8 - 26
COD/PV ratio approx.		4	4 - 6
Suspended solids	mg/ℓ	40 - 160	40 - 6,000

(Ref. table MW-5)

From the above figures, it can be said that the water quality in Surabaya river is highly critical. This bad qualities of raw water may be attributable to the direct disposal of the municipal and industrial waste water into the river and to the insufficient quantity of river flow to dilute.

In order to keep the quality of the surface water in the level of permissible, it will be necessary to control the industrial waste water according to the guideline of quality standard of industrial waste water (Ref. Table MW-6) and to introduce treatment of the municipal waste water. Even if these measures are realized, the natural high turbidity of the surface water in Brantas river will still need costly treatment.

6. Industrial Water Intake

Besides the industrial use of the water supplied by PDAMs, there are industrial water intakes from the Brantas river and its distributaries. Intakes are permitted by the Irrigation Service through issuance of licence. The amount of industrial water licensed and under processing for license is as follows;

Areal/Canal	Licenced	Under Processing
Kediri	0.7 m^3/s	
Nganjuk	0.6	
Mojokerto	1.23	
Sidoarjo/Mangetan	0.638	0.5 m^3/s
Porong	1.291	
Surabaya		
K. Surabaya	1.092	
K. Mas	0.444	
Jeblokan	0.075	
Total	6.07 m^3/s	0.5 m^3/s

Note ; Detailed breakdown refer to Table MW-7 in ANNEX MW.

According to the Irrigation Service, there is no schedule to increase the licensed water from the present level, since all the available run-off is already allocated.

According to Surabaya Water Use Study, the net consumption of some industries taking water from the Brantas or Surabaya rivers is reported to be 26% of the licensed amount. Details are shown in Table MW-7.

According to the factory survey of the Urban Development Planning Study, which covers small scale industries as well as large scale industries, about 28% of the answers pointed out lack of sufficient water supply as serious problem in their operation and about 25% as slight problem.

3.4.3 On-going and Proposed Water Supply Project

1. Surabaya Metropolitan Area (Ref. Fig. 3.4.1)

After aquiring the allocation of 3 m^3/sec on the purified water basis from the Surabaya river for the Ngagel treatment plant near the

Jagir dam, PDAM Surabaya had no more water sources for its expansion. Then, efforts to find out other water sources has been continued.

In 1978, development of the Umbulan spring locating about 10 km south of Pasuruan was firstly studied in a supply capacity of 3 m^3/sec. The first stage of the Umbulan spring development was planned to include 61 km of 1,800 mm diameter transmission main pipeline to carry 2.1 m^3/sec by gravity. The second stage was planned to add booster pump 4 m^3/sec. The capital cost of the first stage was estimated at Rp. 38.7 billion (US$ 93 million), and that of the second stage at Rp. 14.2 billion (US$ 34 million).

The Umbulan spring development was once again studied on feasibility study level in 1982 for the supply capacity of 3 m^3/sec. The capital cost was estimated at Rp. 70 billion at the 1981 price level.

Detailed design of this project is under way for the supply capacity of 2.2 m^3/sec (1.4 m^3/sec by about 1990 and 0.8 m^3/sec by about 1991 or 1992). The capital cost of the project is estimated at Rp. 110 billion at the 1984 price level. Implementation of the Project is not yet determined.

In 1983, the Karangpilang treatment works project was studied. The project site is located along the Surabaya river 25 km south-west from Surabaya. This project is planned to have three stages. The planned supply capacity is 1 m^3/sec in the first stage, and 2 m^3/sec each in the second and third stage. The water source of the first stage is water to be saved by rehabilitation of the irrigation canals in the Delta area. The capital cost of the first stage is estimated at Rp. 18.7 billion at the 1983 price level, including the cost for canal rehabilitation. Water source for the second and third stages is not yet clearly defined. As one of alternatives, water to be released from the on-going Wonorejo reservoir in the Ngrowo river basin to the Brantas river is considered.

3.4.4 Future Water Demand

1. Basic Concept of Water Supply Development

Clean water is the basic need of human life. In view of the present low level of piped water supply, increase of clean water supply is of urgent necessity, especially in the urbanized areas.

The basin has the second largest city in Indonesia; Surabaya. The Surabaya metropolitan Area (here in after referred to as "SMA") is well equipped with transportation, communication, commercial facilities, and has been playing as the economic center of the eastern half of the Indonesia. In future, SMA will be one of the most important growth poles of Indonesia. In this context, the water supply to SMA should be in line with the growth of SMA.

The target of REPELITA IV for the clean supply is to meet the basic demand of 75% of urban population with a daily average of 60 liters per person for small town and 90 liters for medium town by the end of the 1980's. In planning of water supply development, this policy is fully taken into account.

In this study, the minimum potential water demand to cover all the expected needs are taken into account from the viewpoint of water resources allocation.

2. Potential Domestic Water Demand

Domestic water demand is calculated by multiplying the population to be served by the unit per capita consumption rate.

Population to be served

In the Brantas basin, about 12 million population lived in 1980. The population in the basin is considered to increase with some growth rates. GOI is now promoting its family plan with strong emphasis and transmigration plan, from the Java island to the other island. These two policies will give influences to the population in the Basin. The Statistical Year Book present a populating projection as shown in Table 3.4.5. These figures present an expectation of decrease in the annual population growth rates. These figures are almost similar to those estimated by the Bereau of Census, USA (Ref. Table MW-9) which are based on the medium estimate of the birth and mortality rated. Referring to these figures, the birth and mortality rates are extended beyond the year 2000 up to the year 2020 as shown in Fig. 3.4.3 and the annual population growth rate in the same period in the entire Indonesia is estimated.

According to the 1980 census, the population growth rates in the Brantas river basin in the period from 1971 to 1980 are already lower than the national average. It is assumed that the same tendency will continue in future. Then, the future population growth rates in the basin is assumed to be lowered in proportion to the decrease in the population growth rate in the entire Indonesia, as shown in Table 3.4.6. Applying the assumed growth rates, the total population in the basin is calculated as shown in Table 3.4.7.

As for the population growth in the SMA, the population projection prepared by the Urban Study is taken and extended up to year 2020 using same growth rate between 1990 and 2000. This arrangement causes large social population increase in the SMA. It is assumed that such social population increase will come from the rural area in the East Java Province.

In the Kotamadya other than SMA, urbanization is considered to continue. The urban population growth rate is assumed at 2.5% per annum.

The rural population is calculated at the balance between the projected Kabupaten population and the urban population and population to migrate to SMA. Results are as shown below;

Unit : x 10^3

	1980	1990	2000	2010	2020
SMA	2,867.5	4,187.0	6,119.0	8,938.1	13,056.0
(Surabaya)	(2,017.5)	(2,861.6)	(4,163.9)	(6,046.1)	(8,779.1)
(Other SMA)	(850.0)	(1,325.4)	(1,955.1)	(2,892.0)	(4,276.9)
Urban	1,608.4	2,058.8	2,635.4	3,737.6	4,318.4
Rural	7,870.5	8,479.2	8,798.1	8,636.9	7,671.6
Total	12,346.3	14,250.0	17,552.5	20,948.6	25,046.0

Details are shown in Table 3.4.7

3. Other Urban Area

CIPTA KARYA is now undetaking the East Java Water Supply project with the assistance of IBRD. The project takes up 62 municipalities in the Brantas basin, which are classified into the following two groups according to their population size;

Population	Project Name
Over 20,000	Basic Need Approach (BNA)
3,000 - 20,000	Ibu-Kota Kecamatan (IKK)

Each of them is divided into two stages. The first stage is under implementation and scheduled to complete in 1985. The second stage is scheduled to be completed by 1990.

Stage	No.of Municipality	Population Served	Total Capacity	Per Capita
1st BNA	8	313,300	366 ℓ/s	100 ℓ/d
1st IKK	23	115,200	80 ℓ/s	60 ℓ/d
2nd BNA	4	134,050	153.7 ℓ/s	100 ℓ/d
2nd IKK	27	179,030	122.5 ℓ/s	60 ℓ/d

Towns to be covered by these projects are as follows;

BNA Project

First Stage

Singosari, Batu, Kepanjen, Tulungagung, Ngunut, Mojosari, Sidoardjo, Krian.

Second Stage

Wlingi, Trenggalek, Pare, Kertosono

IKK Project

First Stage

Kab. Malang (9 IKKs), Kab. Mojokerto (6 IKKs)
Kab. Sidoarjo (6 IKKs), Tulungagung (4 IKKs)

Second Stage

Kab. Nganjuk (7 IKKs), Kab. Kediri (7 IKKs), Kab. Blitar
Kab. Blitar (6 IKKs), Kab. Trenggalek (7 IKKs)

Location of the above is as shown in Fig. 3.4.2 and Details are shown in Table MW-8.

The water source of these water supply schemes are all spring or well except Pule in Kab. Trenggalek which will use river water. The design of facilities are standardized as shown in Table 3.4.3. Construction costs of theseprojects are as shown in Table 3.4.4.

Per Capita Demand

There are some different figures about the per capita domestic water demand. Some of them are presented in Note MW. According to the address given by the East Java Water Works, the following figures are also presented;

Population	Per Capita
Over 1,000,000	120 ℓ/d
500,000 - 1,000,000	100 ℓ/d
100,000 - 500,000	90 ℓ/d
20,000 - 100,000	60 ℓ/d
3,000 - 20,000	45 ℓ/d

International comparison is as shown in Table 3.4.8.

From the viewpoint of people's welfare, larger per capita will be preferable. However, it is considered that there will be some constraints coming from cost of water, eq. payment capacity.

Approach from Capacity to Pay

There are very scarce data on the willingness to pay the water tariff. Some reports suggest a rate of 2% of the monthly income as the maximum. Taking this figure, the family size of 4.9 person, and the water tariff of Rp. 30/m^3 in 1975, when data on the income distribution is available as shown in Table 3.4.9, the households which can be connected by pipes water are estimated as follows.

Unit Water Demand ℓ / c / d	Household Demand m^3 / month	Water Tariff Rp. / month	Required Min. Income Rp. / month
100	14.70	441	22,050
150	22.05	661.5	33,075
200	29.4	882	44,100
220	32.4	970.2	48,510

From Table 3.4.9, the number of households above the minimum income requirement is calculated by interpolation.

Unit Water Demand	No. of Household above Requirement					
	1980		1990		2000	
ℓ / c / d	10^3	%	10^3	%	10^3	%
100	421.0	70.5	694.7	81.4	1,109.7	89.3
150	308.1	51.6	561.7	65.8	972.4	78.2
200	223.4	37.4	448.1	52.5	835.3	67.2
220	195.8	32.8	409.2	47.9	781.5	62.8

Assuming the per capita consumption of 30 ℓ/d for the consumers not connected with household convection, the average per capita conssumption is estimeted as follows;

Unit Water Demand of Household Connection ℓ/c/d	Average Per Capita Demand		
	1980	1990	2000
100	79.4	87.0	92.5
150	91.9	109.0	123.8
200	93.6	119.3	144.2
220	92.3	121.0	149.3

The above figures are plotted on Fig. 3.4.4 and the lines are extended up to the year 2020. From this figure, per capita demands are set as follows;

	1990 ℓ/c/d	2000 ℓ/c/d	2010 ℓ/c/d	2020 ℓ/c/d
SMA	119.3	149.3	167.5	190.0
Large urban	109.0	123.0	137.5	150.0
Urban	87.0	92.0	96.0	100.0
Rural	37.5	45.0	52.5	60.0

Domestic Water Demand

By multiplying the per capita demand with the population, potential domestic water demand is calculated as shown below.

Unit m^3/day

	Year				
	1985	1990	2000	2010	2020
SMA	370,755	499,509	913,567	1,497,132	2,480,640
(Surabaya)	(257,159)	(341,388)	(621,670)	(1,012,722)	(1,668,029)
(Other SMA)	(113,596)	(158,120)	(291,897)	(484,410)	(812,611)
Large Urban	89,496	109,828	158,646	227,013	317,010
Urban	80,102	94,120	127,834	170,688	227,660
Rural	277,116	308,971	395,903	453,477	472,967
Total	817,469	1,012,428	1,595,950	2,348,310	3,498,277

Details are snown in Table MW-12

4. Non-Domestic Water

(1) SMA

Industrial Water

There are various methods to estimate industrial water supply demand, eg. per employee, and per area of factory. Samples are shown in Table MW-10. Since data on the factory areas at present and in future are not available, per employee estimate is taken in this study. From the industrial statistics of East Java, composition of the employees among the kinds of industry is assumed as shown in Table 3.4.10. By multiplying the number of employees by the unit water demand, the water demand under the assumed composition is estimated as shown in Table 3.4.10.
By dividing the total demand by the total employees, the average unit water demand per employee is calculated at 1.61 m^3/capita/day. There is a report that the ratio to the net consumption to the actual abstruction is about 30% (Ref. Table MW-11). Applying this ratio, the average net consumption of industrial water is estimated at about 0.5 m^3/employee/day.

The Urban Study assumed the average industrial water demand of 500 l/employee/day. This figure is well coinciding to the above estimate. Then, as the average figure, 500 l/employee/day is taken. According to the Study, the present level of the industrial water consumption in SMA is as low as less than 100 l/employee/day, and the Study assumed an intermediate stage of industrial water consumption of 200 l/employee/day for the year 1990. This assumption is taken.

The growth rate of the number of industrial sector employee is taken from the Urban Study, eg. 6.57%/a. in 1980-1990 and 6.62%/a. in 1990 - 2000. Beyond the year 2000, the annual growth rate of 6.62% is applied.

In the Surabaya Urban Study, the port water is estimated in relation to the cargo volume through the port. Same approach is taken. The annual cargo volume growth rate beyond the year 2000 is assumed same as that between 1990 and 2000, and the unit water demand is assumed to gradually decrease from 0.143 ℓ/1000t in 2000 to 0.103 ℓ/1000t in 2020.

Commercial Water

The Urban Study assumed that the commercial water demand would grow in proportion to the output of the commercial sector and the unit water demand would be 68.8 l per Rp. one million of the tertiary sector's GRDP measured in the 1975 constant price level. Same approach is taken in this study, and the growth rate of the tertiary sector's GRDP beyond the year 2000 is assumed as 6.55%/a.

Social Water

The social water is considered to grow in proportion to the population growth. The unit water demand of 8.1 l/capita/day is taken.

The non-domestic water demand in SMA is estimated as shown below.

Unit : m^3/s

Water Use	Year			
	1990	2000	2010	2020
Industrial				
Authorized	4.04	4.04	4.04	4.04
Unauthorized	0.71	3.38	6.18	11.27
Port	0.03	0.04	0.08	0.16
Commercial	0.44	0.83	1.60	3.00
Social	0.39	0.57	0.84	1.23
Total	5.61	8.86	12.74	19.70

Details are shown in Table MW-14.

(2) Non-Domestic Water Demand in Areas other than SMA

According to the design standards of the BNA and IKK projects the non-domestic water demand in the areas other than SMA is assumed as follows;

Large urban : 30% of domestic demand
Urban : 20% of domestic demand
Rural : 5% of domestic demand

The non-domestic water demand in other areas is estimated as shown below;

Unit : m^3/day

	1985	1990	2000	2010	2020
Large urban	26,849	32,948	47,594	68,104	95,105
Urban	16,019	18,884	25,572	34,139	45,532
Rural	13,856	15,889	19,795	22,676	23,650
Total	56,724	67,721	92,961	124,919	164,287

Details are shown in Table MW-12.

5. Allowances and Losses

In the water supply system, some allowances and losses are unavoidable. Then, the following allowances and losses are taken into account;

Distribution (unaccounted for)
SMA and urban area 20%
Rural 5%
Purification loss for river water 8%

As mentioned in the water quality, the spring water and ground water need only chlorination due to their good quality, in general. Therefore, no purification loss is considered.

6. Total Potential Water Demand

By summing up the above estimates, the total potential domestic and industrial water demands are calculated as shown below:

Total Potential Domestic and Industrial Water Demand

Unit : m^3/s

	1990	2000	2010	2020
SMA	13.01	22.06	34.04	54.70
Large urban	1.98	2.87	4.10	5.72
Urban	1.57	2.13	2.85	3.79
Rural	4.45	5.53	6.34	6.61
T o t a l	21.01	32.59	47.33	70.82

7. Net incremental requirement to water resources

In Water Allocation Study, water balance is examined at Jabon - Perning in the downstream of the Brantas river. For use for the study, it is necessary to estimate the net potential water demand at the Jabon - Perning site, with the following adjustment

- excluding the Ngrowo basin
- excluding the water use included in the present hydrological cycle

The majority of the Ngrowo river basin is now separated from the Brantas basin. Therefore, the water demands in Kab. Tulungagung and Kab. Trenggalek in the Ngrowo basin is excluded from the total potential water demand in the Brantas basin.

At present, some amount of water is used in the rural areas as well as the urban areas. In case of the urban areas, the water supplied through the supply systems is assumed to be net loss of the water in the water resources and it is considered that such losses are already included in the present hydrological cycle. In the areas which are not served by the water supply system, people uses water from well and others, and consume also some amount of water. Such consumption is also considered to be included in the present hydrological cycle. Therefore, of the potential water demand estimated in the previous sub-section, the net incremental requirement of the water resources for the domestic and industrial water supply is considered as the potential demand minus the present net consumption included in the present hydrological cycle.

In the basin, only 17% of the basin population receives piped-water supply, and rest of the population is taking water from wells and others. Net consumption of water by such people may be drinking water and some cooking water. Then, the per capita net consumption is assumed as 20 l/capita/day.

In the urban areas where water supply systems exist, the net consumption of water is assumed as 70% of the supply capacity(daily average supply level).

Based on the above considerations the net consumption of water presently included in the hydrological cycle in the Brantas river basin is calculated as shown below.

Water supply system 0.741 ℓ/s
Well and other sources 1.991 ℓ/s
(Details are shown in Table MW-13)

Through the above adjustment, the potential domestic and industrial water demand for examination of water balance is estimated as follows;

Unit m^3/s

Demand	Source	1985	1990	2000	2010	2020
SMA Domestic	Surface	5.57	5.80	11.56	20.31	35.06
	Spring	-	1.70	2.20	2.20	2.20
SMA Social	Surface	0.36	0.43	0.62	0.91	1.33
SMA Commercial	Surface	0.35	0.48	0.90	1.73	3.24
Other Domestic	Ground.w	2.54	3.45	5.41	7.52	9.60
Other Non-domestic	Ground	0.70	0.84	1.16	1.57	2.08
T o t a l		9.52	12.70	21.85	34.24	53.51
Industrial						
SMA Licensed	Surface	3.57	3.57	3.57	3.57	3.57
Other Licensed	Surface	2.53	2.53	2.53	2.53	2.53
S u b - total		6.10	6.10	6.10	6.10	6.10
Future	Surface	1.58	1.81	4.70	7.77	13.36
T o t a l		7.68	7.91	10.80	13.87	19.46

8. Cost Estimated for Water supply Development

Costs of water supply development are largely varying case by case according to the conditions. Sample of the construction costs is as shown in Table MW-15. From this table, the following unit construction costs including source works, transmission, treatment and distribution are taken for rough cost estimate of water supply development;

River water with treatment Rp. 629 x $10^3/m^3/day$
Ground water for urban area Rp. 405 x $10^3/m^3/day$
Ground water for rural area Rp. 545 x $10^3/m^3/day$

As for the industrial water, it is considered that costs of source works and transmission works shall, in principle, be borne by the in - dustry itself. Therefore, the costs for industrial water supply development is excluded from the estimate of public investment requirement for the water supply sector.

The estimated costs are as shown in Table MW-16 and summarized as shown below.

Unit Rp. x 10^9

From	1985	1990	1995	2000	2005	2010	2015
To	1990	1995	2000	2005	2010	2015	2020
	P e r i o d e						
Supply from river water	97.2	127.1	185.4	194.9	245.6	329.5	412.8
Supply from ground water	51.3	50.3	51.3	56.0	50.1	50.6	52.3
Total in 5 years	148.5	177.4	236.7	250.9	295.7	380.1	465.1

3.5 Flood Control Plan

3.5.1 General

The present design discharge distribution as shown on Fig. 3.2.9 was determined under the Brantas river basin development plan (1973 - MASTER PLAN). The flood control works have been implemented so far with the said discharge distribution. Some flood control works were completed up to now and some are under implementation. Owing to the effects of such flood control works, the drainage condition in the basin has been year by year improved.

More than 10 years have been passed since the formulation of the above-mentioned master plan. For the past decade, the basin has been gradually and widely developed in various sectors, especially in the Surabaya area. Urbanization in the basin is one of the remarkable changes during the last 10 years. Such development has increased properties in the basin, and potential flood damage in the basin has become large year by year.

Aside from the above, it was reported that big floods occurred recurrently for the past 10 years in the Brantas river basin. Among the sub-basins, the Widas river basin has habitually and extensively suffered from severe inundation due to lack of effective flood control and drainage countermeasures.

From such situation in the basin, the basin-wide flood control plan is studied in the following section. The present condition and problem of the rivers in the basin are firstly investigated and reviewed, and subsequently, the flood control plan of the Brantas river is studied in relation with the formulation of a comprehensive flood control plan in the Widas river basin.

3.5.2 Present Conditions and Problems

The previous study reports and data related to the flood control works are collected at BRBDEO and other provincial agencies concerned. Previous flood control studies in the Brantas river basin are reviewed to clarify the background, basic principle, present status of the flood control projects. Supplemental data is presented in the ANNEXES RC

1. River system

The total catchment area of the Brantas basin is about 12,000 km^2. The main tributaries of the Brantas are the Lesti, Konto, Ngrowo and Widas rivers. The Ngrowo river except its small area is independent of the present Brantas drainage system. The Porong river is the main drainage of the lowermost Brantas into the Madura strait. The Surabaya river is a distributary of the Brantas, which is independent of the Brantas river in view of the flood drainage. The river system and its profile of the Brantas river are shown in Figs. 2.2.1 and 2.2.2, respectively.

The catchment area of the main rivers is as follows.

K. Lesti	625 km^2
K. Konto	687 km^2
K. Widas	1,539 km^2
K. Ngrowo residual basin	177 km^2
K. Brantas and other remaining basins	6,718 km^2
Sub total	9,746 km^2
K. Ngrowo	1,423 km^2
K. Surabaya	631 km^2
T o t a l	11,800 km^2

The total length of the main Brantas from its source to the Madura strait is approximately 340 km.

2. Present Condition of Rivers

(1) Brantas river mainstream

(a) Upper Brantas river

Total catchment area of the Brantas at the Karangkates dam site is about 2,050 km^2 including about 625 km^2 of the Lesti river basin. The average river slope is steeper than 1/200 in the upper Brantas. The river system of the upper Brantas basin is shown in Fig. RC-1.

In this reach, flood control problem seems to be quite small, since the river runs in the valley or hilly areas. Also the sediment problem is not so severe comparing with those of the Lesti river and Mt. Kelud basins due to rich vegetation in the mountainous area. Downstream from the Lesti river confluence, the Sengguruh dam construction is ongoing for the purpose of hydropower generation.

(b) Karangkates dam to Lodoyo dam

In this reach, the main tributaries are right bank tributaires originating in the southern slopes of Mt. Butak and of Mt. Kelud, and there exist three dams of the Karangkates, Wlingi and Lodoyo. The total catchment area of the Brantas at the Lodoyo dam site is about 3,014 km^2.

This reach is subject to habitual sedimentation caused by the eruption of Mt. Kelud. Among others, the Lekso river and Putih river are remarkable ones.

In the upper reaches of the above two rivers, large-scale sand pockets and Sabo dams were constructed in the past by Mt. Kelud project in order to trap the sediment materials produced by the past eruption as shown in Fig. RC-2 and 3 in ANNEX RC Also their riparian areas are habiutally suffering from inundation due to insufficient river channel capacity. For this reason, river channel improvement including the sediment control is being considered in the right bank tributaries and its plan is being studied by the said project. In formulation and execution of the above plan, close cooperation between the both projects of Mt. Kelud and BRBDEO is required to maximize the effect accrueing from the both projects.

(c) Lodoyo dam to Ngrowo river confluence

In this reach, the main tributaries are right bank tributaries of the Abab , Sentuk rivers and a distributary of the Jatiplen, and the Ngrowo river a left bank tributary having a catchment area of about 177 km^2.

Also this reach suffers from sedimentation caused by the past eruption of Mt. Kelud and by the sediment load coming from the upper reaches of the Brantas river. At the Ngrowo river confluence, the riparian area in the right bank is subject to frequent inundation due to insufficient channel capacity and therefore, some rehabilitation works of river channel is being considered by Mt. Kelud project as aforementioned (Ref. Fig. RC-2).

The total catchment area of the Brantas river at its confluence with the Ngrowo river is about 3,600 km^2. The average river slope in this reach is 1/600 to 1/1,000.

(d) Ngrowo river confluence to Lengkong dam

After joining the Ngrowo river, the Brantas river flows north-east towards Ploso. The main right bank tributaries are the Termas, Lanang, Kresek, Sukorejo, Dermo, Serinjing and Konto which originate in Mt. Kelud and Mt. Arjuno complex. On the other hand, the Brantas river is joined by the Widas river a large left bank tributary originating in Mt. Wilis.

In the lower end of this reach, the Brantas river branches the Surabaya river. The flood flow is not diverted to the Surabaya river. The Surabaya river contributes to supply low flow to downstream and to local drainage in its basin.

Downstream from several kilometers upstream of Kediri, the Brantas river has a diking system at the both banks. The total length of the diking system in this reach is about 90 km. The average river width between both dikes is about 200 m and the average top width of low water channel, about 150 m. The average river bed slope is 1/2,000 to 1/1,500. The average flood carrying capacity below the design flood water level (river channel which will be improved by stage II) ranges from 1,700 m^3/s to 500 m^3/s. The longitudinal profile and flow capacity of this stretch are shown in Figs. RC-4 and 5 in ANNEX RC

In the upper unleveed reach, the riparian areas of the Brantas river function as a natural flood retention areas and in the leveed reach, the riparian areas of Brantas river, especially in the both banks near the Widas river confluence, are subject to frequent inundation. In addition, the Brantas river in this reach is severely subject to sedimentation. This sedimentation is resulted mainly from the sand materials produced by the past eruption of Mt. Kelud.

In order to prevent the riparian area in this reach from flooding and sedimentation, the river improvement works of the Brantas mainstream are undertaken by BRBDEO and the debris control works including the river improvement in Mt. Kelud basin is undertaken by Mt. Kelud project.

Owing to the efforts of such works, the inundation codition has been improved to a certain extent. However, there still remains inundation in the riparian areas densely populated though smaller in area and shorter in duration than those of the past. This inundation results mainly from insufficient flood control facilities and from high flood level with longer duration in the mainstream. To solve such constrains and raise the protection level against flood, a basinwide flood control and drainage works including the debris control are required.

The total catchment area of the Brantas river at the Lengkong dam site is about 8,650 km^2 and the total length of the mainstream in this reach is about 113 km.

(e) Lengkong dam to river mouth

The Brantas river main stream in this reach is locally called as the Porong river. The Porong river flows through a very flat plain lower than the elevation of 25 m, SHVP and finally pours into the Madura strait.

In this reach, the main tributaries are the Sadar river and Kambing river. The total catchment area of the Brantas river at Porong bridge is about 9,130 km^2 and the total river length is about 48 km. The average flood carrying capacity below the existing design flood water level is 1,500 to 2,000 m^3/s. The longitudinal profile and flow capacity of this stretch are shown in Figs. RC-4 and 5.

The Porong river has a diking system at its both banks. The average river width between the both dikes is about 200 m and the low water channel width, about 100 m. Downstream from the Kambing river confluence at near Porong bridge, there exists a narrow point of the river width about 140 m. In this reach, riparian areas of the Sadar river, a right bank tributary of the Porong river, are subject to inundation due to insufficient channel capacity, and some countermeasures for drainage will be needed.

(2) Major Tributaries

(a) Lesti River

The Lesti river has its source on the western slope of Mt. Semeru, from where it flows south-east through the mountaineous area and joins a left bank tributary of the Genteng river. After joining the Genteng river, it flows towards east and finally joins to the Brantas river mainstream at about 14 km upstream of the Karangkates dam site. The total catchment area of the Lesti river is about 625 km^2 and the total river length is about 55 km.

In this river basin, the flood control problem seems to be small, however, the main stream of the Lesti river is facing to a sedimentation problem caused mainly by surface erosion. From such situation, some countermeasures to the surface erosion will be needed to protect the downstream reaches of the Lesti river and mainstream of the Brantas river.

(b) Konto River

The Konto river has its source on the western slope of Mt. Arjuno. After flowing down in the mountainous area, it reaches to the Selorejo dam which is the multipurpose dam for irrigation, hydroelectric power generation, and flood control. Downstream from the Selorejo dam site, the Konto river is joined by a left tributary originating on Mt. Kelud, and flows down towards north-west and finally joins to the Brantas mainstream. The total catchment area is about 690 km^2.

In this river basin, major problems seem to be flooding especially in its middle reach and sedimentation. From such reasons, a diking system in the lower reach and some sabo dams or sand pockets in the middle and upper reaches were constructed upto now. However, the above problems are still remained in the Konto river basin.

From such situation, the detailed design for the Konto river improvement of rehabilitation works is ongoing by Mt. Kelud project and its construction works are partly being carried out.

(c) Widas River

The Widas river has its source on the northern slope of Mt. Wilis. After flowing down towards north for about 30 km, the Widas river is joined by a left bank tributary of the Bening river and flowing down towards south for about 20 km, it is joined by a large right tributary of the Kedungsoko river. Downstream from the Kedungsoko river confluence the Widas river flows towards east-north-east collecting several left bank tributaries and finally joins to the mainstream of the Brantas river at about 89 km upstream from the rivermouth of the Porong river. The river system of the Widas river basin is shown in Fig. 3.5.1. The longitudinal profiles and flow capacities of the Widas, Kedungsoko, Ulo rivers are shown in Figs. RC-5 to 8 in ANNEX RC

In the Kedungsoko river originating in the said Mt. Wilis, it is joined by left bank tributaries of the Kuncir and Ulo rivers. These rivers flow through the urban area of Nganjuk.

In the Widas river basin, three natural flood retention areas exist in the lower reach of the mainstream of the Widas river, in the lower end of the Ulo river and in the middle reach of the Kedungsoko river near the Kuncir river confluence, respectively. These function as the natural retarding basins to reduce a flood peak to downstream reach during the rainy season.

The total catchment area of the Widas river is about 1,539 km^2 and the total river length is about 80 km. The average river bed slope in the middle and lower reaches of the mainstream of the Widas river is 1/900 to 1/3,800.

The Widas river upstream from Lengkong town and the Kedungsoko river, has a diking system in the right bank and locally in the left bank. It was constructed to prevent the irrigated paddy field in the right bank of the above rivers from inundation. Also the Ulo river near Nganjuk town has locally dikes to prevent Nganjuk from inundation.

Flood carrying capacity of the Widas river mainstream and major tributaries is quite low, ranging from 100 to 400 m^3/s in the Widas river and 10 to 100 m^3/s in the major tributaries. From such present channel condition, the basin is subject to frequent inundation almost every year. Among others, Nganjuk, Lengkong, and their surrounding areas are really facing to a serious flood problem. In the basin, a historical flood occurred in 1979 causing a srious damage with a 9,000 ha inundation area and an average duration of 2 weeks. It was reported by BRBDEO that the above flood corresponds to 25 yr probable flood.

The recurrent floodings mentioned above are resulted mainly from

- Insufficient flood control facilities
- Backwater from the Brantas river mainstream

From such situation, flood control and drainage works are essentially and urgently requied in the Widas river basin from the viewpoint of economic and social aspects.

(d) Ngrowo river basin

The Ngrowo river basin is bounded by Mt. Wilis in the north, the Gamping hills in the south, and the mountains ranging from Mt. Wilis and the Gamping hills in the west, and the low alluvial fan formed by the Brantas river in the east as shown on Fig. RC-9 in ANNEX RC The total catchment area of the basin is about 1,600 km^2.

There are many rivers of various sizes in the Ngrowo river basin. At present, the Ngrowo river basin has two drain outlets. One is the Brantas river, which drains the flood water coming from the rivers of the Klantur, Babakan and Bajal rivers, and the total catchment area by this system is about 177 km^2. The another one is the two South Tulungagung tunnels of No. I and No. II through the Parit Agung and Parit Raya canals. The longitudinal profile and flow capacity of these canals are shown in Figs. RC-10 to 13 and drain about 90% of the total basin area in the Ngrowo river into Indonesian ocean. The total flood carrying capacity of the tunnels is about 1,100 m^3/sec.

The Ngrowo river basin, especially Tulungagung town and its surrouding area had suffered from habitual inundation since the beginning of their history. In this basin, many flood control works were carried out and some are subsequently being undertaken. Owing to the completed and ongoing works, the inundation condition has been improved to a great extent. At present, the Ngrowo river basin is no longer under the condition of habitual inundation as those in the past. However, much attention should be paid to sedimentation in the Parit Raya and Parit Agung canals.

(e) Surabaya River

The Surabaya river was originally a branch of the Brantas river and separated from the main Brantas by installation of the Mlirip sluice near Mojokerto in view of flood. The present Surabaya river originates in the Marmoyo river basin and gathers the water diverted from the Brantas river through Mlirip and Gedeg sluices and the run-off from its residual basins. Then, flowing down for about 35 km in north-easterly direction, it enters into Surabaya city, the second big city in Indonesia. There is a dam at Gunungsari. At the just downstream of the Gunungsari dam, the Surabaya river joins the Kedurus river which has suffered from the habitual inundation.

In Surabaya city, the river is divided into two branches, the Wonokromo canal and Mas river. The Wonokromo canal, which forms a mainstream, flows straightly eastwards through Jagir dam and pours into the Madura strait. The Mas river flows through Wonokromo sluice northwards meandering through the center of the Surabaya city and finally debouches to the Madura strait at Surabaya harbour passing through the Gubeng dam. Features of the Surabaya river are shown on Figs. 13 to 18 in ANNEX RC

On the other hand, there exists sea dike along the north-eastern coast of the city over a length about 17 km from Ujung to the Wonokromo canal. This dike has served for years for prevention of flood and salt intrusion from the sea.

The Surabaya river has a total catchment area of 630 km^2 totalling the Marmoyo 290 km^2, Watudakon 99 km^2, Mas 14 km^2, and other small tributaries, 227 km^2. The total length of the river from the rivermouth of the Wonokromo to the uppermost of the Marmoyo is about 100 km.

In this basin, the mainstream of the Surabaya river including the Marmoyo river and Mas river was largely improved by the Surabaya river improvement project. However, there still remains habitual inundation in the urban areas of Surabaya and its hinterland including the Kedurus river basin. Especially, the Surabaya urban areas are really facing to flooding problem almost every year. To solve such constraints, the drainage improvement works are strongly required in the Surabaya urban area and its hinterland.

3. Existing river structure

Many kinds of river structures exist in the river courses of the Brantas river basin. They are classified into dam, sluice, gate, pumping station, bridge and so on. Locations and major dimensions of the structures are presented in the ANNEX RC

The principal structures in the Brantas river and its tributary are dams and sluices. They are the Karangkates, Wlingi, Lodoyo and new Lengkong dams in the main Brantas, Selorejo dam in the Konto river and Bening dam in the Widas river, which function as multipurpose dams or sluice. In the upper reach, the Sengguruh dam is under construction with a purpose of hydropower generation. The others are the intake facilities, culvert and pumps, and bridges. Locations of the existing major structures are shown in Figs. 3.5.2 and 3.5.3.

Among the existing bridges, the Mojokerto highway, Ploso highway and Ploso railway are to be modificated to secure sufficient celarance above the design flood water level.

Construction of Mrican barrage for the Waru-Turi irrigation project is being proposed in the middle reach just downstream of Kediri. In realization of the above barrage construction, special attentions should be paid in view of river maintenance and river administration.

In the upper reach of the Kuncir river, there exists a diversion weir to control flood water to the respective downstream reaches of the main Kuncir river and Kiri Kuncir a left distributary of the Kuncir river. This weir is so superannuated that it is dangerous for flood.

In the Ngrowo river basin, the major river structures are the flood diversion tunnels of the South Tulungagung No. I and No. II, and head works or gates for the irrigation use.

The Wonorejo dam including a connection tunnel and a intake weir of the Segawe, and the Tulungagung gate are being constructed under the Tulungagung drainage project by BRBDEO.

In the Surabaya river basin, the major river structures are the intake facilities and pumping stations. In the lowest reaches of the Surabaya river, there exist three dams or sluices, namely, Gunungsari dam, Jagir dam and Wonokromo sluice. The above three structures function mainly for water supply to the urban area.

4. Riverbed movement and sediment transport capacity

(1) Riverbed movement on the Brantas river

The riverbed movement on the mainstream of the Brantas river and Porong river has been measured by Irrigation Section, DPU, East Java. The data were collected at 19 measurement sites (ref. Fig. RC-24 in ANNEX-RC) including the data of the annual average riverbed elevation at these sites.

Figure 3.5.4 shows the annual average riverbed movement, which was drawn by adding the recent data collected to the past data prepared by the Brantas Middle Reaches River Improvement Project. As seen on this figure, it is recognized that riverbed elevations at each given site, although these show only tendency at a local point, has been lowering as a whole in the recent 10 years. The degradation is remarkable especially in the downstream reaches from Purwasari near Kertosono.

Variations in the discharge rating curves at 6 measurement sites (shown on Fig. RC-25), show almost similar tendency of degradation of the riverbed.

(2) Sediment transport capacity on the Brantas river

Sediment transport capacity on the present and proposed channels of the Brantas river has been studied by the Brantas Middle Reach River Improvement Project. This result is presented in Fig. 3.5.5 (1), which shows annual sediment load estimated varies from 1×10^6 to 6×10^6 m^3 with location as given below.

Sediment load	Site					
	Kaulon	Pakel	Jeli	Jongbiru	Kertosono	Jabon
Bed and suspended	870	1,150	1,250	1,400	1,500	1,650
Wash	400	750	900	1,300	2,000	4,600
Total	1,270	1,900	2,150	2,700	2,700	6,250

Furthermore, the following sediment distribution has been proposed for the total sediment load at Jabon.

K. Brantas at Jabon : 1.65×10^6 m^3/yr
K. Porong : 1.40×10^6 m^3/yr
K. Surabaya and Voor canal : 0.25×10^6 m^3/yr

In this study, sediment transport capacity on the present channel of the Brantas river is preliminarily estimated at major points by using the formula derived from Hydrological Study in the Section 3.2 and mean monthly discharge in the respective years of 1979 and 1982. The year of 1979 was a rich discharge year and the year of 1982, a drought year for the past ten years. Total sediment loads on the present channel for the above respective years are estimated below and presented in Fig. 3.5.5 (2).

(Unit: m^3/yr)

Year	Pakel	Jongbiru	Kertosono	Jabon
1979				
Bed	113,000	192,000	214,000	128,000
Suspended	565,000	961,000	1,072,000	642,000
Sub-total	678,000	1,153,000	1,286,000	770,000
Wash	363,000	1,269,000	1,356,000	3,415,000
Total	1,041,000	2,422,000	2,642,000	4,185,000
1982				
Bed	98,000	138,000	171,000	109,000
Suspended	492,000	690,000	857,000	544,000
Sub-total	590,000	828,000	1,028,000	653,000
Wash	210,000	577,000	734,000	1,715,000
Total	800,000	1,405,000	1,762,000	2,368,000

From the above table and figure, it is recognized that the estimated sediment transport capacity of the present channel is still lower than that of allowable ones proposed. From such viewpoint, river channel improvement of the Brantas middle reaches is needed to increase the sediment transport capacity of the channels.

3.5.3 Probable Flood Damage

(1) General

In recent 10 years, there was no flooding in the main Brantas due to overtopping the levees. In the tributaries, there were floodings like in the Widas basin in 1979. However, there is no complete flood damage record covering all the damages due to flooding. Therefore for measuring the effects of the flood control works, probable flood damage are estimated by analytical methods.

The following analytical procedures are adopted.

(1) Delineation of the floodplain according to the topography

(2) Estimation of area - depth - duration of flooding for different flood magnitudes

(3) Estimation of property values subject to flooding within the delineated floodplain

(4) Estimation of damage rates according to the flooding depth - duration and nature of properties

(5) Estimation of probable flood damages based on (2), (3) and (4) mentioned above

The details of the analysis are to be referred to ANNEX RC.

2. Delineation of floodplain

The floodplain subject to inundation is delineated as shown on Fig. 3.5.6, based on 1 to 50,000 topographic maps and the reference to the previous studies.

The floodplain starts from the confluence with Ngoro river and extends along the main Brantas up to near Mojokerto. Then, it turns to Surabaya river, and reaches the seashore. One branch is set in the low land on the Widas basin. The floodplain includes Kediri, Kertosono, Ploso, Jombang and Mojokerto along the main Brantas and Nganjuk and Lengkong along Widas river.

3. Area - depth - duration analysis

Area - depth - duration analysis is made using the estimated probable floods as explained in the Hydrological Study, topographic information from 1 to 50,000 topo maps, and river cross sections by non-uniform flow analysis.

Cross sections of the floodplain are prepared from 1 to 50,000 topo maps at the interval of 1 km from the seashore, covering both the boundaries of the floodplain. The Manning's coefficient of hydraulic roughness is assumed as follows:

- overland flow	n = 0.20
- in river channel	
0 k to 60 k	0.025
60 k to 88 k	0.028
88 k to 140 k	0.032
upstream of 140 k	0.035
K. Widas	0.030

Discharge rating curve at each cross section in the floodplain is calculated by the non-uniform flow analysis. From rating curves, the inundation depth due to the probable floods of 2-, 5-, 10-, 25-, 50-, and 100-year return period is estimated as shown in Table 3.5.1. Based on these results, the area to be inundated by each probable flood is drawn as shown on Fig. 3.5.7.

4. Property Values in the floodplain

(1) Kind of properties in the floodplain

Through reconnaissance over the floodplain, existence of the following properties is found;

- various crops on the farm lands
- fish pond
- buildings for households, stores/hotels/restaurants, commercial buildings, factories, etc.
- indoor movables by the types of building specified above
- infrastructures such as roads, bridges, levees, canals

(2) Assessement of property values

For assessment of the property values,the following items are selected as representative ones;

- crops --- rice, maize, soybeans, and peanuts
- fish ponds --- fishes and fish pond facilities
- buildings --- households (urban and rural), factory, store/ hotel/restaurant and commercial building
- indoor movables --- corresponding to the classification of buildings

Since data on the economic values of the above items covering the entire floodplain are not available, present economic unit value and yield of each item are estimated analytically. Details of estimation of economic unit value of each item in 1984 are given in Note RC-1, and results are summarized as shown below.

(a) Unit value of crops per ha

Kind of crops related to the floodplain are rice, maize, soybean and peanut. The details of present unit yield per ha and economic price per ton concerning these crops are estimated, then unit value of crops per ha is estimated as shown below.

Unit: Rp./ha

Year	Paddy	Maize	Soybean	Peanut
1984	648,360	305,280	265,260	778,700

(b) Unit cost of buildings

Present unit cost per each type of building is estimated below.

Unit: 10^3 Rp./building

Year	House		Factory	Commercial building	Store/hotel /restaurant
	Rural	Urban			
1984	905	2,580	302,500	107,910	23,925

(c) Value of indoor movables per building

Value indoor movables per building corresponding to the classification of buildings mentioned above is shown below.

Unit : Rp.

Year	Household	Factory	Commercial building	Store/hotel /restaurant
1984	745,400	$7,2 \times 10^6$	32×10^6	$1,82 \times 10^6$

(d) Value of fishes and facility per ha

Value of fishes and related faciltiy per ha is shown below.

Unit : 10^6 Rp.

Year	Fish	Facility
1984	0.84	0.13

Since the economic development in the basin will continue in future, variations in unit value of crops, fish, fish pond facility, buildings and indoor movables related to buildings are projected as follows. Future unit value of items mentioned above is forecasted up to 2060, taking into account economic life of flood control facilities. Future values of items are shown at 1984 constant price.

(a) Future unit value of crops per ha

Future unit value of crops per ha is shown below.

Unit: Rp./ha

Year	Paddy	Maize	Soybean	Peanut
1990	1,143,620	414,740	368,286	483,749
2000	1,397,390	657,475	538,412	661,973
⋮	-do-	-do-	-do-	-do-
2060	-do-	-do-	-do-	-do-

(b) Future unit cost per building

Unit: 10^3 Rp./building

Year	House		Factory	Commercial building	Store/hotel / restaurant
	Rural	Urban			
1990	1,062	3,030	354,750	126,500	28,220
2000	1,383	3,940	462,000	164,860	36,470
2020	2,230	6,340	743,875	264,980	58,740
2040	3,420	9,740	1,142,630	407,060	90,260
2060	5,030	14,329	1,680,250	598,900	132,830

(c) Future value of indoor movables per building

Unit: Rp./building

Year	Household	Factory	Commercial building	Store/hotel /restaurant
1990	870,140	14.2×10^6	37.4×10^6	2.4×10^6
2000	1,124,140	17.9×10^6	42.0×10^6	3.3×10^6
2020	1,778,410	$34,1 \times 10^6$	68.0×10^6	5.1×10^6
2040	2,684,390	$64,4 \times 10^6$	$98,0 \times 10^6$	7.5×10^6
2060	3,888,410	$95,3 \times 10^6$	145.0×10^6	10.9×10^6

(d) Future value of fishes and facility per ha

Unit: 10^6 Rp./ha

Year	Fish	Fish pond facility
1990	0.94	0.14
2000	1.15	0.16
2020	1.65	0.21
2040	2.28	0.26
2060	3.06	0.32

(3) Distribution of properties

Distribution of the properties is assessed by two processes, mesh survey and average density of properties drived from economic data.

The floodplain is divided by meshes having mesh intervals of 500 m. In each mesh, the elevation of the ground surface and the landuse condition (paddy, upland, fish pond or building area) are read out from 1 to 50,000 scale mpas. The total number of the meshes is 6,041, and information on each mesh is shown in DATA BOOK. Distribution of meshes by Kab/Kodya and land use categories is as shown in Table 3.5.2.

The number of houses, factories, stores/hotels/restaurants, and commercial buildings is estimated by using data on average building density per 25 ha (500 m^2) by the Kabupaten. An assumption is that buildings are evenly distributed in every mesh of building area of flood plain. Therefore, the number of each type of buildings is calculated on the Kab. level in such a way that the number of meshes related to the building area is multiplied by building density per mesh in each Kab.

The classification into urban and rural household in the Kab. level is based on the ratio of urban households to rural ones which was derived from Population Census of East Java in 1980. Table 3.5.3 shows the number of buildings by Kab./Kodya and type of building.

Moreover, the future increase of buildings is estimated in the following table.

(Unit: % per annum)

Year	House		Factory	Commercial building	Store/hotel/ restaurant
	Rural	Urban			
1984					
	0.11	1.68	1.5	1.5	2.0
2000					
	0.19	2.05	1.0	1.5	1.0
2020					
	0.10	0.50	0.5	1.5	1.0
2040					
	0.10	0.50	0.5	1.5	1.0
2060					

5. Flood damage

(1) Kind of flood damages and damage rates

Flood damages consist of tangible and intangible ones, and the tangible damages can be divided further into direct and indirect ones. The direct flood damages are losses of values of properties due to physical contact with flooding and inundation. Indirect flood damages stemms from losses in the economy caused by the direct damages. Indirect damages comprise losses of wages, evacuation cost, etc.

Damage rates of the direct damages are assumed as follows;

- crops and buildings	:	standard rates developed as shown in Table 3.5.4
- fish pond	:	100% for facilities and fishes
- infrastructures	:	30% of total damage of crops and buildings. This 30% is taken referring to the damage rate obtained in Malaysia, since the actual data are not avialable in the Brantas basin.

Damage rate of the indirect flood damages is assumed at 10% of the direct flood damage.

(2) Probable flood damages without flood control facility

Innudation areas by the probable floods are estimated according to the classifications of paddy, upland crops, fish ponds and building areas as shown in Table 3.5.5.

For the estimation of damages, objective inundation area is divided into two districts of the Brantas main stream basin and Widas river basin. Valuation of the flood damages by each probable flood is made in two level of the economic conditions of the year of 1984 and 2000. They are shown in Table 3.5.6.

From the above results, the annual average flood damage up to certain return period are shown in Table 3.5.7. When objective reaches up to 139 K and the Widas basin are taken into account, the total potential flood damage are $117,422 \times 10^6$ and $8,202 \times 10^6$ Rp up to 100 yr probable flood for present lavel.

(3) Protective flood damage with existing flood control facilities

The river canal of the main Brantas can safely flow floods having the return periods of 30 years. The protective flood damages by the existing flood control facilities are calculated at Rp. $114,000 \times 10^6$ when objective reaches up to 139 K is taken into account.

Thus, the protective rate is estimated to be 97%, assuming that total potential flood damage is $117,422 \times 10^6$ Rp, up to 100 yr probable flood.

(4) Protective flood damage with proposed flood control facilities

As mentioned in the next section, proposed is protection against the 50-yr probable flood in the main Brantas and the 25-yr probable flood in the Widas basin. With the proposed flood control facilities, the protective flood damage is $115,661 \times 10^6$ Rp. in the reaches up to 139 K for the present level. Thus, the protection rate is estimated to be 98.5%. In case of protection against the 25-yr probable flood in the Widas basin, the protective flood damage is estimated to be $7,689 \times 10^6$ Rp. for the present level. Thus, the protection rate is estimated to be about 94%, assuming that total potential damage is $8,202 \times 10^6$ Rp. up to 100-yr probable flood.

3.5.4 Ongoing and Future Flood Control Project

Under the basin development plan, many flood control works have been carried out by BRBDEO since 1968. The major works undertaken in the basin are (1) Porong river improvement, (2) Surabaya river improvement, (3) Tulungagung drainage project and (4) Brantas middle reach river improvement project which is under implementation.

The major ongoing flood control works and future ones which have been identified by BRBDEO are described below.

1. Ongoing project

Ongoing flood control projects in the basin are the Brantas middle reach river improvement project (Stage II) for which OECF, Japan has provided a financial aid, and Tulungagung drainage project (Phase I and II) for which ADB has provided a financial aid. The location map of the ongoing projects including the completed ones in presented in Figs. 3.5.8 and 3.5.9.

(1) Brantas middle reaches river improvement (Stage II)

The Brantas middle reaches river improvement project is essentially a flood alleviation project, concerned with the part of the Brantas mainstream between Kediri and the Lengkong dam. The project aims to increase the discharge capacity of the Brantas middle reaches by dredging and by remodelling critical reaches. The stage I project consists of channel improvement for the 10-yr flood and the rehabilitation of some minor irrigation offtakes. The stage II project is designed for the 50-yr flood and consists mainly of heightening levees and dredging.

The stage I is scheduled to be finished within the 1984/85 fiscal year and subsequently, the stage II is to be commenced from the coming fiscal year. The work quantities and the typical cross sections are presented in ANNEX-RC.

(2) Tulungagung drainage project

The feasibility study for the Tulungagung drainage project was made in December 1979. The project is to be implemented in two phases;

phase I and phase II. Phase I construction works commenced in 1980. Flood water from the Ngrowo basin, upstream of the confluence of the Song and Ngrowo rivers, is diverted south to drain into Indonesian ocean. The works consist of the construction of a main drainage canal (Parit Agung canal), 24 km in length and the construction of a new drainage tunnel (South Tulungagung tunnel II), which runs in parallel with the existing South Tulungagung tunnel I. The Parit Agung canal is designed for the 10-yr probable flood.

2. Future projects

Future flood control projects which have been identified by BRBDEO or other agencies are presented in Fig. 3.5.9.

The outlines of the future projects are described below.

(1) Widas basin flood control and drainage project

A preliminary study of the Widas basin flood control and drainage project was made by the BRBDEO in 1979. According to the study, the planned development of the Widas basin is basically divided into two categories; the first is the improvement of the main river channels, and the second the construction of three dams having the functions of flood control and irrigation. River channel improvement works proposed are for an overall length of 98 km, covering the Widas, Kedungsoko, Kuncir and Ulo rivers. The proposed dams are Kuncir, Kedungwarak and Semantok.

Feasibility study for the selected among the above will be carried out in the 1985/86 fiscal year by JICA, Japan.

(2) Surabaya river improvement project (stage II)

Surabaya river improvement project, stage II has been formulated by BRBDEO with coordination of DGWRD. The objective of the project is to protect the Surabaya urban area and its hinterland from habital inundation following the first stage works which were already accomplished in 1981 (Refer to ANNEX RC). The proposed works involve;

(a) Gunungsari canal improvement works (for 5-yr flood)

- Improvement of Gunungsari drainage canal	:	ℓ = 6.7 km
- Construction of Gunungsari flood diversion channel	:	ℓ = 6.1 km
- Alignment of Gunungsari irrigation canal	:	ℓ = 6.7 km

(b) Kedurus river improvement works (for 20-yr flood)

- Channel improvement	:	ℓ = 5.0 km

(c) Repair of Wonokromo sluice

- Motorization of gates	:	2 nos
- Sealing of navigation locks	:	2 nos

The detailed design for the above project is scheduled to start from 1985, with the financial aid of OECF, Japan.

(3) Widas lower reach and Mrican barrage project

This is a project for which ADB has provided a financial aid. Feasibility study on the project was completed in 1981 by the UK grant.

The purpose of the project is to rehabilitate the irrigation systems about 24,000 ha in the Warujayeng and Turi-Tunggorono areas. The proposed works consist of the following (Refer to ANNEX-RC for details).

(a) Construction of a new Mrican barrage (Mrican barrage) on the Brantas river mainstream

(b) Construction of the irrigation facilities of about 24,000 ha of paddy field

(c) Improvement of the Widas river lower reaches to protect its irrigation area from the flood (Ref. Fig. RC-28)

The works on the items (a) and (c) have been scheduled to be conducted by BRBDEO, and the item (b) by another agency. The item (b) is under the construction and the items (a) and (c) are under the detailed design.

(4) River improvement works by Mt. Kelud project

The purpose of this work is to rehabilitate and to improve the lower reach of the Konto river and small rivers which originate on the slope of Mt. Kelud in relation with disaster prevention works in Mt. Kelud by Mt. Kelud project office in Kediri.

The project consists of emergency works, short term works and long term works. The lower channel of the Konto river is to be rehabilitated as emergency works and its detailed design are ongoing by the said project. The channel in the lower reach of the Konto river is designed for 25-yr probable flood according to Mt. Kelud project. The basic concept of the Konto river improvement is shown on Fig. RC-29. Small rivers originating in the slope of MT. Kelud are to be improved or rehabilitated. The details of the project features are not yet decided.

(5) Mrican barrage construction for Waru-Turi irrigation project

As aforementioned in the above (3) : Waru-Turi irrigation project with construction of Mrican barrage has been planned to serve the irrigation area of about 38,000 ha through the construction of a barrage on the main Brantas at about 2 km downstream of the existing Mrican intake.

The most serious problem implicated in the barrage construction would be (a) sediment deposition in the upstream reaches of the barrage and (b) river bed erosion in the downstream of the barrage.

The Brantas Middle Reaches River Improvement Project has presented a report on "The Influence of the Construction of Waru-Turi Barrage on Brantas Middle Reaches River Improvement Project". The report has suggested the following.

(a) When the barrage is constructed, river dredging amounting to about 630,000 m^3 per annum is to be carried out for maintaining the designed river condition.

(b) Required dredging volume would be much more during several years after the Mt. Kelud eruption

(c) There would be the case that all the gates are obliged to be fully opened when unexpected serious situation occurs due to Mt. Kelud eruption.

(d) With regard to the degradation in the lower reaches, proper counter measures to be taken.

The above suggested would be qualitatively true.

To eliminate or to minimize the future problems with regard to the barrage construction, the JICA Study Team recommends the following.

(a) To clarify the development scheme of Waru-Turi Project through detailed studies on such conceivable alternative plan as;

(i) Pumping irrigation scheme without a barrage
(ii) Free intake scheme without barrage
(iii) Barrage scheme

(b) To clarify the following in case barrage scheme is selected as the optimum development scheme.

(i) Operation rule of the barrage
(ii) Responsibility of barrage operation
(iii) Responsibility of river maintenance

3.5.5 Flood Control Plan

1. Necessity of flood control countermeasures

The Middle Reaches River Improvement Projects which is designed with 50-yr probable flood is under implementation. Recently, considerable big flood occurred on 1981 and 1984 which were almost the same magnitude with the design flood. There is a possibility that flood magnitude in future will increase owing to the urbanization, tributaries improvement, etc. Therefore, overall review of basin wide flood control scheme is made herein. The Widas basin, suffering from severe floods frequently, has only a partial flood control and drainage plan which will protect the Warujayeng irrigation area. The most important areas to be protected from flood inundation are the urban area of Nganjuk, Lengkong, etc. and the inter-provincial connections of highway and railway. Therefore, and overall plan for the flood control and drainage improvement is needed for the Widas basin.

2. Basic concept of flood control panl

Referring to the probable flood scales applied to the rivers in Indonesia as shown in Table 3.5.8 and Fig. 3.5.10, the following basic conditions are recommended as a basis of flood control study.

(1) Brantas and Porong river mainstreams

(a) Objective probable flood : 50-yr return period

(b) Objective stretches : River mouth to Ngrowo river confluence

(2) Widas river basin

(a) Objective probable flood : 25-yr return period

(b) Objective stretches

- Widas river : Confluence with Brantas river to Ngudikan dam (42.7 K)

- Kedungsoko river : Confluence with Widas river to Badung bridge (10.0 K)

- Ulo river : Confluence with Kedungsoko river to bridge (17.8 K)

- Kuncir river : Confluence with Kedungsoko river to bridge (13.0 K)

(3) Other tributaries

Other tributaries joining to the Brantas river main stream are taken up in this study as these are the present conditions.

3. Review of the present design discharge distribution

The design discharge distribution which is currently adopted in the Brantas river is given in Fig. 3.2.9 and outlined below.

Porong river	:	Rivermouth to new Lengkong dam	1,500 m^3/s
Brantas river	:	New Lengkong dam to Widas river conf.	1,500 m^3/s
		Widas river conf. to Konto river conf.	1,100 m^3/s
		Konto river conf. to 139 K point	900 m^3/s

With regard to the flood distribution, 50-yr probable floods are newly estimated based on the hydrological data obtained lately including the recorded largest flood in 1984 under the present condition. The result is summarized below.

Porong river	:	Rivermouth to Lengkong dam	1,600 m^3/s
Brantas river	:	Lengkong dam to Brangkal river conf.	1,600 m^3/s
		Brangkal river conf. to Widas river conf.	1,500 m^3/s
		Widas river conf. to Konto river conf.	1,250 m^3/s
		Konto river conf. to Ngrowo river conf.	1,050 m^3/s

According to the above results, the present design discharge of the Brantas river is evaluated to have 20 years or more yrs return period.

4. Flood Control Plan in the Brantas and Porong rivers

(1) Alternative flood control schemes

The flood control scheme of the main Brantas is studied on conceivable alternative cases in order to select the most preferable plan. The alternative cases are based on the following considerations.

(a) The main Brantas and Porong river have already been improved by the channel excavation and diking in the densely populated riparian area in the stretches of about 140 km long, associating relocation of the riparian peoples and restriction on the land use. In due consideration to the above, it is conceivable that a large-scale re-improvement of the main Brantas after the second stage of the Middle Reaches River Improvement project causes large-scale resettlement of the riparian population, which would result in sociological problems that are beyond the economic effectiveness of the flood control works. Alternatively, a flood diversion scheme from the Lodoyo reservoir to the Indonesian ocean in the upper Brantas is one of the conceivable flood control plans.

(b) Even though there is a countermeasure as mentioned in (a), the increase in the flow capacity in the main Brantas will be expensive and take time. Therefore, any sub-basin development, tributary improvement and urbanization which will result in increasing in the flood inflow into the main Brantas, shall be planned in due consideration to the costs to be required for supplement of the flow capacity to the main Brantas or to controlling the increase of the inflow within the sub-basin.

(c) Mt. Kelud has erupted with the average cycle of 15 years. The erupted materials flowed into the middle reaches of the Brantas river and caused aggradation of the river bed. Although there have been intensive efforts to control the erupted materials by sabo facilities, it will be difficult to avoid temporary aggradation of the river bed after eruption. In such occasion, the flood carrying capacity in the middle reaches would be largely lowered.

From the above, conceivable alternative flood control schemes are selected in view of feasibilities that the Brantas and Porong rivers main streams are to be largely reimproved or not, and that the unleveed reach upstream from Kediri which functions at present as a natural flood retarding basin is to be confined by diking system or not. Based on the above considerations the following flood control components are taken into account.

(i) Flood diversion channel to the Indonesian ocean in the upper reach of the Brantas river

(ii) River channel improvement of the main Brantas and the Porong river

(iii) Flood control dam in the Widas river basin

(iv) Artificial retarding basin in the Widas river

(v) River channel improvement of the Widas river

(vi) Combinations of the above components

For the above item (iii), Kuncir flood control dam scheme was separately studied as described in the following sub-section. According to the study results, it is said that a reduction effect of the speak discharge to downstream by the dam is negligibly small due to the limited reservoir capacity and small catchment area at the dam site, and it is not economically viable. From the above reason, the Kuncir flood control dam component is abandoned in the present study.

From the above, the following two alternative schemes are firstly studied in order to clarify the influence of the confinement of natural retarding area in the main Brantas.

Scheme 1 : The unleveed reach upstream from Kediri (natural retarding basin) is kept as it is.

Scheme 2 : The above reach is confined by dikes

In this study, the ongooing Brantas middle reaches river improvement project (Stage II), is assumed as the completed one.

The following alternative cases for each scheme are considered.

(a) Alternative cases of scheme 1

The scheme 1 is principally divided into 2 cases with and without flood diversion channel in the upper Brantas river. Concept of the scheme 1 shown in Fig. 3.5.11. The retarding basin scheme is described in the subsequent section and is presented in Fig. 3.5.12.

Scheme 1

Case 1 : Brantas mainstream channels are improved without flood diversion channel and the case 1 is further divided into the following 2 cases.

Case 1-1 : Widas river and its tributaries are to be improved by constructing confining diking system so that natural retarding basin is no longer functioning as flood retention purpose (without retarding basin).

Case 1-2 : Widas river and its tributaries are to be improved by minor improvement in combination with retarding basins (with artificial retarding basins)

Case 2 : Flood diversion channel is constructed in the upper Brantas river. The flood diversion channel is to be branched off at just upstream of Lodoyo dam and the concept of the flood diversion channel is presented in Fig. 3.5.13. This case 2 is also divided into the following two cases and a diversion capacity of 100 m^3/sec is applied as minimum cost case based on the preliminary study results of the respective cases.

Case 2-1 : Without artificial retarding basins in the Widas river

Case 2-2 : With artificial retarding basins in the Widas river

(b) Alternative case of scheme 2

In the scheme 2, unleveed reach upstream from 139 K point is confined by new dikes. The scheme 2 is divided into the following two cases with and without flood diversion channel in the upper Brantas river Concept of the scheme 2 is presented in Figs. 3.5.11(2) and 3.5.12.

Scheme 2

Case 1 : Brantas main stream channels are mainly improved and this case is further divided into the following 2 cases.

Case 1-1 : Without artificial retarding basins in the Widas river

Case 1-2 : With artificial retarding basins in the Widas river

Case 2 : Flood diversion channel is constructed in the upper Brantas river and the Brantas river main stream channels are improved by minor improvement. Also the case 2 is divided into the following 2 cases and a diversion capacity of 400 m^3/s is applied as minimum cost case based on the preliminary study results of the respective cases.

Case 2-1 : Without artificial retarding basins in the Widas river

Case 2-2 : With artificial retarding basins in the Widas river

(2) Flood Discharge Distribution

Flood discharge distribution for each alternative scheme is estimated on the basis of the Storage-Function model constructed under the present river condition. The basic criteria in calculating the flood runoff of each alternative scheme are mentioned as follows.

(a) Confining the inundated area by dike

The following river width based on the relation between catchment area and river width shown on Fig. RC-30 is applied for the inundated areas;

- The upstream from Kediri	200 m
- The Widas river	200 m
- The Kedungsoko river	150 m
- The Ulo river	150 m
- The Kuncir river	100 m

The storage functions of the above channels are estimated by uniform flow calculation.

(b) Controlled retarding basin

It is assumed that the present natural retarding areas in the Widas basin; Widas, Ulo and Kedungsoko are to be modified to the retarding basins with control facilities. Taking into account the topographic conditions and distribution of houses, the retarding areas are delineated, and the control capacities are assumed as 10 MCM in Widas, 8 MCM in Ulo and 9 MCM in Kedungsoko. The retarding effects of three basins are estimated that the excess flood discharge beyond the given condition of inflow into the main Brantas is controlled by and stored in the retarding basins. Control is assumed as horizontal cutting of flood hydrograph at each retarding basin.

(c) Flood Diversion

The excess flood discharge beyond the present design discharge capacities of the middle and lower reaches of the main Brantas is assumed to be diverted to the Indonesian ocean through the flood diversion channel and tunnel, leaving a constant amount equivalent to the base flow in the rainy season in the main stream.

(d) Inflow from major tributaries into the Main Brantas

According to the above flood control plans, the basin and channel coefficients of the Storage Function model are modified, and distributions of the probable flood flow are estimated. For the main stream, Fig. 3.5.14 shows the flood hydrographs in Scheme 1, Case 1-1, and Fig. 3.5.15 shows those in Scheme 2, Case 1-1. Fig. 3.5.16 shows the flood hydrographs in the Widas basin without retarding basin. Flood distributions of the alternative cases are determined as shown in Table 3.5.9 and 3.5.10 and as illustrated on Fig. 3.5.17 and 3.5.18.

5. Design Criteria

The following are the design criteria adopted to the river channel improvement based on the existing site condition.

(1) River width

A standard developed in Japan is adopted in principle. This standard is given in Fig. RC-30. For rivers which were already improved by a diking system, the existing both dikes are fully employed as it is, in principle.

(2) Low water channel

Low water channel is widened and/or deepening depending on river condition. Remarkable meandering of low water channel is smoothened by means of cutoff channel.

(3) Design flood water level

For rivers which has employed or will employ diking system, the design high water level is set not so as to heighten the existing design flood water level or max. flood level in the past, in principle. For rivers which will be improved by excavation of low water channel without a diking system, it is set at average ground height as much as possible.

Design cross sectional area of the channel is estimated based on the uniform flow method. The following are adopted to the Manning's coefficient of roughness considering existing design value or existing river channel condition.

Porong river	:	Rivermouth to 47 K	0.025

Brantas river	: 47 K to 60 K	0.025
	62 K to 88 K	0.028
	90 K to 140 K	0.032
	140 K to 160 K	0.035
Widas river		
Kedungsoko river	Low water channel	0.030
Ulo river	High water channel	0.050
Kuncir river		

The design flood water levels are shown on Fig. RC-31, 32, 33, 34, 35.

(4) Dike section

Dike section as shown in Fig. RC-36, is adopted to the design of dike.

(5) Treatment of sub-tributary

For sub tributary which joins to rivers to be improved, a back water levee is provided.

(6) Flood diversion tunnel

Horse-shoe type is adopted to the design cross section of the tunnel and a single section to the open channel. The following diversion tunnels are taken up in the study.

Design discharge	Diameter of tunnel
Q = 800 m^3/s	D = 6.2 m x 2 lanes
600	7.5 x 1
400	6.2 x 1
300	5.4 x 1
200	4.4 x 1
100	3.1 x 1

(7) Retarding basin

The surrounding dike, side overflow dike and drainage sluice are provided to utilize existing flood retention areas in the Widas more effectively. Surrounding dike height is designed with a height less than 1 m.

5 Proposed river improvement plan

In accordance with the design criteria and respective flood discharges, river improvement plan for each case is studied in the master plan study level.

6. Selection fo proposed flood control plan

(1) Construction cost and net present value

In this comparative case study, the direct cost for civil works, and land and compensation costs are estimated for each case. The adopted unit construction costs are estimated based on these applied to the ongoing flood control projects in BRBDEO.

Some cases among the comparative ones have possibilities to create new available land for irrigation by confining levees.
The above net areas are estiamted at 700 ha to 3,500 ha depending on cases. Benefit from land enhancement is estimated as negative cost in this comparison and the benefit is assumed at Rp. 721,000/ha per annum. Assuming that the land to be free from habitual inundation is used as paddy field with the present cropping pattern and crop yields in the vicinity of such area. The estimated construction costs and net present values are presented in Table 3.5.11 and 3.5.12 respectively. The above estimation includes flood control benefit and other project costs such as engineering, administration, contingency and so on.

As seen in the above tables and Fig. RC-37, the case 2-2 of scheme 1 is the least cost one among the 8 alternative cases and shows the highest EIRR of 8.2 % under the present basin development level.
In addition, it can be said that the reimprovement of the Brantas river and Porong river main stream channels is considerably costly. On the other hand, the respective cases of scheme 2 which confine the unleveed reach upstream from Kediri (natural retarding basin) by dikes are further costly ones more than those of scheme 1.

The above study reveales that tributaries should be improved not so as to increase inflow to the Brantas river main stream and the unleveed reach upstream from Kediri should be remained as it is.

(2) Selection of proposed flood control plan

The case 2-2 with a flood diversion channel scheme with the discharge capacity of 100 m^3/s under scheme 1 is the least cost one among the 8 alternative cases, and is a preferable flood control method in view of technical, economical and social aspects in order to realize the objective to protect the riparian area from the 50-yr probable flood.

Mt. Kelud erupted in the averae cycle of 15 years in the past, and is still active. In any case of Mt. Kelud eruption, it can be admittedly said that the riverbed elevation in the middle reaches of the Brantas river will rise with a fluctuation of 1 to 2 m, owing to large sediment inflow in a short time after eruption, exceeding the sediment discharge capacity of the Brantas river. Consequently, the discharge capacity of the middle reaches will be lowered by about 400 m^3/s from the design capacity. In this case, the safety level of the main Brantas against floods will be lowered to less than 10-yr return period. In order to cope with the case of Mt. Kelud eruption, it is necessary to provide an additional capacity of 400 m^3/s to the diversion system. Taking into account an allowance of 100 m^3/s for decrease of channel regulation effect, the required diversion capacity is estimated at 600 m^3/s (100 m^3/s + 400 m^3/s + 100 m^3/s) in order to discharge safely the new design discharge at any time.

From the above, the case 2-2 of scheme 1 with a diversion capacity of 600 m^3/s is selected as the proposed flood control plan in the Brantas basin. However, it should be noted that the diversion capacity is further studied considering the abovementioned factors.

The design discharge distribution of the above proposed plan is presented in Fig. 3.5.19 and principal features of the proposed plan are as follows;

-Construction of a new diversion channel to the Indonesian Ocean just upstream of the Lodoyo dam.

Open channel;	4.7 km
Tunnel ;	5.5 km
Control gate;	3 no
Capacity ;	600 m^3/s

-Minor improvement of the main Brantas

Excavation ;	0.25×10^6 m^3
Revetment ;	13,000 m^2

The constructon cost of the proposed plan is estimated at Rp. $121{,}000 \times 10^6$ and the project cost including administration, engineering and physical contingency is presented in Table 3.5.13.

7. Flood Control Plan in the Widas river basin

The comprehensive flood control plan in the Widas basin is studied within the framework of the overall flow control plan in the Brantas basin, and on the conditions that the design flood is 25-yr flood and the maximum inflow into the main Brantas is 270 m^3/s. The discharge of 270 m^3/s is the allowable one when the 50-yr flood flows in the main Brantas, under the present condition as described in Section 3.2, and make it possible to commence the flood control works in the Widas basin separately from the main Brantas flood conrrol works.

The main purpose of the above plan is to protect Nganjuk, Lengkong and those surrounding areas and two trunk systems of railway and highway from habitual inundation. To solve such constraints, the following flood control components are conceivable in the Widas river basin.

- Flood control dam in the upper Kuncir river
- Retarding basin
- River channel improvement

In this section, the above mentioned components are further studied to select the basin flood control scheme in the Widas river. The general basin map of the Widas river basin is shown in Fig. 3.5.20.

(1) Kuncir flood control dam scheme

To directly protect Nganjuk from flood, the Kuncir dam scheme is considered as an alternative. The effect of the Kuncir dam construction is studied in the following.

The selected cases are the with and without Kuncir dam in combination with the river channel improvement scheme. Those cases are described below and shown in Fig. 3.5.21.

Scheme 1 : River channel improvement with Kuncir dam

The Kuncir dam is planned to retard full volume of the objective flood run-off from the upper basin. The following are adopted to the proposed dam scheme.

- Purpose : Flood control
- Dam type : Concrete gravity
- Dam height : 44 m
- Reservoir capacity : 9×10^6 m^3
- Catchment area at dam site : 73 km^2

General features are shown on Fig. RC-38.

Scheme 2 : River channel improvement without Kuncir dam

River channels are improved by constructing diking system and excavating low water channel. This scheme is just same as that of case 1-1 of the scheme 1 studied in the previous section.

Design discharge distributions for the above are shown in Fig. 3.5.22.

The construction costs of the both schemes are summarized below and broken down in Table 3.5.14.

Estimated construction cost

($Rp. 10^6$)

Scheme	Construction cost	
Scheme 1	Dam	25,738
	River channel	60,726
	Total	86,464
Scheme 2	River channel	63,939

Ref. Table RC-8

From the above, the scheme 1 is more costly than that of the scheme 2 and therefore, the flood control dam is not adopted in the proposed basic scheme.

(2) Retarding basin scheme

There exist three flood retention areas in the lower reaches of the Widas river basin which function as natural retarding basins. Those locations are given in Fig. 3.5.20. Major dimensions of the existing retarding basin including occasionally affected area are assumed as follows, at the following respective water levels.

Dimension of retarding basin

I t e m	Retarding basin		
	Widas	Ulo	Kedungsoko
Water level (m,SHVP)	38.0	45.0	45.0
Area (km^2)	10	7	12
Volume (10^6 m^3)	10	8	9

Ref. Fig. RC-39, RC-40

The both flood control schemes with and without retarding basin have been studied as described in the preceeding section, and this study result proves that the flood control method in combination with the artificial retarding basin scheme is more preferable in view of not only river flood control but also future basin development in the whole Brantas river basin.

Required area for retarding basins are roughly estimated as follows. These figures are to be re-studied in the feasibility study under Part-II study of JICA.

Retarding basin	Required area (Km^2)
Widas retarding basin	10
Ulo retarding basin	5
Kedungsoko retarding basin	5
T o t a l	20

Ref. Fig. RC-39, RC-40

The above total retarding basin area covers the habitual inundation area which is severely influenced throughout every rainy season. The outline of the retarding basin scheme is given in Fig. RC-40.

(3) Flood Control Plan in the Widas river

In the preceding section, it has been concluded that the Widas river flood control and drainage project is to be carried out prior to and independently of that of the Brantas river main stream. Prior to the implementation of the above works, a comprehensive plan is to be formulated with due consideration of stegewise development.

The details of the comprehensive and urgent plans are to be studied as the Feasibility Study in the Part II stage.

The following are the basic strategy for the coming Part II study.

(a) Basic design flood

Comprehensive plan	:	25-yr probable flood
Urgent plan	:	10-yr probable flood

(b) Maximum inflow to Brantas river : 270 m^3/s

In accordance with the above basic strategy, the Widas flood control scheme is to be formulated under the following considerations.

(a) Kuncir flood control dam scheme is not considered

(b) Existing three retarding basins are to be carefully studied whether these are to be remained as natural retarding basins or to be improved as artificially controllable basins

(c) To protect Nganjuk from flooding, flood way scheme is to be considered, i.e. the Ulo river is improved as one alternative for main flood way or new diversion channel is constructed to divert a part of flood discharge originating in the upper basin of the Kuncir river and Ulo river directly into the Widas river.

Based on the above considerations, the principle alternative schemes are provisionally presented in Fig. 3.5.23.

8. Tributaries Originating from Mt. Kelud

The Mt. Kelud project plans/executes the construction of sand pockets and sabo dams, and the river improvement on the tributaries originating from Mt. Kelud. In this study, it is assumed that the river improvement works in Mt. Kelud area do not affect the flood discharge ofthe main Brantas. However, it is feared that the execution of the river improvement may cause the increase of flood discharge on the main Brantas, if it is done in consideration only of the Mt. Kelud area.

In this sub-section, the retarding effect of the river channel is analyzed for the following river width using 10 years flood on the Ngobo river.

River Width (m)	Estimated by
90 m	the regime theory
60 m	
30 m	the relation between the flood discharge and river width studied in Japan

Then, the retarding volume of the Lahar Pocket for flood control is estimated.

(1) Retardation in river channel

The following table shows the 10-year flood peak discharge and retarded volume in channel on the Ngobo river for the river width of 90 m, 60 m, and 30 m.

I t e m	River width (m)		
	90	60	30
Flood Peak Discharge (m^3/sec)	160	170	200
Retarded Volume (m^3)	0.82	0.43	0.13

Assuming that the present river width is expressed by the regime theory, the decrease of river width by diking causes the increase of flood peak discharge. Therefore, it is required to keep the river width estimated by the regime theory or present inundated width during flood in case of no flood prevention facility.

(2) Retardation effects in lahar pocket

Usually, the sabo facility has no retarding effect for flood control However, the Lahar pocket constructed around the Mt. Kelud is expected to retard the flood discharge considering the width of levee to levee more than about 500 m.

The Lahar pocket in the Ngobo river basin has the control volume of 2.912×10^6 m^3. The vacant volume above the spillway crest is estimated to be 20 % of the control volume assuming that the height of dam and spillway crest are 6 m and 5 m respectively. The vacant volume which is calculated to be 0.58×10^6 (2.912×10^6 $m^3 \times 0.2$), is considered to be effective for the retardation of flood discharge. This volume is corresponding to the retarded volume in river channel of about 70 m river width from the above table.

Consequently, the river width should be decided based on the examination of the retarding effect/volume for flood control in case which the lahar pocket will be constructed.

9. Project Evaluation

(1) Economic project cost and behefit

(a) Economic project cost

The economic project cost including costs of engineering, administration and physical contingency of the proposed flood control plan consisting of the flood diversion channel in the main Brantas and the river channel improvement with artificial retarding basins in the Widas basin is estimated at Rp. 161,200 x 10^6 as shown below.

Project cost of the proposed plan

(Rp. 10^6)

K. Brantas (Flood diversion channel)	K. Widas	Total
85,000	76,200	161,200

The breakdown and the adopted conditions of the above cost estimates are presented in Table 3.5.15.

(b) Flood control benefit

The direct and indirect benefits are measured as reduction of the direct and indirect flood damages estimated in Section 3.5.3. With existing flood control facilities, the annual average flood damages are expected to be Rp. 3,422 x 10^6 in the main Brantas and Rp. 8,202 x 10^6 in the Widas basin as shown in Table 3.5.7 at the present development conditions.

With implementation of the proposed flood control works against the 50-yr flood in the main Brantas and the 25-yr flood in the Widas basin, the expected annual average flood damage will be reduced to Rp. 1,761 x 10^6 in the main Brantas and Rp. 513 x 10^6 in the Widas basin as shown in Table 3.5.7. Then, the annual reduction of flood damage is Rp. 1,661 x 10^6 in the Brantas and Rp. 7,689 x 10^6 in the Widas basin.

As shown in the retarding basin scheme in the Widas basin, the present natural retarding basins of 3,000 ha in total will be modified as the artificial retarding basins of 2,000 ha. Then, the balance of

1,000 ha can be used for productive purposes. Assuming that the areas to be free from habitual inundation will be used as paddy field, the annual land enhancement benefit is estimated at Rp. 721,000/ha.

The total expected annual flood control benefits are estimated as follows;

(Rp. 10^6)

Basin development level	K. Brantas	K. Widas
Present (1984)	8,900	8,200
Future (2000)	17,500	16,400
(2035)	46,800	39,300

(2). Evaluation of the proposed flood control projects

Based on the abovementioned cost and benefit, economic internal rate of return (EIRR) of the proposed plan is calculated assuming a project life of 50 years under the basic year of 1984. The EIRR of the case that flood control projects in the main Brantas and the Widas basin are assessed as one project (combined) is estimated at 8.2 % under the present development level and at 15.0 % under the future one which considered annual increases of properties for each sector, respectively. The results are summarized below.

Estimated EIRR of the project

(Rp. 10^6)

Basin development level	EIRR (%)		
	K. Brantas (Flood diversion channel)	K. Widas	Combined
Present	7.9	8.4	8.2
Future	15.7	14.5	15.0

The net present value and benefit-cost ratio at a discount rate of 12 % is worked out below.

Estimated NPV and B/C

(Rp. 10^6)

Basin development level	(Flood diversion)		K. Widas		Combined	
	B-C	B/C	B-C	B/C	B-C	B/C
Present	-10,300	0.62	-14,500	0.66	-24,800	0.64
Future	13,600	1.50	13,700	1.32	27,300	1.39

3.6 Watershed Management

3.6.1 Introduction

Watershed management (land and water conversation in the water - shed area) is one of the important aspects of water resources development. Sediment load brought from the watershed area heightens the river bed and decreases storage capacities of reservoirs. Although it is impossible to make the sediment production nil, it is necessary to control the sediment production within an allowable range. Since there are many river structures such as dams, reservoirs, diversion canals and improved river channels in the Brantas basin, watershed management shall be considered with attention.

In this context, objectives of the Study on watershed management are set as;

- identification of problem areas from the viewpoint of sediment production based on available data and reports
- clarification of the present conditions in the problem areas including on-going conservation works there,
- identification of possible countermeasures, when needed
- preparation of action plan for future watershed management.

Due to limitation of study time, clarification of the present conditions and identification of possible countermeasures are made based on available data and reports, and remains in the preliminary stage of study. Main forcus is placed on the action plan for future watershed management.

3.6.2 Problem Areas from Viewpoint of Watershed Management

1. Erodible Areas

Mechanism of sediment production and transport is complexed one. Among various factors, soil and ground surface slop are factors explaining erodible areas. Erodible areas in the basin were examined by the Agrarian Office during the period from 1972 to 1976, and the said office prepared the soil capability maps "Kemampuan Tanah" in a scale of 1 to 50,000. indicating the erodible areas. From these maps, the erodible areas are picked up as shown on Fig. 3.6.1, and estimated in area as shown below;

ERODIBLE AREA IN THE BRANTAS RIVER BASIN

		Sub-basin area (km^2)	Erodible area (%)	Erodible area in sub-basin (%)	Erodible area in total basin (%)
1.	Upper Brantas river basin from Sengguruh	1,034	108.48	10.49	11.11
2.	Lesti river basin	625	128.66	20.59	11.99
3.	Mt. Kelud basin	2,003	10.84	0.54	1.01
4.	Upper Konto river basin from Selorejo	238	18.45	7.75	1.72
5.	Ngrowo river basin	1,600	518.36	32.40	48.33
6.	Widas river basin	1,539	40.42	2.63	3.77
7.	Other basin	4,761	247.47	5.20	23.07
	T o t a l	11,800	1,072.68	9.09	100.00

Source : Soil Capability Map 1976

2. Problems Areas

The most extensive erodible area in the basin can be seen in the Ngrowo river sub-basin, which occupies about 48 % of the total erodible area in the Brantas basin. At present, a drainage system consisting of the Parit Agung canal and the South Tulungagung tunnel No. 2, is under construction. Protection of the sytem from sedimentation is needed. However, there are no data and no reports on the watershed management in this area. Therefore, study on this area is to be made in future upon accumulation of basic data.

Mt. Kelud area (the southern and western area of Mt. Kelud) has only 10.8 km^2 of erodible area or 1% of the total erodible area, according to the Agrarian Office's classification. This may come from rather coarce soil texture covering the area. However, Mt. Kelud eruption and have loose accumulation of erupted materials. Special consideration would be needed to this area.

The Karangkates lahor reservoir and Selorejo reservoir are important storage researvoirs in the basin, and elongation of the reservoir life is highly desireable.

Taking into account the necessity of watershed management from economical, ecological, and social viewponts, the following areas are taken up as the objective areas for the watershed management study;

(1) Upper Brantas and Lesti basin
(2) Mt. Kelud area
(3) Upper Konto from Selorejo dam

It is recommended that other areas than these taken up in this study must be studied in due time with progress of developments in those areas.

3.6.3 Upper Brantas and K. Lesti area

1. Characteristics of the basin

The Karangkates reservoir is located at the downstream end of the upper Brantas which forms narrow gorges of its stream with mountainous watershed. The Lesti river is the major tributary of the upper Brantas. The Brantas drains a large area of cultivated land which rises up to the forested slopes of Mt. Arjuno and Mt. Semeru. The land near Malang and its surrounding is irrigated and well cultivated with paddy and sugarcane. On the higher slopes above the irrigation area, crops of maize, vegetables and cassava are grown interspersed with estate crops. On the highest slope areas, natural vegetation, tropical forest can be seen.

The Brantas river is joined at Sengguruh with the Lesti river which drains an area of approximately 620 km^2 to the east. The Lesti river transports so much sediment which is produced by surface erosion in its basin.

2. Land use

Land use in the Lesti basin was surveyed by the Ministry of Agriculture, Ministry of Forestry and their agencies. Much of the basin upstream of Karangkates dam is under perenial cultivation. The area where the slopes are too steep and the topsoils are too thin to remain as the natural forest. The land use in the upper Brantas and the Lesti tiver basin is as follows.

Land use	Upper Brantas Area (km^2)	Upper Brantas Percent (%)	Lesti sub-basin Area (km^2)	Lesti sub-basin Percent (%)
Paddy field	410.25	20.0	103.60	16.6
Upland field	721.83	35.2	255.00	48.8
Plantation	81.51	4.0	80.62	12.90
Forest area	578.74	28.2	126.47	20.3
(Natural)			(81.69)	(13.1)
(Reforest)			(44.78)	(7.2)
Waste land	3.32	0.2		
Homestead/Settlement	235.58	11.5	58.14	9.3
Other use	18.77	0.9	1.17	0.2
T o t a l	2,050.00	100.0	625.00	100.0

Source ; WS-1 and Land Use Map, 1984

The most significant land use in terms of sediment production is the upland field which crops maize, cassava, potatoes, etc. These crops are, as the whole, poorly cultivated with inadequate terracing and negligible contour ploughing. Contour ploughing is not favoured as this increases the retention of water and causes field water logging during the wet season, resulting in the destruction of crops such as potatoes. By speeding up the flow of water from these fields, the farmers are aiding the high rate of soil removal.

3. Erodible area in the Lesti river basin

According to the land use map by Agrarian Office, the erodible area in the Lesti river basin is located around the Mt. Semeru slope under 1000 m in elevation. The boundary between the erodible area and unerodible area of the lower land is about EL. 500 m along the Lesti river about EL.750 m along the Mt. Genteng. The erodible area in the Lesti river basin is shown in Fig. 3.6.2

Land use within the erodible area in the Lesti river is as follows:

Land Use in Erodible Area in Lesti Basin

Land use	Area (km^2)	Percent (%)
Upland field	79.4	61.7
Plantation (Estate)	21.3	16.6
Reforest area	12.1	9.4
Houstead/settlement	7.7	6.0
Natural forest	4.9	3.8
Paddy field	3.3	2.5
Total	128.7	100.0

Note; Refer to Fig WS-2 in ANNEX.

As seen above, about 88% of total erodible area is occupied by upland field, estate plantation and reforest area.

4. Reforestation

According to the information of Perum Perhutani Unit II Jawa Timur, refoestation in the Lesti river basin has been carried out since 1969 under REPELITA I, II and III and will be further continued under REPELITA IV. The completed reforest up to 1983 is said to be 870 ha and the scheduled under REPELITA IV is 520 ha. as shown in Table WS 1. Pterospermum SPP and Sweetnia Mahagani are major species of trees in reforest area which occupies about 53% and 21% of total area respectively as shown in Table WS-2.

5. Soil conservation

Several demonstration plots of erosion control have been carried out 2 to 4 years ago to examine soil conservation. The plot is especially making terraces at sloped areas. Interim results are referred to as shown in NOTE-1 in ANENX WS.

The above investigation resulted in the following.

- Rainfall intensity and amout are usually high in the plot area. Cultivated land should not be retained uncultivated if monthly rainfall is extremely high.

- Terracing technically diminishes erosion rate. The erosion rate of terraced soil is less than half compared with that of unterraced soil.

- Mixed crops system is able to reduce erosion rate.

6. Sediment yield

Sediment yield in the upper Brantas basin (upstream from Karangkates damsite) has been estimated from the sedimentation survey in the Karangkates reservoir, and sediment discharge measurement in the Brantas upstream and the Lesti river as follows.

(1) Sedimentation in Karangaktes reservoir

Reservoir sedimentation survey has been carried out three times in 1977, 1980 and 1982 since 1972 upon commencement of water impounding in the Karangkates reservoir. Sedimentation survey in 1977 was carried out by HRS, who concluded the annual sedimentation during 4 years from 1973 to 1977 was 6.99×10^6 m^3. However, the said results were finally judged erroneous due to inaccurate observation. The sedimentation thus finally concluded is summarized below.

Reservoir	Catchment Area (km^2)	Estimated Period	Annual Deposit x $10^3 m^3$	Annual Yield Rate mm/year
Karangkates	2,050	1977 - 1980	1,600	0.78
		1977 - 1982	2,045	1.11
		1981 - 1983	1,426	0.77

From the above table, annual sedimentation in the Karangkates reservoir is estimated at about 1.0 $mm/yr/km^2$.

(2) Sengguruh project site

This project locates just in the downstream of confluence of the Brantas river with Lesti river. The catchment area at Sengguruh is 1.659 km^2. Annual sediment yield at Sengguruh site has been estimated at 2.26 x 10^6 m^3 based on the sediment measurements in the Lesti river and the upper Brantas as follows.

	Annual suspended load	Annual bed load	(10^6 m^3/y) Total
Brantas sub-basin	0.82	0.065	0.89
Lesti sub-basin	1.34	0.031	1.37
Total	2.16	0.096	2.26

7. Basic concept of watershed management

(1) Objective of watershed management

The principal need of watershed management in the Lesti basin and the upper Brantas is to prevent the Karangkates reservoir from the sedimentation. After completion of the Sengguruh dam, this reservoir will become the main objective to be protected against the sedimentation. Hereunder, preliminary plan is presented.

(2) Design condition of Sengguruh dam and reservoir

According to the design report of Sengguruh dam project, the life time of Sengguruh reservoir is about 20 years unless otherwise upstream sabo works are not carried out. General features of the Sengguruh reservoir are summarized below.

Basin Area	1,659 km^2
High water level	EL. 292.5 m
Low water level	EL. 291.4 m
Gross storage capacity at H.W.L.	$21.5 \times 10^6 m^3$
Effective storage capacity	$2.5 \times 10^6 m^3$
Average runoff	55.2 m^3/s
Sediment space	$19.0 \times 10^6 m^3$

(3) Required annual sediment inflow reduction

To secure 50 years life time of Sengguruh reservoir, allowable sedimentation in the reservoir is to be 0.38×10^6 m^3 per year on average or 19.0×10^6 m^3 for 50 years. Annual sediment inflow into the reservoir is estimated at 2.26×10^6 m^3 as presented previously. Assuming the average reservoir trap efficiency of 0.45, allowable sediment inflow is 0.84×10^6 m^3/yr. The balance of 1.42×10^6 m^3 (2.26 - 0.84) must be arrested in the upper Brantas river and Lesti river area.

The above figures are summarized below.

Annual sediment inflow	2.26×10^6 m^3/yr.
Total sediment inflow during 50 years	113.0×10^6 m^3
Allowable sedimentation during 50 years	19.0×10^6 m^3
Trap efficiency assumed	0.45
Allowable sediment inflow during 50 years	42.2×10^6 m^3 or 0.84×10^6 m^3/yr
Sediment inflow to be reduced during 50 years	70.6×10^6 m^3 or 1.4×10^6 m^3

As shown above, a target of watershed management in the upper Brantas and Lesti river is to reduce sediment inflow into the Sengguruh reservoir from 2.26×10^6 m^3/yr to 0.84×10^6 m^3/yr or from 113.0×10^6 m^3 to 42.2×10^6 m^3 for 50 years.

(4) Countermesures

To attain the above objective to arrest sediment production and/or sediment inflow into the reservoir, Sabo works, reforestation and terracing are to be considered.

According to the sediment discharge measurement at Clumprit and grain size analysis, grain size on the 50% passing has been estimated at approximately 0.3 mm for suspended load and at approximately 0.9 mm for bed load. The gradation of sediment discharge is shown in Fig. WS-4 and ANNEX WS.

From these gradation curves, it can be said that the grain size of the suspended load occupying about 90% of sediment load is so small that sabo dam or check dam with small storage capacity will not reduce the flow velocity sufficient for the small particles to deposit in the sabo dam and will not be able to control the sediment in the required volume, if the sabo dams are located in the upper reaches where the stream bed slope is steep. Therefore, the sabo dam sites are located not just downstream of the erodible area but in the area where the stream bed slope becomes rather gentle, and the capacity of storage is limited to big ones.

The conceivable nine Sabo damsites which can retain sediment volume more than one million m^3 are selected on the basis of 1/5,000 topo-maps. These selected damsites are shown in Fig. 3.6.3. Riverbed profiles along Lesti river and Upper Brantas river together with location of the proposed sabo damsite are as shown in Fig. 3.6.4. Sediment area of each Sabo dam is assumed as the space between the backsand formation level of 2 to 3 of the original riverbed slope and the original riverbed as shown in Fig. WS-5 in ANNEX WS Based on the sediment area assumed, the sediment volume of the proposed sabo dams is estimated as shown in Table 3.6.4.

Typical design of sabo dam is assumed as shown in Fig. WS-6. Relationships between the crest length of sabo dam and construction cost are estimated as shown in Fig. WS-7. Applying these relationships, construction cost of each proposed sabo dam is estimated as shown in Table 3.6.2. Summary is as shown below with its estimated cost.

Sabo damsites	Sedimentation capacity ($10^6 m^3$)	Construction cost (Rp 10^9)	Unit cost per cubic meter sedimentation (Rp/m^3)
I. Upper Brantas			
1. Brantas - 1	3.6	3.15	875
2. Brantas - 2	2.2	3.35	1,523
3. Brantas - 3	5.7	3.79	665
II. Lesti river			
4. Lesti - 2	4.4	5.67	1,289
5. Lesti - 2	3.0	5.67	1,890
6. Lesti - 3	1.4	3.15	2,250
7. Genteng - 1	4.7	4.19	891
8. Genteng - 2	1.2	4.19	2,917
9. Genteng - 3	1.1	2.80	2,545
III. Lesti III dam /1 dead storage	6.0	-	-
Total	27.3 (33.3) /2	35.27	1,292

/1 Dead storage of Lesti III dam under construction is taken into account for sedimentation capacity.

/2 Including Lesti III dam

As seen above, conceivable maximum sedimentation capacity would be 33.3×10^6 m^3 which is less by 37.3×10^6 m^3 than the required sediment reduction of 70.6×10^6 m^3.

As explained in previous paragraph 4, reforestation and terracing would bring the erosion control effect. To attain the sediment control to reduce sediment yield by 0.75×10^6 m^3/yr corresponding to 37.3×10^6 m^3 for 50 years, reforestation and terracing are recommended.

When sediment yield from non-erodible area is assumed at 0.5 mm/km^2/yr which is normal production rate in non-erodible area, sediment yield from erodible area can be estimated at 6.5 mm/km^2/yr. This implies that major part of sediment is produced in the erodible area. Experimental plots show the sediment reduction rate of more than 0.5. When this value is taken, the required reforestation/terracing area is calculated as follows.

			Upper Brantas	Lesti Basin	Total
(1)	Catchment area	km^2	1,034.0	625.0	1,659.0
(2)	Erodible area	km^2	108.5	128.7	237.2
(3)	Non-erodible area (1) - (2)	km^2	925.5	496.3	1,421.8
(4)	Sediment production				
(5)	Before reforestation				
(6)	Erodible area (2) x 6.5mm	$10^6 m^3/y$	0.705	0.837	) 2.253
(7)	Non-erodible area (3)x0.5mm	$10^6 m^3/y$	0.463	0.248	
(8)	After reforestation				
(9)	Erodible area (2x6.5x0.5)	$10^6 m^3/y$	0.353	0.419	) 1.483
(10)	Non-erodible area	$10^6 m^3/y$	0.463	0.248	
(11)	Balance (16)+(7)-(19)+(10)	$10^6 m^3/y$	0.352	0.418	0.770

Based on the above figure, total possible reduction of sediment yield is estimated at 0.77 x 10^6 m^3/yr or 38.5 x 10^6 m^3 for 50 years, which is almost the same amount as 37.3 x 10^6 m^3 of additionally required sediment yield reduction after the Sabo dam construction.

As the conclusion, it is needed to implement Sabo works as well as the reforestation in both the upper Brantas and Lesti river areas.

(5) Sequence of Implementation

The above is the total requirement of watershed management works in the upper Brantas basin. In order to keep the sediment level below LWL at the end of the 50 year, the following sequence of implementation is needed; (Details are shown in ANNEX WS)

Reforestation in the erodible area and terracing of the critical land.

Upper Brantas .... from 1st to 7th year

Lesti river .... from 8th to 15th year

Sabo dams

Upper Brantas

1. Brantas - 1 to be completed by 2nd year
2. Brantas - 2 6th year
3. Brantas - 3 4th year

Lesti river

4. Lesti - 1 3rd year

5. Lesti - 2 7th year

6. Lesti - 3 11th year

7. Genteng - 1 9th year

8. Genteng - 2 15th year

9. Genteng - 3 13th year

A yearly disbursement schedule based on the above requirement is assumed as follows;

Unit: Rp 10^6

Year	Sabo facilities	Reforestation	Total
1st		500	500
2nd	1,575	500	2,075
3rd	4,410	500	4,910
4th	4,730	500	5,230
5th	1,895	500	2,395
6th	1,675	500	2,175
7th	4,510	500	5,010
8th	2,835	500	3,335
9th	2,095	500	2,595
10th	2,095	500	2,595
11th	1,575	500	2,075
12th	1,575	500	2,075
13th	1,400	500	1,900
14th	1,400	500	1,900
15th	1,750	500	1,900
16th	1,750	500	2,250

3.6.4 Mt. Kelud area

1. Volcanic Activity of Mt. Kelud

Mt. Kelud is an active volcano located in the centre of the Brantas basin. According to the record as shown in Table WS-3, Mt. Kelud erupted ten times in the last one and a half centuries starting with the eruption in 1811 up to the lastest eruption in 1966. The intervals of the eruption are between 3 and 37 years and 15.5 years on an average. It was reported that one eruption extruded volcanic materials amounting to 100 to 300 million m^3. Most of the erupted materials rushed down as lahar to the plain area mainly in the west and south of Mt. Kelud. Considerable amount of those materials were brought into the Brantas river, and caused the rise of river bed especially during several years after the eruption as shown in Fig. 3.6.5.

The area suffering from such lahar (hereinafter called as "lahar area") covers about 2,003 km^2 including the basin of the Semut, Putih, Badak, Ngobo and Konto rivers, the tributaries of the Brantas river joining between 46 km and 210 km point from its rivermouth. Whereas the remaining basin of the Brantas river excluding the lahar area is recognized as unaffected area of about 10,000 km^2.

The crater of Mt. Kelud retains the rain water and some additional water from springs along the crater lake. The retained water rushed down the crater slopes together with the volcanic efflux as hot and muddy masses at the eruption. This is called the "Primary Lahar".

The primary lahar rushes down the valley of the tributaries originated in the crater and spreads at the foot of the hilly areas, about 500m in the elevation. The distribution is as shown in Fig. 3.6.6. This primary lahar is very destructive due to its high velocity and hot mass. The extent of disaster caused by the primary lahar seems to relate directly to volume of the lake water. To minimize the disaster due to the primary lahar, the construction of tunnel to drain the crater water to the western slope of Mr. Kelud was initiated in 1919. The lake water decreased from 40 million m^3 to 2 million m^3 due to lowering of water level in the lake by a drain tunnel constructed during 1923 - 1928 period. No serious damage due to the primary lahar occurred in the 1951 eruption. The tunnel was damaged by the eruption in 1951. According to the records, on the eruption in 1951, the lake bottom has been lowered by about 70 m and the capacity of retaining water much increased up to 20 million m^3. Before effective measure had been worked out for decreasing the lake water, the eruption occurred in 1966. In this eruption, though the amount of erupted material was less than half of that in the former eruption, serious damage by the primary lahar was caused. On the eruption in 1966, the tunnel was damaged again. However, the lake bottom rose by about 50 m.

The rehabilitation and re-construction works of the tunnel have been repeated and present volume of lake water is estimated at 4.3 million m^3. If this volume is maintained untill the next eruption, distance travelled by primary lahar is estimated at 13.1 km.

Aside from the primary lahar, the volcanic efflux such as ash, sand, and volcanic bombs deposit on the hilly slopes. They flow down as muddy flow in the ravines together with rain water. This mud flow is called the "secondary lahar".

During the eruption, the other special phenomenon is found. Higly heated and gas-charged lava with temperature as high as 900°C is ejected and flows down several kilometers with extremely high velocity, that is called "nuee ardente". The deposit as a result of this phenomenon is called "ladu" and consists of scorice lapilli, ashes and debris of lava. The area destroyed and scorched by nuee ardente reaches as far as 10 km around the crater of Mt. Kelud.

The acreage of lahar is 2,003 km^3, in which hilly area is 190 km^2 and plain area is 1,813 km^2. All the area is covered with the volcanic materials consisting chiefly of lapilli, sand and ash. The boundary between hilly area and plain area is 500 - 600 m in elevation. In the hilly area, ground slope is 5° - 6° near the boundary and 25° - 45° near the summit of Mt. Kelud.

Lahar area is divided into 11 area divisions as shown in Fig. 3.6.6. Acreage of hilly and plain area in each area division is shown in Table 3.6.3.

Out of tributaries of Brantas river, which are flowing down from the lahar area, 10 rivers such as Lekso, Semut, Putih, Badak, Gedok, Petungkobong, Sukorejo, Ngobo, Serinjing and Konto rivers are selected and their slope variation along the longitudinal profile are shown in Table 3.6.4. On this table, it is learned that the river bed slope at around the boundary of hilly and plain areas, 500 - 600 m in elevation is 1/15 - 1/25.

The mean annual rainfall in lahar area is 2,224 mm, and among the area, much rainfall occurs in divisions I to III, located at southern part of Mt. Kelud. The basin mean monthly rainfalls estimated for respective 11 areas are shown in Table 3.6.5.

2. Land Use

According to the land use map prepared by Agrarian Office, the land use in the lahar area is estimated as follows.

Land use	Area (km^2)	Percent area (%)
Paddy field	1,072.3	53.5
Upland field	384.4	19.2
Plantation (Estate)	127.5	6.4
Mixed plantation	8.8	0.4
Forest	220.0	11.0
Natural	(156.9)	(7.8)
Reforest	(63.1)	(3.2)
Homestead/settlement	182.3	9.1
Waste land	7.7	0.4
Total	2,003.0	100.0

Note; Refer to Fig. WS -7.

As explained in 3.6.1, erodible area identified in the Mt. Kelud area is about 11 km^2 or only 1.0 % of total erodible area in the Brantas river basin. In terms of watershed management, the Mt. Kelud area is districtly different from other areas. Debris control to prevent disaster due to the erupted materials is the main subject in Mt. Kelud area.

3. Sediment yield from Lahar area

The sediment yield from both the lahar area and unaffected area has been estimated based on the sediment balance study taking into account the sediment discharge at the end of river channel, sediment deposit in the river and sediment inflow from the upper end of river channel as follows.

(10^6 m^3)

	Total erupted material	Sediment brought to the Brantas from Lahar area		
		5 years after eruption	Subsequent 10 years	Total
1951 eruption	192	30.8 (16%)	24.9 (13%)	55.7 (29.1%)
1966 eruption	90	20.8 (23%)		

On the other hand, according to the examination made by GOI after the 1966 eruption, the amount of volcanic material dirived from ladu, primary and secondary lahars to the main 5 rivers in lahar area such as Semut, Putih, Badak, Ngobo and Konto river during the period from the eruption to the end of 1966 has been estimated as shown in Table 3.6.6. It is learned from this table that the total amount brought to these 5 rivers in 57.03 million m^3 or 63% of the total amount of erupted material.

As the total amount of erupted material in the 1966 eruption is reported to be 90 million m^3, the remaining amount of volcanic material being 32.97 million m^3 (= 9 - 57.03) is considered to be spread by falling on the wide area around the Mt. Kelud.

4. Present basic concept for sediment balance in lahar area

At present, there are two sediment distribution plans; one was prepared by the Brantas Middle Reach River Improvement Project, and the other by Mt. Kelud project. Between two plans, there are some differences in assumption, but consulution on the amount to be controlled by sabo facilities is almost same. The following summarized reference is made following concepts of the Middle Reach River Improvement Project, and more detailed reference is as shown in ANNEX WS.

Applying the aformentioned proportion to the model pattern (one eruption in every 15 years, 200 x 10^6 m^3 in one eruption), the amount of volcanic material brough to the lahar area divided into main 5 river basins and that spread by falling have been estimated at 130 x 10^6 m^3 and 70 x 10^6 m^3 respectively. They are distributed proportionally to the lahar area as shown in Table 3.6.7. Based on the table above mentioned and the sediment yield from lahar area, the sediment balance in the lahar area has been clculated as shown in Table 3.6.8. The realizable amount of sediment brought from the lahar area to the Brantas river has been estimated considering the present condition of the debouch of the tributaries and increase in sediment tractive force due to channel work. The sediment balance in the lahar area during the modelled 15 year period are estimated with division into 5 major tributaries, and the excess sediment to be controlled by the sabo facilities is estimated at 64.6 x 10^6 x 10 m^3 as shown in Fig. 3.6.7.

The Mt.Kelud project takes little different assumptions and reaches a conclusion that the required sabo facilties control the excess sediment of 66 x 10 m^6, as shown in Fig. 3.6.8.

5. Review of present basic concept of sediment balance in Lahar area

Although the basic concepts mentioned in the above do not state dynamic riverbed movement clearly in relation to the sediment inflow from the lahar area, it seems that both concepts assumes static riverbed movement based on the average inflow of sediment and the average sediment discharge capacity of the Brantas river. However, according to the experience of one of the Team members at inspection of the conditions after the 1966 eruption, the following different views rise ;

(1) The present concepts assumes that all the lahar area of 2,003 km^2 is controllable by the sabo facilities. However, the sabo facilities are located in the downstream from the facilities. The eruption materials fallen in such area could not be controlled.

(2) The present concept does not clarify rise of riverbed in the Brantas river after eruption and subsequent lowering. According to the riverbed movement of the Brantas river, the riverbed rose in the first 5 years and gradually lowered. The amount of sediment to raise the riverbed is estimated far bigger than the sediment discharge capacities of the tributaries in the lahar area assumed by the present concept. It is considered that such large sediment inflow into the Brantas river was caused not only by the flow in the tributaries but also by overland flow. As mentioned in the above, there is the large uncontrollable area in the lahar area. Therefore, it is considered that riverbed rise after eruption would be unavoidable and that it is necessary to examine the sediment balance taking into acoount the riverbed rise.

Fig. 3.6.9 shows the balance of sediment deposit in the river channel of the Brantas river. In the first 5 years, the infow of sediment exceeds the sediment discharge of the Brantas river, but in the later 10 years, the sediment discharge capacity exceeds the amount of normal sediment inflow. Then, the excess volume of sediment depcsit in the first 5 years shall be equal to the excess sediment discharge capacity in the later 10 years.

In a view of the above, an alternative concept of sediment balance in the lahar area is examined based on the data on the Middle Reaches Project and Mt. Kelud Project. Details of the alternative concept are as follows ;

(1) The former lahar area is divided into two areas; sabo area and non-sabo area.

(2) The eruption materials are divided into ladu, primary lahar and falling materials. The amounts of ladu and falling materials are assumed to change in proportion to total amount of the erupted materials. However, since the amount of the primary lahar is considered to relate not to the total amount, but to the amount of the crater water, the amount is assumed to be same as that in the 1966 eruption.

(3) The ladu and primary lahar are assumed to enter into only the sabo area. The falling materials are assumed to spread over the sabo,non-sabo and non-lahar areas. Two thirds of the falling materials are assumed to fall in areas within 50 km from the crater and the remaining in areas outside of 25 km circle.

(4) From the sediment discharge capacity of Brantas river, bed and suspended loads and wash load are estimate by BRBDEO as follows:

Bed and suspended load	23.70 MCM in 15 years
Was load	69.00 MCM in 15 years

In the BRBDEO's report, the sediment loads from the non-lahar area is estimated as follows :

Bed and suspended load	9.90 MCM in 15 years
Was load	48.88 MCM in 15 years

(5) Under the normal condition, sediment yields from the sabo and non-sabo areas are 130 m^3/km^2/year of bed and suspended loads and 750 m^3/km^2/year of wash load as same as the non-lahar area.

(6) As shown on Fig. 3.6.9, the excess sediment discharge capacity of Brantas under the normal condition, is 0.66 MCM/year. If the normal condition continues for 10 years, the total excess capacity is 6.60 MCM. This excess capacity will scour the riverbed. From a point of riverbed stability, allowable amount of temporary deposit in the river channel must be equivalent to

6.60 MCM. It is assumed that sediment of 9.90 MCM (60.60 + 3.30) will flow out into Brantas river from the non-sabo area during the first five years after eruption. (This assumption is drawn from the river bed movement after 1966 eruption. Then, the total sediment yield from the non-sabo area is calculated at 29.09 MCM (2.84 + 16.36 + 9.90).

(7) Balance between the falling on and the flowing out from the non-sabo and non-lahar areas is considered as amount remaining in those areas.

(8) Of the total eruption materials of 74.92 MCM in the sabo area, the ladu is assumed to remain in place. Then, the amount flowing down to the deposit zones is estimated at 48.68 MCM (74.92 - 26.24). The sediment yield from the sabo area is to be controlled as near normal condition by the sabo facilities, and is estimated at 6.91 MCM. The amount to be retained in the sabo facilities becomes 41.77 MCM (48.68 - 6.91).

(9) Flow of the eruption materials according to the above assumption is as shown on Fig. 3.6.10.

(10) Since the amount of 6.60 MCM is assumed to come from the non-sabo area, temporary rise of riverbed will be unavoidable. The riverbed will rise about 0.66 m on an average over the river stretches of 100 km long, and more than 1 m locally. This phenomena was shown after the 1951 and 1966 eruptions.

Although the above examination is based on many assumptions, it is considered that the newly presented balance shows one of possible cases that could give more sever condition to the river stability. In conservative view from river stability, it is recommended that this newly presented balance be adopted for the planning of the Brantas river flood control, and further that the sediment balance being currently adopted by Mt. Kelud Project be used for sabo planning from also its conservative presumptions for sabo requirement.

6. On-going Mt. Kelud project

The Mt. Kelud Volconic Debris Control project was established under DGWRD in 1969, for controlling of debris over an area of 2,003 km^2, covering the sub-basins of Lekso, Putih, Badak, Ngobo and Konto river with the basic arrangement of facilities as shown on Fig. 3.6.11.

This project is a long-term project, and up to now, the following structures are completed and going to be implemented in REPELITA IV.

Structures	Overall Plan	No.	Completed Storage Capacity	REPELITA IV No.
Sabo dam	21	4	$1.33 \times 10^6 m^3$	2
Check dam	40	25	1.28	1
Ground sill	140	29	0.72	16
Drempie	19	-	-	2
Lahar pocket	12	9	15.58	2
Total			19.41	

The above structures are planned based on the sediment control plan as shown on Fig. 3.6.8. Design storage capacity of the existing lahar pockets and other structures is $19.4 \times 10^6 m^3$, and the retained volume in these pockets and structures to-date is $14.5 \times 10^6 m^3$ as shown in Table 3.6.9. Then, the present remaining storage capacity is $4.9 \times 10^6 m^3$. Assuming that the slope of back sand formation is to be two thirds of the original riverbed slope, the temporary control capacity between the present sand surface and the assumed back sand line in estimated at $25.6 \times 10^6 m^3$ as shown in Table 3.6.9. Then, the combined capacity is $30.4 \times 10^6 m^3$.

This project is everlasting one until Mt. Kelud rests.

6. Countermeasures against the next eruption

Although the average cycle time of eruption is 15 years, 18 years have already passed without eruption since the last eruption in 1966. Then, it can be said that the next eruption may occur at any time in future. Therefore, it is necessary to prepare countermeasures against the next eruption in advance. Required countermesaures are examined hereunder.

Based on the eruption material balance (Fig. 3.6.10), the amount to be controlled by the sabo dacilities is assumed at $41.77 \times 10^6 m^3$, if the next eruption is in a scale of modelled one. Since it is considered that it will take at least 3 years for survey, design, financial arrangement land aquisition and construction of new sabo facilities, there will be no additional control capacity against the next eruption in the first three years. Then it is necessary to prepare the additional capacity of sand pocket before the next eruption. According to the inflow of sediment in the Brantas river after the 1966 eruption, it is assumed that 85% of the sediment to be controlled by sabo facilities flows down in the first 3 years. The volume to be controlled in the first 3 years is $35.5 \times 10^6 m^3$ ($41.77 \times 10^6 m^3 \times 85$). The present combined control capacity is $30.4 \times 10^6 m^3$. The balance of $5.1 \times 10^6 m^3$ ($35.5 \times 10^6 - 30.4 \times 10^6$) is to be met by countermeasures against the next eruption as follows ;

River	Required additional capacity (10^6 m^3)
Ngobo	1.3
Dermo - Sukorejo	2.4
Gedok	0.3
Badak	0.8
Lekso	1.6
Total	6.4

3.6.5 Upper Konto River Basin

1. Present Conditions

The upper Konto river basin is defined as the area upstream of the Selorejo dam, and has a catchment area of 238 km^2, which consists of Konto river of 142 km^2, Kwayangan river of 53 km^2 and Pinjal river of 43 km^2. The ground elevation at the damsite is around EL. 580 m, and the area higher than EL. 1,000 m occupying two thirds of the basin.

The basin receives rainfall amounting to 2,340 mm annually on an average. The rainfall occurs concentratedly in the rainy season from November till next April, as shown in Table 3.6.10.

Several land use surveys have been carried out in the basin. The land use is mapped out summarily as shown in Fig. WS-8 and Table WS-6. Based on this map, the land use condition is estimated as follows;

Paddy field	1,794 ha	7.5 %
Upland field	4,500 ha	19.1 %
Forest	15,400 ha	64.9 %
Homestead/settlement	1,268 ha	5.3 %
Miscellaneous	758 ha	3.2 %

The paddy field extend along Kwayangan river and Pinjal river, the major upland field exists in the Pujon Plateau. The forest area consists of the natural forest of 12,975 ha and the reforest area of 2,465 ha.

2. Sedimentation

(1) Sediment Deposit in Selorejo Reservoir

The sediment production in the upper Konto river basin can be known from the reservoir sedimentation in Selorejo reservoir. Reservoir sedimentation survey was firstly carried out in 1977 and has been continued since 1982. By comparing corresponding cross sections obtained by sounding survey in 1977 and 1982 , the sediment deposit in this period is estimated by Hydraulic Research Station, UK as follows;

Sub-basin	Catchment Area (Km^2)	Annual Sediment Volume ($x\ 10^6\ m^3$ / yr)
Konto river arm	185	0.175
Kwayangan river arm	53	0.057
Konto Total	238	0.232

Form the above figures and assuming the trap efficiency of the reservoir of 90 %, the annual sediment yield rate is estimated at 1.08 mm for the catchment area of 238 km^2.

(2) Source of sediment

Fig. 3.6.12 shows the soil erodibility (refer to Note in ANNEX) in the upper Konto river basin. According to this figure, the area with the higher erodibility is seen on the left bank of Konto river and along its tributaries. The paddy fields and upland fields are classified as areas with moderate or less erodibility.

Since there is no specific sediment production area such as sliding zone, the main source of sediment materials is presumed as the top soil erosion. Rainfall with high intensity falling on the ground surface with poor vegetation cover erodes the top soil and the rain water suspends the eroded soil. The small streams with steep riverbed slope transport the sediment materials to the tributaries and the tributaries transport them to Konto, Kwayangan and Pinjal rivers.

Major source of sediment material is the cultivated areas developed by clearance of the jungle within the last 30 years due to population pressure. These areas are developed without proper terracing and planted mainly with upland crops such as cassava, maize, brassicas and onions owing to the unavailability of irrigation water. All these crops leave the large proportion of the planted area uncovered by their leaves. Such uncovered land is very weak against the direct attack of the rain drop.

In contrast with the above, the streams drain out from the natural forest are quite clean. However, such clean streams change to brown in color after passing the upland fields in only few hundred meters.

3. Past and On-going Watershed Management Works

(1) Sabo Facilities

Existing sabo facilities for preventing the Selorejo reservoir from the sedimentation have been constructed by BRBDEO since 1973. The sabo facilities are 1 sabo dam (Tokol sabo dam) and 30 check dams, and cover 3 tributaries of Konto, Kwayangan, and Pinjal rivers. These are located 1 sabo dam and 25 check dams on Konto river stream, 4 check dams on Kwayangan river stream, and 1 check dam on Pinjal river stream. The amount of the existing sediment volume is approximately 1.2×10^6 m^3, of which 0.95×10^6 m^3 in Konto river 0.09×10^6 m^3 in Kwayangan river and 0.002×10^6 m^3 in Pinjal river, respectively. The location and the storage capacity of the existing sabo facilities are as shown in Fig. WS-9 and in Table WS-7 respectively.

(2) Reforestation

As shown in Table WS-31, the forest area occupies the total area of 15,400 ha. Of this, 2,465 ha is the reforest area. The management of the area is divided into four sub-areas; Kawi Utara, Kawi Barat, Kelud-Luksongo and Anjasmoro, as shown in Table WS-9. In the area with the high erodibility, reforestation is carried out over the area of 878 ha. Details are shown in Table WS-8 and Fig. WS-11. The species applied for reforestation are Pinus merkusii for pulp wood (30 %) and Calliandra calothyrsus for fuel wood (21%). Details are shown in Table WS-9.

(3) Soil Conservation had been carried out at Ngroto village, Kecamatan Pujon in the Konto river and Jombok village, Kecamatan Ngantang in Kwayangan river. Location of them are as shown in Fig. 3.6.15.

In each location, 7 plots with different treatment were prepared and the soil erodibility and rain erodibility were observed. Based on the observation, the soil erosion and possible soil conservation measures are reported as quoted hereunder.

(a) The factor values of the soil erodibility of the lands along the areas of Selorejo reservoir stream range around 0.1 up to 0.4. One of the factors that influence the soil erodibility value is the organic matters content on the upper layer. If the organic matters content is high like in the upper lands the soil erodibility is low. Most of the lands in Selorejo stream areas have less than 0.2 erodibility value. The research results show that the lost of soil from the demonstration plots during the 16 week-observations could reach 1.5 - 1.7 mm.

(b) The test in both locations (Ngantang & Pujon) shows that the plants to cover lands that are combined with the land cultivation could much decrease erosion. Coffee plants have the plant and cultivation factor value (CP factor) as much as

0.01. Annual crops that are planted individually (monoculture) could decrease erosion with the CP factor value as much as 0.28 (for peanut), 0.40 (for corn) and 0.45 (for cassava). If these crops are mixed and planted the erosion could be reduced and become lower, so that the CP factor for mixed plant(corn + peanuts = 0.15) and (corn + cassava = 0.25.)

(c) Terracing could reduce erosion to some extent. But these tests showed a fact that on the lands already terraced, erosion still occurred. Besides terracing, it is necessary to pay attention to the form of land cultivation for decreasing erosion further. According to the test in Ngroto, where the land cultivation was done to make the ditches and ridges in parallel to the contour line, it was found that this form of cultivation was effective to control soil erosion, and further that this form of cultivation decreased the surface runoff coefficient to 25%, while that in Jombok was over 60%.

4. Future Watershed Management

In the upper Konto river basin, the sediment problem shall be examined in relation with the dead storage capacity in the Selorejo reservoir.

The designed dead storage capacity in the Selorejo reservoir for the economic life time of 50 years is 12.2×10^6 m^3 below the low water level of EL. 598 m. The annual sediment deposit in the Selorejo reservoir is estimated at $0.232 \times 10^6 m^3$. From these figures, it can be said that the dead storage capacity is sufficient for sediment inflow for 50 years.

In this context, it will not be necessary to take immediate measures for watershed management. However, it is recommended to continue reforestation and soil conservation works, in order to reduce top soil erosion as much as possible.

3.7 Electric Power Development

3.7.1 Introduction

Hydropower potential in the Brantas basin is one of the important water resources, since it has advantages of unexhaused and rapid responding source of energy. Objectives of the study on the electric power development are;

- clarification of present supply and demand conditions
- examination of future demand
- examination of possibility of hydropower development in the basin

However, due to the topographic condition, there are few sites which can be developed for single purpose of hydropower. Possibility for hydropower developmnet is examined in connection with dam development in the next section and results of the dam development study are reffered to.

3.7.2 Present Conditions

1 Power Supply System

The Esat Java power system has 13 major power stations and 74 substations linked with each other through 150 kV trunk transmission lines, 70 kV subtrunk lines and 30/25 kV sub-transmission line as shown in Fig. 3.7.1 and their particulars are as follows:

(1) Power Plant

9 - hydropower plant	210.5 MW (33%)
2 - oil-fired steam power plant	350.0 MW (56%)
2 - gas turbine power plants	67.5 MW (11%)
T o t a l	628.0 MW

(2) Transmission & Sub-transmission lines (Fig. 3.7.1)

150 kV transmission line	841 km (1,052 km-cct)
70 kV transmission line	758 km (1,361 km-cct)
30/25 kV sub-transmission line	310 km (510 km-cct)
T o t a l	1,919 km (2,923 km-cct)

(3) Substation transformer capacity

150 kV transformers	767 MVA
70 kV transformers	846 MVA
30/25 kV transformers except pole transformer	64 MVA
T o t a l	1,677 MVA

Details of the above are shown in Table EP-1,2 and 3.

Diesel power plants working in remote areas are going to be relocated to other inlands or decommissioned. The total installed capacity in the East Java system shares about 26% of that in all Java system. The total capacity of substations' transformers shows about 50% increase since October 1981. The East Java power network has been interconnected at 150 kV level with the Central and West Java systems.

2. Past Power Demand

The past power demand data is available from 1975/76 to 1982/83 as PLN operation statistics for each sub-system of Java. As summarized in Table 3.7.1, the energy sales in East Java sub-system was recorded at 1,798 GWh in 1982/83 with more than 25% annual increase for recent 5 years, against 20% annual increase for the same period in all Java.

Power consumption in 1982/83 is classified as follows; (Details are shown in Table EP-4).

	Energy Sales	Consumption	Average Annual Growth Rate
(a) Residential	641 GWh	35.7%	19.2%
(b) Commercial	232 GWh	12.9%	23.1%
(c) Industry	863 GWh	48.0%	34.3%
(d) Public	62 GWh	3.4%	1.8%
Total	1,798 GWh	100.0%	25.4%

As can be seen in the above figures, almost half of energy sales is occupied by the industrial demand whose annual growth rate in average for recent 5 years is the heightest. The captive power in this field was 408 MVA, and 123 MVA, only was connected to PLN system in 1980 as shown in Table 3.7.2. Besides, there are many waiting big customers amounting to 190 MVA as of 1982/83 in East Java as shown in EP - 5. The residential and commercial demands occupy another half of the energy consumption.

The per capita power consumption in East Java in 1982/83 is calculated at 60 kWh per annum which is still low consumption level.

3. Load Curve

The latest daily load curve in the East Java system is shown in Fig. 3.7.2. The daily load factors are about 76% both in working day and holiday. Comparing the working-day load curve with the holiday load curve, the difference of daytime loads may suggest industrial and commercial demand. The load curves are almost similar to the historic daily load curves before interconnection to Java system, as can be seen in Fig. 3.7.3. Annual load factor is estimated at 70% in 1983/84.

According to the latest load curves in early September, 1984, peaking time varies from 4.1 to 5.2 hours in weekday in East Java Sub-system, while peaking time under all Java interconnected system ranges from 4.4 to 5.3 hours. Most of peaking hours which have been adopted at 5 hours for hydropower plants in Brantas river basin development project seems reasonable in the present system scale.

4 Power Loss

The power losses including station use were improved from 28% in 1977/78 to 23% in 1982/83 in the Esat Java sub-system.

5 Average Tariff of Electricity

The average electric tariff in whole Indonesia was calculated at Rp. 56.8 per kWH in 1982/83 and Rp. 74.1 per kWh in the first quater of 1983/84. Details are shown in Table EP-6.

6 Organization

Organization of PLN in East Java is as shown in Fig. 3.7.4. PLN Generation and Transmission East Java Region is responsible for overall management, operation and maintenance of all power stations, 150 kV and 70 kV tarnsmission lines and substation in East Java. PLN Distribution in Esat Java is responsible for management, operation and maintenance of 20 kV, 6 kV and low tension lines including energy sales. The load dispatching centre (hereinafter called LDC) for all Java System has been organized at Waru near Surabaya and the Waru LDC is instructing all stations belonging to the Esat Java sub-system accordingly.

3.7.3 Needs of Development

1 Future Power Demand

Fig. 3.7.5 shows the load demand forecast for all Java system and East Java sub-system. Transition of the installed capacity of the power plants are also shown in Fig. 3.7.5 including those of the on-going and committed projects.

2 On-going and Proposed Project

(1) Power Plant

The followning projects are on-going, committed, proposed or studied;

	Name of Project	Type	Installed Capacity (MW)	Year of Commissioning Status	Remarks
On-going or Committed					
(i)	Gresik No.3 & No.4	Gas	42	1984/85	
(ii)	Gresik No. 3	Steam	200	1986/87	
(iii)	Sengguruh No.1 & No. 2	Hydro	29	1986/87	
(iv)	Gresik No. 4	Steam	200	1987/88	
	Sub Total		471		
Proposed					
(i)	Paiton No. 1	Steam	400	1988/89	-
(ii)	Paiton No. 2	Steam	400	1989/90	-
(iii)	Tulungagung	Hydro	30	1989/90	Under D/D REP EP-09
(iv)	Wonorejo	Hydro	13	1991/92	Waiting for appraisal by ADB
(v)	Paiton No. 3	Steam	400	1990/91	-
(vi)	Paiton No. 4	Steam	400	1992/93	-
	Sub-Total		1,643		
Studied					
(i)	Kesamben	Hydro	32		F/S completed
(ii)	Lesti III	Hydro	12.5		D/D completed but review needed
(iii)	Kepanjen	Hydro	30		Under Study
(iv)	Karangkates No. 4 & No. 5	Hydro	100		Study Report completed
	Sub-Total		174.5		
	T o t a l		2,275.5		

Kesamben Project is required to be reviewed at least in cost of dam foundation treatment due to the very poor geological condition.

Lesti III and Kepanjen projects which are under study by BRBDEO are also required to be reviewed in all aspects.

Additional installation of 2 units in the Karangkates project utilizing the existing reservoir water is to be reviewed in terms of peaking power operation under some restriction for responsible water-release for irrigation and water supply. Especially, deoendable output of additional units has to be reviewed in view of available inflow.

(2) Sub-Station and Transmission Facilities

In REPELITA IV, substation equipment is to be expanded by about 1,400 MVA including the porposed facilities as shown below :

		Committed	Proposed
(i)	150 kV transformers	917 MVA	340 MVA
(ii)	70 kV transformers	120 MVA	30 MVA
	T o t a l	1,037 MVA	370 MVA

Details are shown in Table EP-7

Expansion of the transmission lines are planned and committed in REPELITA IV as shown below :

(i)	150 kV transmission line	745 km (853 km-cct)
(ii)	70 kV transmission line	56 km (90 km-cct)
	T o t a l	810 km (943 km-cct)

Details are shown in Table EP-8.

In REPELITA IV, Madura and Bali power systems will be interconnected with the Esat Java network at 150 kV level.

In addition to the present 150 kV trunk line, EHV (500 kV) transmission system had been planned for interconnection in Java and its construction was already started in West Java according to the progress of Surabaya and Saguling Power Projects. Fig.3.7.6 shows an expansion plan of 500 kV transmission line in Java during REPELITA V up to the proposed Paiton thermal power station through Ungaran and Krian substations.

3 Future Peak Load

According to increase of the system scale in future, peak load portion will become acute in its scale and shortning of peaking time may be considered to some extent, say 4 hours especially after interconnected by 500 kV EHV transmission system.

The load demand forecast in Fig. 3.7.5 indicates 4,446 MW of peak demand in East Java in the year of 2003/04, and peak load portion will be about 1,000 MW (4,446 MW x (414.9 - 316.3) / (414.9) from Fig. 3.7.2), assuming load curve shape is similar to the present.

The installed capacity for peak load portion in Esat Java region will be about 380 MW as of 1989/90 and deficient by 620 MW as of 2003/04. While, interconnection by 500 kV EHV transmission line is scheduled in REPELITA V along with development of large scale hydropower plants in other regions.

4 Needs of Hydropower

The composition of power sources including on-going and planned projects is as shown below :

Plants	East Java (%)			All Java (%)		
	1983/84 /1	1988/89 /2	1993/94 /3	1983/84 /1	1988/89 /2	1993/94 /3
Hydro	33	22	10	19	28	28
Thermal (Oil)	56	68	27 } 86	51	31 } 57	19 } 63
Thermal (Coal)	-	-	59	-	26	44
Gas Turbine	11	10	4	29	13	8
Geo-Thermal	-	-	-	1	2	1
T o t a l (MW)	628	1,099	2,729	2,565	6,190	10,010

Note: /1 - Present status /2 - Including on-going project
/3 - Including planned projects.

Although the share of hydropower system will be high in the all Java system, in the East Java system it will decrease to about 10% in 1993/94. The planned EHV interconnection is scheduled to complete single circuit during REPELITA IV and another circuit upto REPELITA VII.
In view of low reliability of single circuit, it will be risky for the East Java system to rely on the hydropower in the Central and East Java. Therefore, from the viewpoint of sound system composition, the share of hydropower capacity is to be at least 10% of the total installation capacity. In this context, it is said that development of hydropower plant will be needed.

3.7.4 Project Formulation

1 Project relating to the Existing Power Plant

(1) 2 m Heightening of HWL in Karangkates Reservoir

There is an idea to heighten HWL by 2 m from El. 272.5 m to increase water storage in wet season and to utilize it in dry season. As for power station equipment it should be checked whether the existing facilities can be safely operated or not, due to 2 m heightening of HWL. As a result of review of the contract specification and commissioning test data, following are revealed:

Items	Influence of 2 m Heightening
- Pen stock and surge tank	Within design pressure
- Intake gates	Exceed allowable stress
Skin plate	by 2%
Roller	by 13%
Concrete shearing stress	by 18%
- Generating equipment	
Maximum output	Should not exceed 36 MW
Speed, voltage and pressure rize	Within granted value
- Servomotors and thrust bearing	Within granted value
- Cavitation	No adverse effect
- Efficiency.	Lower by 0.2% at 36 MW output

Note: Details are shown in Note EP-1

As shown in the above, except the intake gate, there will be no serious problem in WHL heightening by 2 m.

(2) Extension/Improvement of Konto River System

Mendalan and Siman power stations were constructed in 1931 and 1932 respectively, and their service time has been over 50 years. According to the result of investigation, both power plants were already re - placed with new parts in 1955.

Recently, spare parts such as turbine runners, wearing parts, etc. already procured for all units in both stations and some units were already overhauled with replacement of new runners and others except for generators. Other units are also scheduled to follow.

Operation and maintenance (O & M) cost in 1983/84 are summarized below.

	Mendalan (Rp. 10^6)	Siman (Rp. 10^6)
O & M cost	28.1	12.8
Repair cost /1	32.1	18.9 For overhaul
Personnel expense / 2	6.0	3.2
T o t a l	66.2	34.9

(Source : PLN KITLUR)

Note: /1 - Repair cost is annualized assuming that runner and wearing parts are replaced with new ones at an interval of 20 years.

/2 - Personnel expense is assumed at 10% of the sum of O & M and repair costs.

If the Mendalan and Siman power stations are reconstructed in the same installed capacities as the present ones, the construction cost will be Rp. 4,700 million for Mendalan and Rp. 2,180 million for Siman. Then, the above annual maintenance costs become 1.4% of the construction cost for Mendalan and 1.6% for Siman. These figures are a bit higher than 1% usually assumed for new plants, but still within an allowable range. Therefore, reconstruction of the Mendalan and Siman power stations within the lifetime of the replaced parts is not economically justifiable.

2 Pumped Storage Hydropower Plant

In the region of the East Java system, there remain low hydropower potential sites only owing to the topographic condition. Therefore, to cope with the future needs for peak power supply, one of possibility is development of pumped storage hydropower plant (hereinafter called as DSPP).

In case of PSPP, securing of large pumping power utilizing off-peak power supply from high efficiency thermal power plants is of prime importance. To sound the time thereof, rough assumption of PSPP is set up as follows:

- Output	: 300 MW
- Peak operation hour	: 4 - 6 hours
- Pumping hours	: more than 8 hours per day
- Pumping efficiency	: 0.7
- Transmission line loss	: 5%

Assuming that off-peak power below the average power can be used for additional pumping power and energy, a necessary system peak is:

300 MW/(1-0.05) x 100%/(76.2% - 72.7%) = 9,020 MW, say 9,000 MW.

The demand forecast in East Java shows 4,446 MW in 2003/04. It means that development of PSPP will be in the remote future. It is, however, recommended to make a detailed study in the latter half of the objective period to grasp the economical development scale of PSPP against the system scale together with installation of high efficiency and large-scale thermal power plants.

3 Development of Conventional Hydropower

As for conventional hydropower development, studies are made together with the dam development study. Studied schemes and their installation capacities are as follows:

Name of Scheme	Capacity (MW)	Annual Energy (GWh)
K. Genteng I HPS	18.6	54.9
K. Konto II HPS	62.0	207.4
Babadan HPS	9.4	28.1
Kuncir HPS	4.3	28.3
Kedungwarak HPS	0.7	3.0
Beng HPS	12.0	10.4
Lumbangsari HPS	10.8	46.9
Kepanjen HPS	6.0	32.5

3.7.5 Evaluation

The identified schemes will be evaluated in the dam development study in section 3.8 based on the alternative thermal concept and using the following capacity and energy values; Details are shown in Note EP-2.

kW value at 12% discount rate

	Capital Cost (US$/kW)	Service Life (Years)	Capital Recovery (%)	O & M Cost (%)	Annual Equiv. Cost (%)	Adjustment Factor (P.U)	kW Value (US$/kW)
Coal-400	1,080	25	12.75	2.0	14.75	1.252	199.4
Coal-600	970	25	12.75	2.0	14.75	1.252	179.1
Geo - 55	1,120	25	12.75	2.0	14.75	1.252	206.8
Oil -200	720	25	12.75	2.0	14.75	1.252	133.6
Gas -100	310	25	13.39	2.5	15.89	1.148	56.5

KWh Value

	Fuel Cost (US$/$10^6$ Kcal)	Thermal/1 Effifiency (%)	Heat Value (Kcal/ KWh)	Fuel Cost (US$/ KWh)	O & M Cost (US$/ KWh)	Adjustment Factor (P.U)	kW Value (US$/ KWh)
Coal-400	8.5	33	2,606	0.0222	0.0006	1.039	0.0237
Coal-600	8.5	33	2,606	0.0222	0.0006	1.039	0.0237
Geo - 55	-	-	-	-	-	-	0.0400/2
Oil -200	19.8	32	2,687.5	0.0532	0.0006	1.028	0.0553
Gas -100	24.2	24	3,583	0.0867	0.0003	1.007	0.0876

Note: /1 - Assumed plant factor is 50% for coal and oil and 15% for gas

/2 - Steam cost concluded between PLN and Pertamina (Ref.-03)

Since the crude oil is a limited resources, its real value will increase in future, even though the present world market price is stagnating. Hydropower scheme will be realized after 1990. Therefore, the energy value of oil-fired thermal plant and gas turbine plant are evaluated assuming an annual increase rate of real value of 3% for 15 years from now on, based on the price prospects by IBRD.

3.7.6 Results of Hydropower Development Study

Results of the hydropower development study made together with the dam study are as follows:

	Genteng I	Konto II	Babadan	Kuncir	Kedung-warak	Beng	Lumbang-sari	Kepan-jen
Capacity (MW)	18.6	62.0	9.4	4.3	0.7	12.0	10.8	6.0
Annual Energy (GWh)	54.9	207.4	28.1	28.3	3.0	10.4	46.9	32.5
Construction Cost (Rp. 10^6)	91,102	202,741	140,111	75,083	41,503	56,129	33,926	30,712
D a m & Pump	80,475	148,495	134,711	70,018	5,894	21,204		
Power Facility	10,627	54,246	5,400	5,065		13,706	33,926	30,712
Pump	-	-	-	-	35,609	21,219		
Annual Benefit (Rp. 10^6)	14,549	34,904	12,158	5,859	3,060	13,555	6,299	4,280
Water supply	7,032	6,320	8,400	2,250	5,400	15,000		
Hydropower	7,722	28,694	3,946	3,673	363	1,258	6,299	4,280
Negative /1	- 205	- 110	- 188	- 64	-2,703	-2,703		
EIRR of Power Component (%)	11.3	11.9	5.7	5.5	5.4	4.1	14.2	15.6

Note; Dam and pump costs are allocated according to magnitudes of benefifs of water supply and hydropower.

/1 Land cost is included in above projects except Lambangsari and Kepanjen.
Pump up cost is included only in Kedungwarak and Beng.

3.8 Dam Development

3.8.1 Introduction

In the basin, six dams exist and two dams are under construction. As for the existing dams, review is needed on reservoir lifetime with confirmation of available storage capacity, on reservoir operation for optimum use of storage capacity, and on spillway capacity for dam safety.

Rainfall in the basin is not so small, but it concentrates in the rainy seasons. River flow decreases in the dry seasons, and it becomes less than the present water demands in severe drought years, even if the existing storage capacities are fully used. To cope with the future water demands, it is necessitated to construct storage reservoir to shift the rainy season river flow to the dry season use.

In this context, dam development study covers the following aspects;

- review of the existing dams
- examination of potential dam sites from the viewpoints of water supply, hydropower generation, and others.

3.8.2 Review of Existing Dams and Reservoirs

1. Brief Features of Existing Dams

In the Brantas river basin, many water works had been constructed for flow control and irrigation water intake since long years ago. Major one was old Lengkong dam constructed in 1897 at about 4 km downstream from the Mojokerto city. However, dam development with large storage capacity had to wait until 1960's.

In 1961, an overall development plan of the Brantas river basin was formulated with emphasis on hydropower development and flood control, reflecting needs at that time. According to the overall plan, construction of the Karangkates and Selorejo dams were started in 1962 and 1963, respectively. Following two dams, five dams have been constructed up to now. Brief features of these dams are as follows:

Name of Dam	Year of completion	Purpose	Effective storage	Installed power
			$\times 10^6$ m³	MW
Karangkates	1973	F,S,P	253.0	70 (1st stage)
Lahor	1977	F,S,P	29.0	35 (2nd stage)
Wlingi	1977	P,S	5.2	54
Lodoyo	1980	P	4.2	4.5
Selorejo	1970	F,S,P	50.1	45
Bening	1981	F,S,P	33.5	0.65
Lengkong	1973	F,S		

Remarks : F - flood control
S - water supply, (Irrigation and domestic use)
P - Power generation

Details are as shown in MP-14.

Total effective storgae capacity excluding the daily regulating capacities in the Wlingi and Lodoyo, is 366 x 10^6 m^3, and it accounts only 3% of the average annual total discharge of about 12 x 10^9 m^3 from the entire basin.

2. Necessities of Review

There are arguments on the existing dams and reservoirs in terms of reservoir sedimentation, resevoir operation rules and spillway capacities. It is considered that clarification of these issues are the basis of the future development planning. Therefore, review is made for the following aspects.

(1) Reservoir life in the Karangkates, Lahor, Selorejo and Wlingi reservoirs in relation to reservoir sedimentation.

(2) Reservoir operation of the Karangkates - Lahor reservoir and Selorejo reservoir

(3) Spillway capacity of the Karangkates, Lahor, Selorejo and Wlingi reservoirs.

3. Reservoir Life in relation to reservoir sedimentation

The existing reservoirs are designed to have the dead storage to accommodate sediment deposit for 50 years at the sediment yield rate assumed at the design stage. If the actual sediment yield rate is bigger than the design rate, it means faster diminishing of the dead storage and decrease in the effective storage. Details of reservoir life study are presented in NOTE MP-1.

(1) Reservoir Sedimet Survey

The reservoir sedimetation survey was commenced since 1977 in the Karangkates reservoir, by using echo sounder. The following survey were made up to date;

Reservoir	Year					
	1977	1978	1980	1982	1983	1984
Karangkates	0		0	0	0	*
Lahor						*
Wlingi				0	0	*
Selorejo		0			0	
Bening						*

Note : 0 surveyed and data analysis completed
* surveyed but data analysis not yet

The sediment volume calculated by comparison of cross-sections obtained in the different years. Results are as follows;

Reservoir	Catchment Area (Km^2)	Estimated Period	Annual Deposit x $10^3 m^3$	Annual Yield Rate mm/year
Karangkates	2,050	1977-1980	1,600	0.78
		1977-1982	2,045	1.11
		1981-1983	1,426	0.77
Wlingi	680	Data incomplete		
	(sub-basin)			
Selorejo	238	1977-1982	0.232	0.98

From the above table, it can be said that the annual sediment yield rates in the Karangkates and Selorejo reservoirs are around 1 mm/year.

(2) Sediment Discharge

From the sediment discharge measurement record, the relationships between water discharge and sediment discharge are sought for, and the following equation are obtained;

At Blobo on Brantas river $Qs = 1.2 \times Q^{2.1}$

At Sumberejo on Lesti river $Qs = 1.2 \times Q^{2.4}$

At Metro on Metro river $Qs = 4.0 \times Q^{2.0}$

Where; Q : water discharge (m^3/s)

Qs : sediment discharge (ton/day)

By inputting the water discharge record during the period from 1951 t0 1983, the average annual sediment inflow into the Karangkates reservoir is calculated at 2,489 x 10^3 tons. It is equivalent to 1.1 mm/year and corresponds to the sediment yield rate of 1 mm with the trap efficiency of 90%.

(3) Reservoir Life

Based on the above sediment yield rate the sediment distribution in reservoir is examined by the modified empirical area-reduction method. The Karangkates and Lahor reservoirs are classified as US - Type 1 and US - Type II reservoir, respectively. Based on the original storage capacity curves, the present storage capacity curve is estimated as shows in Fig. 3.8.1. The effective storage capacity between EL. 246 m and EL. 272.5 m is given below;

	Effective Storage Capackty (MCM)		
	Karangkates	Lahor	Total
Original	253.0	29.4	282.4
1982	232.5	28.6	261.1
2000	195.6	27.6	223.2
After 50 years	150.5	25.4	175.9

As shown in the above, the effective storage capacity in the Karangkates - Lahor reservoir will be decreased by 37.9 MCM by the year 2000 from 1982 which is equivalent to 2.4 m^3/s of the dry season runoff.

The storage capacity curve thus estimated is adopted for estimation of inflow into the combined Karangkates - Lahor reservoir from the records of outflow through turbines and spillway and changes in the reservoir water level for the reservoir operation study.

The Selorejo reservoir is classified as US - Type II. The effective reservoir capacity is estimated by the same method, assuming the annual sediment yield rate of 1 mm. The result is as follows;

	Effective Storage Capacity (MCM)	
	EWL EL. 620 m Dec. - Apr. (Flood season)	HWL EL. 622 m May - Nov.
Original	46.5	54.6
1982	44.9	52.9
2000	42.4	49.5
After 50 years	39.4	46.5

The decrease in the effective storage capacity in the Selorejo reservoir is estimated at 3.4 MCM by the year 2000 from 1982. It can be said that this decrease will not be remarkable. However, it is necessary to monitor the sedimentation condition near the high water level from the viewpoint of safety against flood.

4. Operation of Karangkates - Lahor reservoir

Among the existing and on going reservoirs, the Wlingi, Lodoyo and Sengguruh reservoirs have the storage capacities limited to daily regulation of the inflow. The Selorejo and Bening reservoirs are assigned to irrigation water supply in the downstream of each, and there is no excess capacity to contribute the main Brantas. Therefore, the Karangkates - Lahor reservoir only can regulate the flow in the main Brantas.

According to water demand study, the potential water demand in the basin are forecasted to increase largely. Therefore, examination is made whether it is possible to create more water by modification of the operation of the Karangkates-Lahor reservoir.

(1) Role of Karangkates - Lahor Reservoir

The Karangkates - Lahor reservoir is a multipurpose reservoir for hydropower generation as well as water supply. When the dam was constructed, the role of the Karangkates power station was very important. At present, the power system has been expanded, and big thermal power plants are scheduled in the future. Therefore, the role of the Karangkates power station as energy source will become comparatively low. In this context, it is considered that operation of the Karangkates - Lahor reservoir can be modified to water supply oriented.

(2) Present Operation

According to BRBDEO, the reservoir has been operated based on the operation rule prepared in 1978 (1978-rule), which set the operational low water level at EL. 260 m at the end of November. In the normal and water rich years, the reservoir has been operated according to the 1978 -Rule. However, in the drought years, the reservoir was operated according to the water requirements in the middle and lower reaches of the main Brantas.

(3) Regulation Capacity

According to the reservoir life study, the present effective storage capacity is estimated at 261.1 MCM. The regulation capacity of the reservoir using this capacity fully is examined by mass curve analysis. The constant out-flow in the dry season from June to November regulated by the reservoir is estimated as follows;

Probability of draught year Constant out-flow	Regulated Out-flow in Dry season (m^3/s)
Once in 2 years	51.5
in 5 years	42.0
in 10 years	38.2
in 15 years	35.7
Minimum constant out-flow	34.8

The above shows the maximum capacity of the reservoir in the respective inflow condition. Therefore, a remaining measure to cope with the water demand is to separate the reservoir according to the pattern of the water demand.

(4) Operational Low Water Level

From the viewpoint of water balance in the basin, which is forecasted to become very tight in future, full utilization of the storage capacity is desirable. For example, the reservoir is to be operated so as to become empty at a certain time in a year, if onset of the rainy season is certain. However, onset of the rainy season sometimes delays by one or two months. Since the Karangkates - Lahor reservoir is a master in the basin, itis better to keep some storage for emergency cases. The required reserve capacity is estimated based on an assumption that the reservoir supplies all the water requirement in the downstream area in December. This requirement amount is estimated at about 26.8 MCM. Then, the operational low water level is set at EL. 205.5 m with reserve of 26.8 MCM above the designated IWL of EL. 246.0 m.

(5) Alternative Operation Rule

Water demand in the downstream area varies month by month according mainly to the variation of the irrigation water requirement. If such fluctuation is removed from the water demand, the demand shows almost constant value. According to the progress of the dry season, the river flow decrease gradually. Therefore, if the reservoir is operated to increase the outflow according to the decrease in the river flow, it will be beneficial to the downstream. From this consideration, a reservoir operation to keep a constant HxQ (reservoir water level multiplied by amount of outflow) is examined. According to the examination for the inflow with different probabilities, a rule curve of reservoir water level can be set as shown on Fig. 3.8.2. By applying this rule curve, the energy output of the Karangkates power station is examined. The results are shown in Table 3.8.1. As shown in Table 3.8.1 the reservoir operation by the HxQ constant method would not much affect the energy output. The out-flows of the actual operation and the HxQ constant operation is compared in the year 1982 as shown in Table 3.8.2. As shown in this Table, the out-flow of the actual operation gradually decreases according to the progress of the dry season, since the reservoir was operated in the first half of the dry season based on the expectation of normal inflow. On the contrary, the outflow by the HxQ constant operation shows gradual increase.

(6) Forecasting of Inflow

One of problems in reservoir operation is uncertainty of the inflow into the reservoir. From the rainfall and runoff records, relationships between rainfall amount in the rainy season and runoff in the subsequent dry season are examined, and a regression equation as shown in Table 3.8.3 is obtained with correlation coefficients larger than 85%. By collecting rainfall data and calculation by this equation, it will be possible to know the dry season runoff at the Karangkates damsite in advance. Then, all the water users such as Irrigation Service PLN and PDAM can discuss on allocation of available water including storage in the reservoir, and decide an operation pattern for that dry season.

(7) Recommendation

The above study on the reservoir operations is still preliminary one, but suggests a possibility to increase the outflow in the later part of the dry season witout affecting the energy output. Therefore, it is reccomended to carry out thorough study on this matter.

5. Spillway capacity

Spillways of the existing dams were designed according to the old design standards in Japan which were effective at that time, and based on the limitted hydrological data. In the recent years, introduction of much severe standards for spillway design becomes a world-wide trend, and many hydrological data have been accumulated after the design of the existing spillway. Therefore, preliminary checking of the capacity of the existing spillway is made for the Karangkates, Lahor, Selorėjo, and Wlingi dams, applying the newly floods estimated by the Storage Function method of which coefficients are determined in the Hydrological Analysis in Section 3.2.

For preliminary checking, two newly estimated floods are used; 200-yr probable flood x 1.2 and 10,000-yr probable flood. The former one is the current Japanese standard for fill-type dam, and the later is selected as representative of the abnormal condition. In case of 200-yr probable flood x 1.2, the Japanese standard for free board is applied, and in case of 10,000-yr probable flood, a free board of 1.0 m is applied. If the spillway is gated type, an allowance of 0.5 m is added. The results of preliminary checking are as summarized in Table 3.8.4. Details are shown in NOTE MP-2.

Since checking of the spillway capacity in this time is very preliminary, it can not be said that the existing spillways of Karangkates, Lahor, Selorejo and Wlingi dams have sufficient capacities against the newly estimated probable floods, if the severe standards are applied. Therefore, it is reccomended to carry out detailed study on this matter, particularly on the following points:

- rainfall analysis based on data as long as available, since the daily rainfall records are available since the end of 19 century.
- flood flow analysis based on at least 5 flood hydrographs.

The cost estimates in the Master Plan do not include costs to be needed for modification of the existing spillways.

6. Heightening of H.W.L. of Karangkates Reservoir

The top elevation of the impervious core of the Karangkates dam is EL. 278.5 m below 0.5 m from dam crest elevation and the crest elevation of the spillway is EL. 272.5 m. This 6 m allowance was prepared for surcharge at the flood time plus wave run-up and particular allowance of 1 m due to the fill type dam.

In view of the overall future water demand in the Brantas basin, and additional storage of about 30 x 10^6 m^3 is obtainable by setting the operation high water level at EL. 274.5 m at the end of the rainy season. Hightening maybe made by installation of a fabric dam over the spillway crest. It may be said that this provision is able to meet the immedaite additional water requirement in the basin, provided that the heightening of H.W.L. by 2 m causes no affection to the stability of the embankment and operation of spillway.

Since this idea includes lots of uncertainties which need detailed analysis and study to be clarified, no further examination including cost estimate is made in this study. The Master Plan does not list up this scheme.

To this end, the team's comment on this subject is that hightening of H.W.L. by 2 m or so shall be reviewed in all aspects of dam embankment stability, spillway operation with fabric dams, etc. in relation with the review of spillway capacity mentioned in the previous subsection.

7. Dam Projects under Study by BRBDEO

Several dam and hydropower projects are under study. From the upstream, they are peaking of the Karangkates power station, Kesamben dam and hydropower project and dam projects in the Trenggalek area.

Peaking of Karangkates Power Station

According to the study report on this subject, installation of additional two units of 50 MW is intended. If all the requlated out-flow in the dry season is used for peak power generation for 4 hours a day, the available discharge for peak operation will be as follows;

Drought year	24 hr Constant outflow	Available discharge for 4 hours peak operation
Once in 2 years	51.5 m^3/s	309.0 m^3/s
in 5 years	42.0	252.0
in 10 years	38.2	229.2
in 15 years	35.7	214.2
Minimum	34.8	208.8

The water requirement of the existing units of 105 MW in total for 4 hours peaking operation is as follows;

Water Level	Possible Output	Max. Discharge of existing units
EL. 272.5	105 MW	132.76 m^3/s
270	105	136.76
265 (Rated)	105	148.34
260	98.2	150.33
255	88.3	143.98
250	77.7	137.68
246	69.4	132.62

From the viewpoint of hydrology and in the sence of firm peak power, the maximum addition will be;

$$105 \text{ MW } (229.2 - 150,33) / 150.33 \doteqdot 55 \text{ MW}$$

Since the energy output is same under with and without the additional units, the additional units shall be assessed only by capacity value. Careful examination is needed.

Kesamben Dam and Hydropower Project

Unfavorable geological conditions in the damsite, which consist mainly of volcanic ash, gravel and clay still need intensive and extensive examination of foundation treatment, before making any decision on this project.

Dam Projects in Trenggalek Area

The Tugu dam is under study together with several alternative dams such as Bagong, and Kampak. These dams have the storage efficiencies as low as marginal or below marginal and small storage capacities which can meet only the local needs. In the study of the overall water balance in the Brantas basin, it will not be necessary to take into account the storage capacities of these dams.

As examined in the Agriculture and Irrigation Study, the Trenggalek area has the low cropping intensity of paddy and is now depending on cassava and maize. From the viewpoint of regional equity, some investment onto to Trenggalek area will be needed, even if the Tugu dam scheme is less economical. Therefore, the Master Plan includes a cost of Rp. 40,029 million by the name of Tugu dam.

3.8.3 Dam Development Study

1. Development Concept

(1) Needs of water resources development

The water resources in the Brantas basin have been developed mainly for irrigation water supply and hydropower generation. The presently available water in dry seasons has already been allocated to the various water users, and there is no remaining possibility of creating water within the basin or of transfering water from other basin. On the other hand, the future water demand for the domestic and industrial sectors, which are late comers to the water allocation, is projected to grow very rapidly for supporting the social and economic activities in future. Water is of vital material not only for human but also industries.

In this context, it is considered that there will be strong needs for the water resources development, particularly for the domestic and industrial water supply in the basin. There are several alternative methods to develop the water resources. If the natural conditions are favourable, the water resources development by dam and reservoir is promising.

(2) Approach to study

The water resources in the basin have been developed since decades ago, and the remaining sites suitable for development is few. Therefore, approach to study takes not only conventional type development method, but also special type development method such as carry-over type storage and inter-seasonal pumped storage. When inclusion of hydropower development in a scheme seems feasible, hydropower development is studied as a component of such scheme.

2. Potential damsites and preliminary screening

The 1973 Master Plan identified 33 schemes within the Brantas basin from the viewpoint only from the hydropower potential. Including these 33 potential sites, all the potential damsites are assessed according to the following criteria.

(1) Preliminary Design Criteria

For the first review and planning of dam development, the following study criteria (assumption) are applied;

- Study shall be based on the recent topographic maps
- Although geological information is limited, dam height possible topographically is examined, since higher dam generally gives higher storage efficiency.
- Since the geological conditions in the Brantas river basin are not so good in general , fill type dam is assumed. If dam height is lower than or equal to 30 m, earth-fill type with

upstream slope of 1 to 3.0 and downstream slope of 1 to 2.5 is assumed. In case dam height is higher than 20 m, rockfill type dam with upstream slope of 1 to 3.0 and downstream slope of 1 to 2.5 is assumed.

- In comparison with dam efficiency, a free board of 4 m above HWL and removal of top soil up to 5 m deep from the surface are assumed for calculation of embankment volume.
- For calculation of hydropower generation, the following are assumed:

installed capacity	:	5-hour peaking capacity using the 90% dependable discharge in case of run-of-river type 5-hour peaking capacity using average in case of storage type.
annual energy	:	plant factor of 50% for run-of-river type and 100% for storage type.

(2) First Screening Criteria

Dams are classified into two groups, one is storage type and the other is channel type which can create reservoir only within the river channel.

For the storage type dam, criteria of storage efficiency (effective storage/embankment volume) is applied. According to the Wonorejo Dam and irrigation project which has recent cost and benefit studies, unit cost of embankment is estimated at about Rp. 10,000/m^3 and unit benefit of water is estimated at Rp. 75 m^3 when water is used for irrigation. Then, if a discount rate is 10% the break-even storage efficiency becomes 14.7 (Rp. 10,000/Rp. 75/0.110859). Then, the dam with the storage efficiency larger than or equal to 15 is classifed as "A", between 10 and 15 as "B", and less than 10 as "C". Class A dam will be selected basically for further studies, unless other needs or reason exist.

The geology of damsite, where the channel type dam is conceived, is composed of the rock in the riverbed and the alluvial deposits in the both banks. This geology does not permit construction of diversion tunnel and spillway in the abutments. Therefore, the channel type dam will consist of costly gated spillway to be constructed on the riverbed and fill dam on the both sides of the spillway. This dam can contribute mainly to power generation by the head to be created by the dam, since effective storage is very limited. Then, ranking of this type is made by power generation efficiency, which is calculated by annual energy divided by rock-equivalent embankment volume. The unit of power generation efficiency is kWh/m^3. Then, dam with the power generation efficiency larger than or equal to 300 kWh/m^3 is classified as "A", between 150 and 300 kWh/m^3 as "B" and less than 150 kWh/m^3 as "C".

(3) Dams preliminarily Studied

According to the criteria and assumptions above mentioned, damsites are studied on 1 to 2,500 scale maps or 1 to 5,000 scale maps, where available, or 1 to 50,000 scale maps.

Out of 33 damsites, Bening dam in Widas river basin (formerly named as "Widas No. 2") has been completed and two dams; Sengguruh and Wonorejo (formerly named as "Song river" is a part of the Wonorejo project) are under construction. For the Kesamben dam, a feasibility study and its review have been made. Tugu dam (formerly named as "Pinggir") is under pre-feasibility study together with dams on Tawing river, Sukowetan river and Bagong river in the Ngrowo basin. These damsites are excluded from the preliminary review.

The following damsites are reviewed;

Storage Type Dam

Brantas river	Kali Lanang
Amprong river	Lojing, Tumpang
Genteng river	Genteng I
Ngrowo river	Klotok
Konto river	K.Konto I, II & III
Widas river	Kuncir, Kedungwarak, Semantok, Babadan
Beng river	Beng

Runoff Type Dam

Brantas river	Malang, Tambaksari, Lumbangsari, Blobo, Kepanjen
Jilu river	Jilu I, II & III
Amprong river	Amprong
Bango river	Bango
Lesti river	Lesti I, II, III & IV
Genteng river	Genteng II
Metro river	Metro

Location of the above damsites is as shown on Fig. 3.8.3

(4) Results of First Screening

Based on the available topographic maps, catchment area storage capacity and embankment volume of each dam is estimated. Storage capacity of each dam is set at the topographically maximum point. Results are as shown in Table 3.8.4.

According to the estimated features and screening criteria, dams are classified as follows;

Storage Type

Class	Sites	Site	Effective storage
Class "A":	4 sites	Genteng I	70 x 10^6 m^3
		Kedungwarak	55.9
		Semantok	40
		Beng	150
		Sub-total	315.9
Class "B"	2 sites	Lojin	17
		Babadan	89.7
		Sub-total	106.7
Class "C"	7 sites	Kali Lanang	42.9
		Tumpang	25.6
		K.Konto I	16
		K.Konto II	43.5
		K.Konto III	114.5
		Kuncir	47
		K.Klotok	31.2
		Sub-total	320.7
		Total	743.3

Channel Type

Class	Sites	Site	Installed capacity
Class "A"	4 sites	Lumbangsari	12.5 MW
		Kepanjen	8.3
		Lesti III	5.4
		Jilu II	1.1
		Sub-total	27.3
Class "B"	2 sites	Tambaksari	8.4
		Lesti IV	6.3
		Sub-total	14.7
Class "C"	10 sites	Malang	6.7
		Blobo	7.2
		Jilu II	1.0
		Jilu III	1.5
		Amprong	3.4
		Bango	1.1
		Lesti I	0.9
		Lesti II	2.2
		Genteng II	15.3
		Metro	-
		Sub-total	45.6
		Total	87.6

In addition to the above, the following are taken into considerations;

(a) The Konto river II damsite, although its storage efficiency is low, has a large hydropower potential which can compensate the low storage efficiency

(b) Although the Babadan and Kuncir damsites have low storage efficiencies, these damsites are important for the Widas basin development.

Then, the following damsites are selected for the further study.

Storage type	:	Genteng I, Konto river II, Kedungwarak, Semantok, Beng, Babadan and Kuncir.
Channel type	:	Lumbangsari, Kepanjen and Lesti III.

3. Criteria and Conditions For Further Study

For the further study of the selected dam schemes, the following criteria and conditions are introduced;

(1) Hydrological Analysis

The results of the past analysis, as far as those are available, are utilized.

For the other sites, the low flow is estimated by the Tank Model and the flood flow is estimated by the Nakayasu's Unit Hydrograph.

(2) Hydraulic Design

- diversion systems	25 years probable flood
- spillway	10,000 years probable flood with retarding effect in reservoir
- waterway	standard flow velocity of 3.5 m/sec.

(3) Structural Layout

- Dam type

For the dam lower than 30 m and/or for the damsites where rock materials is expected not to be available, earth-fill type dam is applied. In other case, rock-fill type dam is selected.

- Spillway

Fill type dam; in principle, non-gated spillway, gated spillway; at least, 3 gates with emergency spillway.

(4) Quantity Estimate

In principle, sum of the mean cross-sectional area multiplied by the unit distance between sections.

(5) Unit Cost

Unit costs used for calculation of construction cost are determined referring to the unit costs estimated for the Wonorejo dam, Cibuni dam, etc. which have been recently studied by several agencies.

UNIT COST

Item	Unit	Unit Cost (10^3 Rp)
Excavation		
earch	(m^3)	3.5
rock	(m^3)	7.5
tunnel (Dia ≧ 5 m)	(m^3)	43.4
tunnel (Dia < 5 m)	(m^3)	65.1
Embankment		
earth	(m^3)	4.4
core	(m^3)	5.5
filter	(m^3)	4.8
rock	(m^3)	4.8
random rock	(m^3)	4.2
Concrete		
open	(m^3)	94.6
tunnel	(m^3)	124.4
Reinforcement bar	(ton)	609.8
Steel support	(ton)	653.3
Grouting	(m)	72.0
New road	(Km)	275,000
Road improvement	(Km)	34,000
Gate, trashrack, etc.	(ton)	5,150.0
Steel penstock	(ton)	2,884.0
Steel pipe incl. installation		
Ø 1800 mm	(m)	1,093.8
Ø 2000 mm	(m)	1,289.4

(6) Provisions

Besides the construction costs estimated from the work quantities and unit construction costs, the following provisions are made for;

Miscellaneous of civil works	;	5% of the sum of the estimated costs
Preparatory works of civil works	;	8% of the sum of the estimated costs including miscellaneous cost
Engineering services cost	;	10% of the direct construction cost
Administration cost	;	5% of the direct construction cost
Base cost	;	Sum of the direct construction cost, engineering services cost and adminis-tration cost
Physical contingency	;	15% of the base cost

(7) Benefit

Benefits to be attributable to the dam schemes are assessed as balance between without-the-project and with-the-project conditions.

(a) Domestic and Industrial Water Supply

Valuation of the domestic and industrial water supply is not easy, since the value of drinking water is uncountable and the productivity of industrial water has the wide range of variation. In this study, the net cost of water is estimated as follows.

- The net cost of water for the irrigation sector is estimated as shown in Table 3.8.6 as the balance of primary profits with the irrigation (irrigated paddy) and without irrigation (non-irrigated polowijo). The net cost of water thus estimated is Rp. $70/m^3$.

- For the domestic and industrial water, umblan spring development project, which is a prospective candidate of water supply project, is taken up for the basis of water cost estimation. As shown in Table 3.8.6 raw water cost by the Umblan is estimated at Rp. $198/m^3$. The Umblan project can supply water without its treatment, while projects which do not need treatment could not be found in the Brantas Basin. Therefore, comparable/competitive project to the Umblan is assessed by the unit water cost Rp. 100 less the treatment cost.

(b) Hydropower Generation

According to the electric power development study, the benefits of hydropower generation are assumed as follows taking it into account that the hydropower will share the peak part of the power demand.

Capacity value	Rp. 58.2 x 10^3/kW
Energy Value	Rp. 121/kWh, taking into account increase of real value of fuel oil.

(c) Flood Control

Dams can contribute to the flood by retarding the floods in the reservoirs.

(d) Sediment Control

If a dam is located in the upstream of the existing storage reservoir, such dam can contribute to elongation of the life-time of the downstream reservoir. The dead storage capacity of the upstream reservoir is valued at Rp. 100/m^3.

(e) Negative Benefit

As negative benefit of the schemes, the value of land to be submerged by reservoirs, and energy to be used for pumping up of water into the reservoir are taken into account.

Land;	paddy field ; Rp. 1 x 10^6/ha/year other use ; Rp. 0.5 x 10^6/ha/year
Pump - up	Since it is considered that the pump-up will be made continuously in the rainy season, the power to be consumed is assumed as the base power. Then, Capacity value; Rp. 205.4 x 10^3/kW Energy ; Rp. 24 / kWh.

4. Results of Dam Study

(1) Genteng I Scheme

(a) Natural condition

The Genteng I damsite is selected on the Genteng river, a tribuatry of the Lesti river, about 2 km southeast of Dampit city. The catchment area at the damsite is 98.7 km^2. The damsite has formed gorge and its geology consists of volcanic breccia with intersecting sand stone according to the test boring carried out by BRBDEO. Three tributaries of the Lesti river are running along the Genteng river Topographically, it is possible to divert water from these tributaries; the Manjing river, located in the western side of the Genteng river, the Gransil river through the Juwok river, which are located in the northern side (See Fig. 3.8.4.).

(b) Development plan

Taking the natural condition into account, this scheme is envisaged as a storage reservoir with hydropower development. Transbasin plan is proposed to develop the water resources at possible maximum extent from the topographic viewpoint. By this transbasin plan, the catchment area is increased to be 160 km^2 and run-off is expected to be about 10 m^3/sec to 7 m^3/sec in the rainy season and 5 m^3/sec to 8 m^3/sec in the dry season.

The dam is planned as rock-fill type with 78 m high above the river bed and a effective storage of 70 x 10^6 m^3. Spillway is of center flow type arranged in the depression in the left abutment of the dam. Fig. 3.8.5 and 3.8.6 shows the general factures of dam and spillway.

The hydropower development of 18,600 kW in installation capacity is expected by firm discharge of 35 m^3/sec and rated head of 63 m.

The total construction is estimated at Rp. 91,102 x 10^6 and the net benefit is estimated at Rp. 14,549 x 10^6/year.

More details are stated in Note MP-4.

(2) Konto river II Scheme

(a) Natural Condition

The Konto river II damsite is selected on the Kontro river, 10 km upstream from the Selorejo reservoir (See Fig. 3.8.7). The catchment area at the dam stie is 107 km^2. The Konto river has formed a deep gorge with the bottom width of about 150 m. Accordingly to the reconnaissance survey, andesite and volcanic breccia outcrops are found in the abutment.

(b) Development plan

Main objectives of this scheme are envisaged as water supply and hydropower generation. This scheme also will contribute to the enlongation of life time of the Selorejo resevoir. Transbasin plan is proposed from the upstream of the Brantas river to the Konto river, since the distance between two rivers is only 8 km (See Fig. 3.8.7) and the hydropower potential is worth for the development. By this transbasin plan, the catchment area is increased about 1.6 times to be 167 km^2. The runoff is expected to be 7 m^3/sec to 13^3/sec in the rainy season and 4 m^3/sec to 7 m^3/sec in the dry season. The total head for the hydropower generation including the existing hydropower plant located in the downstream such as Slorejo, Mendalan and Siman is expected to be 599 m. This figure is quite higher than the total head of Karangkates, Wlingi and Lodoyo in which the water resources are now used for hydropower generation.

For this scheme, transbasin tunnel having a length of 7.9 km is planned. A dam is planned to be of rock-fill type with a height of 116 m above the river bed. A side channel spillway is provided in the right abutment of the dam. Fig. 3.3.8 and 3.8.9 shows the dam and spillway features. The effective storage is 63 x 10^6 m^3. A headrace tunnel and a penstock is to be laid out along the right ridge of the Konto river about 7 km long to a proposed power house.

Hydropower development of 62,000 kW in installed capacity is expected by firm discharge of 24 m^3/sec and the rated head of 310 m.

Total construction cost is estimated at Rp. 202,741 x 10^6 and the project benefit is estimated at Rp. 34,904 x 10^6/year.

Details are explained in Note MP-5.

(3) Babadan Scheme

(a) Natural condition

The damsite is selected on the Bendokrosok river a left tributary of the Brantas river, about 8 km west from the Kediri city. The damsite has a wide riverbed of 600 m. The catchment area is as small as 19.8 km^2, however, the effective storage capacity of 90 x 10^6 m^3 is topographically possible. The runoff of the Bendokrosok river is estimated to be 1.5 m^3/sec to 3.4 m^3/sec in the rainy season or 0.2 m^3/sec in the dry season.

(b) Development plan

A transbasin plan is formulated so as to collect water from five rivers; from the right side, the Bruno river and the Gengsang river and from the left side, the Sawur river, the Cerme river and the Babon river. Total catchment area is 111 km^2. Total length of the transbasin way is 9.0 km. Fig. 3.8.10 shows the layout of the transbasin plan.

A dam is planned to be a rockfill type with a height of 75 m and crest length of 880 m. A side channel spillway is provided on the right side abutment of the dam. Fig. 3.8.11 shows the outlines of the dam and the spillway.

Total construction cost is estimated at Rp. 140,111 x 10^6 including the transbasin plan and the project benefit is estimated at Rp. 12,158 x 10^6/year.

The more details are stated in Note 6.

(4) Kuncir Scheme

(a) Natural condition

The Kuncir damsite is selected on the Kuncir river, about 15 km south west from Nganjuk city. The Kuncir river, originating from the top of Mt. Limas, flows down on the steep mountain slope with wide riverbed and forms a wide alluvial fan composed of sand and gravel at the foot of the mountain. Even in the damsite and the reservoir, the steep and wide riverbed is the inevitable.

According to the test boring made at several point of the alternative sites, it is considered that the bed rock is composed of the volcanic breccia having a low permeability. The abutments are composed of weathered volcanic breccia. The river bed is formed with sand and gravel of which thickness is assumed at around 10 m and the permeability is in the order of 10^{-4} cm/sec.

The catchment area at the dam site is 70 km^2. The runoff at the damsite is estimated to be 4 m^3/sec to 12 m^3/sec in the rainy season and 0.7 m^3/sec to 2.2 m^3/sec in the dry season.

(b) Development plan

The dam height is set at 80 m above the river bed which is topographically maximum height. The effective storage capacity is 22.5 x 10^6 m^3.

The dam type is assumed as gravel-fill type with the center core, since the gravel is ample as river deposit. The center core is extended up to the bedrock level by removing the river deposit so as to avoid leakage through the dam foundation. A side channel spillway is provided in the right abutment so as to smoothly connect to the downstream river bed. Fig. 3.8.12 to 3.8.14 shows the general layout of this dam scheme.

Total construction costs is estimated at Rp. 75,083 x 10^6. Project benefit is Rp. 5,859 x 10^6/year.

More details are explained in Note MP-7.

(5) Semantok Scheme

(a) Natural condition

The Semantok dam site is selected on the Semantok river, 10 km upstream from the confluence with the Widas river. The dam site is located in lowlying hills and the topography of the dam site is very gentle. Catchment area at the dam site is 61 km^2. Run-off at the damsite is estimated to be 1.6 m^3/sec to 5.7 m^3/sec in the reainy season and 0.2 to 5.0 m^3/sec in dry season.

(b) Development plan

This scheme is envisaged for the objective of irrigation water supply or domestic water supply. The dam height is tried to be set at topographically the heightest so as to use the water resources at maximum. The dam height is 33 m above the river bed and the effective storage is 40 x 10^6 m^3. General layout of the dam and the spillway is shown in Fig.3.8.15 to Fig. 3.8.17. The dam is of earthfill type and the crest length reaches upto 3,570 m. Total embankment volume is estimated at 5.3 x 10^6 m^3. The spillway is center flow concrete gravity type to be provided on the left side of the river.

Total construction cost is estimated at Rp.73,167 million, while the benefit is only 3,663 x 10^6/year assuming that the positive benefit is brought by domestic water supply.

(6) Kedungwarak Scheme

(a) Natural condition

The Kedungwarak dam is selected in the narrow valley, 14 km north from the confluence with the Widas river. Due to the topographic conditions in the dam site and widely spreading basin to be expected to become reservoir, it is possible to store large volume of water by a compact dam. However, the catchment area is only 32 km^2 and the runoff estimated is only 1.2 to 1.5 m^3/sec in the rainy season and 0.1 to 0.4 m^3/sec in the dry season. According to the boring investigation at the damsite, the dam foundation consists of tuff-aceous sandstone and volcanic sand stone. The bearing strength is assumed to be enough to allow filltype of several ten meters high. Fig. 3.8.18 shows the location of the Kedungwarak dam.

(b) Dam development

Conventional type of development is not attractive due to the small quantity of annual runoff compared with the large storage capacity of the reservoir. Therefore, an inter-seasonal pumped-up plan is introduced. The water is planned to be taken from the main canal reach of the Widas river during the rainy season and pumped up to the dam through a pipeline.

The storage capacity of the dam is set at 54 x 10^6 m^3 by the average runoff of 4.5 m^3/sec in the Widas river at the nearest point to the dams site. The distance is 12.6 km.

The dam height, 32 m is needed to store 54 x 106 m^3. The dam is planned to be homogeneous earth type and the spillway is of side channel type. Fig. 3.8.19 to 3.8.20 show the features of the dam and the spillway.

Total construction cost is estimated at Rp. 41,503 million out of which the cost of pipeline and pump station accounts for Rp.35,609 million. While the project benefit is Rp.3,039 x 10^6/year.

More details of the project feature are referred to Note MP-9.

(7) Beng Scheme

(a) Natural condition

The damsite is selected on the Beng river, 5 km upstream of the confluence with the Brantas river. The Beng river is running through gentle hilly areas forming a narrow valley at the dam site. According to the reconnaissance, it is assumed that the dam foundation is composed of the volcanic sandstone with low degree of consolidation. Catchment area at the damsite is 134 km^2. Run-off at the damsite is estimated to be 6 m^3/sec to 10 m^3/sec in the rainy season and 0.4 to 2 m^3/sec in the dry season.

(b) Development plan

This scheme is formulated fcr water supply to the Beng irrigation area of 3,200 ha and for water supply to the domestic and industrial use. Also the hydropower generation is planned.

From the topographic viewpoint, a large amount of water can be stored with a compact dam compared with own river runoff. The reservoir is located as short as about 3 km from the main Brantas river. Taking these situations into account, inter-seasonal pumping-up plan from the main Brantas river is contemplated.

From the topographic view point, an effective storage capacity is expected at 147 x 10^6 m^3. Pumping-up requirement is estimated to be about 9 m^3/sec to fill up the reservoir every year. Water to be diverted is led in an open channel for about 2.6 km and then pumped up through a pipe line. Fig. 3.8.21 shows the general layout of the pumping-up plan.

As shown in Fig. 3.8.22 three dams are to be provided to form a reservoir. All the dams are of earthfill type. A main dam has a height of 44 m above the river bed and a crest length of 170 m. Other two sub-dams are provided in depressions on the ridge.

A spillway is of side channel-flip bucket type. Fig. 3.8.23 shows the general features of the main dam, spillway and hydropower plant.

Total construction cost is Rp. 61,303 x 10^6, out of which the cost related to the pipeline and the pump station is Rp. 26,394 x 10^6. Benefit is estimated at Rp. 13,555 x 10^6/year.

(8) Lumbangsari Scheme

(a) Natural condition

The damsite is selected on the Brantas river, 12 km south of Malang city or 9 km upstream of the Kepanjen damsite. The Brantas river has formed a narrow and rather deep valley by eroding the flat palin composed of alluvial deposits and volcanic products up to the surface of bed rock in the upstream reach. At the dam site, the depth of the valley is around 25 m and the bottom width is about 30 m. The catchment area at the dam site is 842 km² and the run-off estimated is 35 m³/sec to 42 m³/sec in the rainy season and 16 m³/sec to 26 m³/sec in the dry season. Topographically, a storage dam is not expected.

(b) Development plan

Considering the natural conditions mentioned above, this scheme is formulated as a run-off river type hydropower development.

Fig. 3.8.24 shows the location of the site. Fig. 3.8.25 shows the general plan of this scheme. A gated overflow section to be made of concrete is planned ones the full width of the present river course so as to smoothly release floods. The both sides are to be filled with homogeneous earth-fill embankment. An emergency spillway is planned to cope with the probable maximum flood. To take a water head at maximum the dam crest is set at the flat plain level. The dam height is 28 m. Fig. 3.8.26 shows the general features of the dam and the powerhouse.

Hydropower generation of 10,800 kW in the installation capacity is expected by firm discharge of 60 m³/sec and the rated head of 21.8 m.

Total construction cost is estimated at Rp. 34,909 x 10^6. While the project benefit is Rp.6,299 x 10^6/year.

More detail informations can be referred to Note AI-11.

(9) Kepanjen Scheme

(a) Natural condition

The damsite is selected on the Brantas river, 20 km south from the Malang city and 5 km upstream from the Sengguruh damsite (See

Fig. 3.8.27). The Brantas river has formed a deep gorge by eroding the flat plain composed of alluvial deposits and volcanic products up to the surface of bed rock. The depth of the gorge reaches up to 20 m at the damstie. There is a small fall of 10 m high just downstream of the selected damsite. The catchment area at the damsite is 912 km^2 and runoff is estimated to be 28 m^3/sec to 35 m^3/sec in the rainy season and 12 m^3/sec to 20 m^3/sec in the dry season. Topographically, storage effect is not expected.

(b) Development plan

Considering the natural conditions mentioned above, a run-off river type hydropower development is proposed for this scheme.

To get the hydraulic head as high as possible,the dam crest is set at the level of both flat plains. The dam height becomes 20 m above the river bed and the total energy head of 22.5 m obtained for hydropower by the effect of a existing natural fall with damming-up.

Figs. 3.8.28 and 2.8.29 show the features of the dam and the power house.

A gated overflow section to be made of concrete is planned over the full width of the present river course. The both sides of the overflow section are to be filled with homogeneous earth-fill embankment. An emergency spillway is planned to cope with the probable maximum flood.

Hydropower generation of 6,000 kW in installation capacity is expected by firm discharge of 35 m^3/sec and the rated head of 20.3 m.

The total construction cost is estimated at Rp. 20,719 x 10^6. The project benefit is at Rp. 4,280 x 10^6 per year.

5. Economic Feasibility and Priority Ranking

Anticipated benefits of each dam project are estimated, assumed that the stored water will be used for water supply which is the most beneficial use of water, and results are as summaried in Table 3.8.7. Taking into account the normal construction period, economic internal return and the net present worth at the discount rate of 12 % per annum are calculated as shown below;

	EIRR (%)	N.P.W. of B-C at 12% (Rp.10^6)
Genteng I	12.4	2,745
Kali Konto II	12.7	10,459
Babadan	6.6	-49,597
Kuncir	5.8	-30,036
Semantok	2.9	-39,660
Kedungwarak	5.3	-17,653
Beng	16.6	21,694
Lumbangsari	14.2	5,663
Kepanjen	15.6	5,745

The Lumbangsari and Kepanjen projects are pure hydropower schemes, whose sequences of implementation will be decided according to the power system requirements.

Among the storage dams, the priority order will be Beng,Kali Konto II and Genteng I.

If the supply and demand situations of domestic and industrial water becomes tighter, and the value of water becomes high enough to bear the high water supply cost, the dams with low EIRR will appear for further examination. In selecting dams for such examination, dams locating near to the main demand site, Surabaya, will be given priority.

3.8.4 Alternative Dam Schemes for Widas Basin Development

In the series of NOTE - MP, the potential damsites in the Widas river basin are examined from the viewpoint of the overall Brantas river basin. In this sub-section, the other possibility to develop the dams for the Widas river basin development is examined.

The Widas river basin has the limited land and water resources. The lands have been developed to the possible maximum extent under the given condition of the limited water availability in the dry seasons. Further development such as increase in the cropping intensity will need securing water in the dry season by storage development.

The potential damsites are on Kuncir river, Semantok river and Kedungwarak river. For these damsites, dam embankment volume and construction cost in the different scale is estimated as shown in Figs. 3.8.30, 3.8.31, 3.8.32.

The irrigable areas in the dowsntream of the dams are as follows;

Kuncir river	; Widas South Area	- 6,270 ha	
Semantok and Kedungwarak rivers	; Widas Extension Area	- 2,250 ha	

Cropping pattern for each area is studied in NOTE-AI,and storage requrement for each dam is calculated taking into account the effective rainfall and the natural runoff at the damsite. The following cases are examined.

	Irrigation Area (ha)	Cropping Pattern	Storage Requirement ($10^6 m^3$)
Kuncir river	6,270	W.S.P. (100%)	22.5
Kedungwarak river	950	W.S.P (100%),D.S.P.(30%) Polowijo (17%)	28.0
	720	W.S.P (100%), D.S.P(100%) Polowijo (100 %)	28.0
Semantok river	1,300	W.S.P (100%), D.S.P(30%) Polowijo (17 %)	16.9
	1,530	W.S.P (100%), D.S.P(100%) Polowijo (100 %)	35.0

Results of economic evaluation is as shown in Table 3.8.8.