FIGURES

CHAPTER 4

HYDROLOGY

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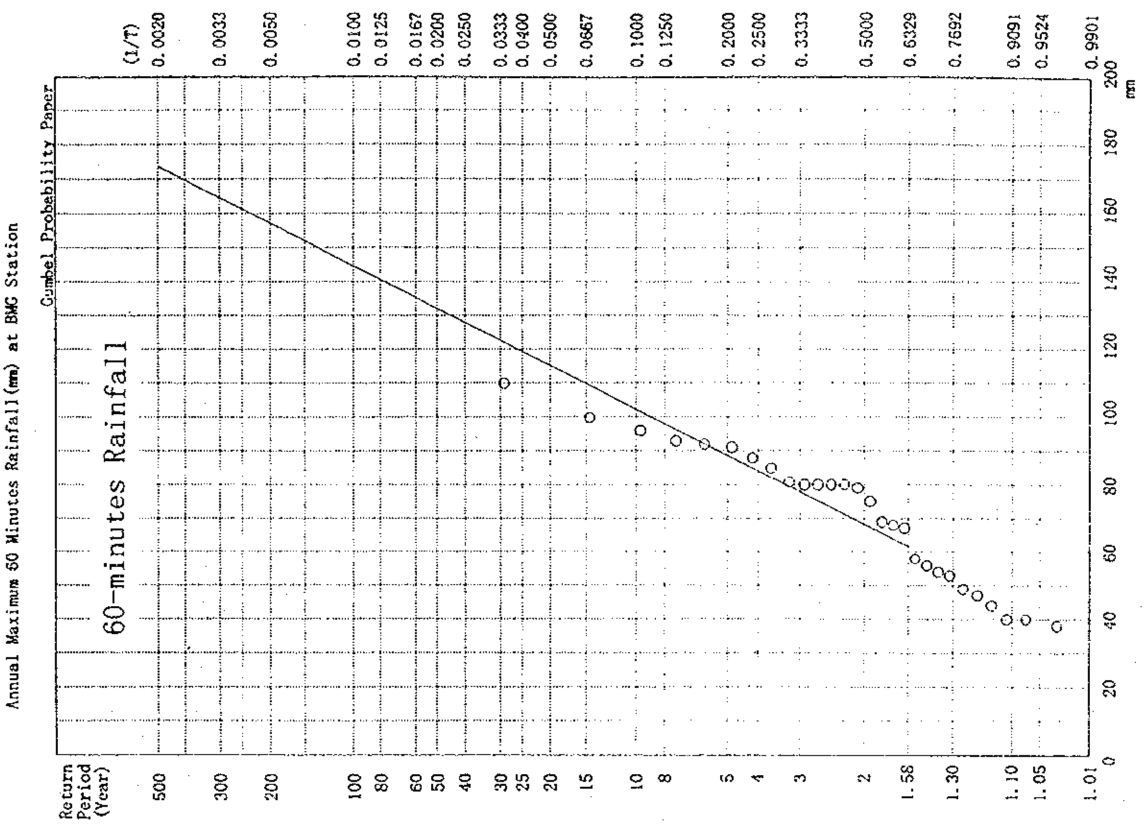
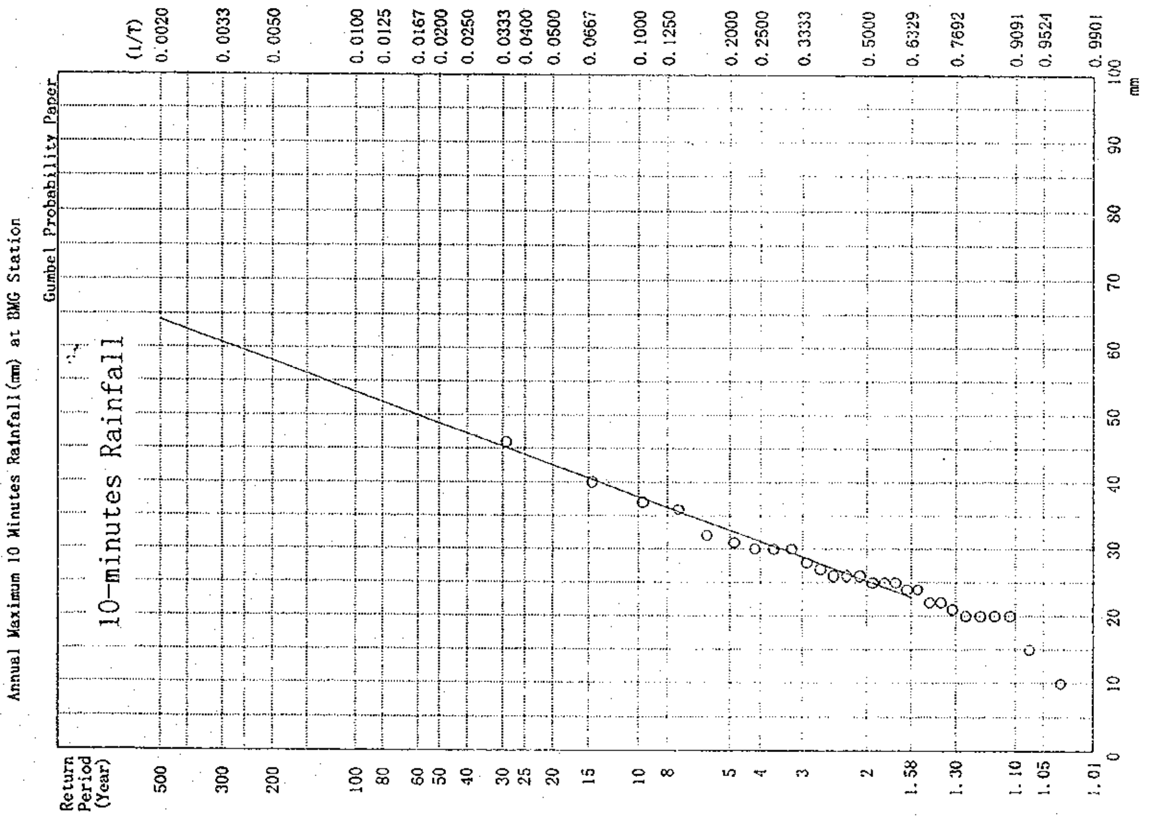
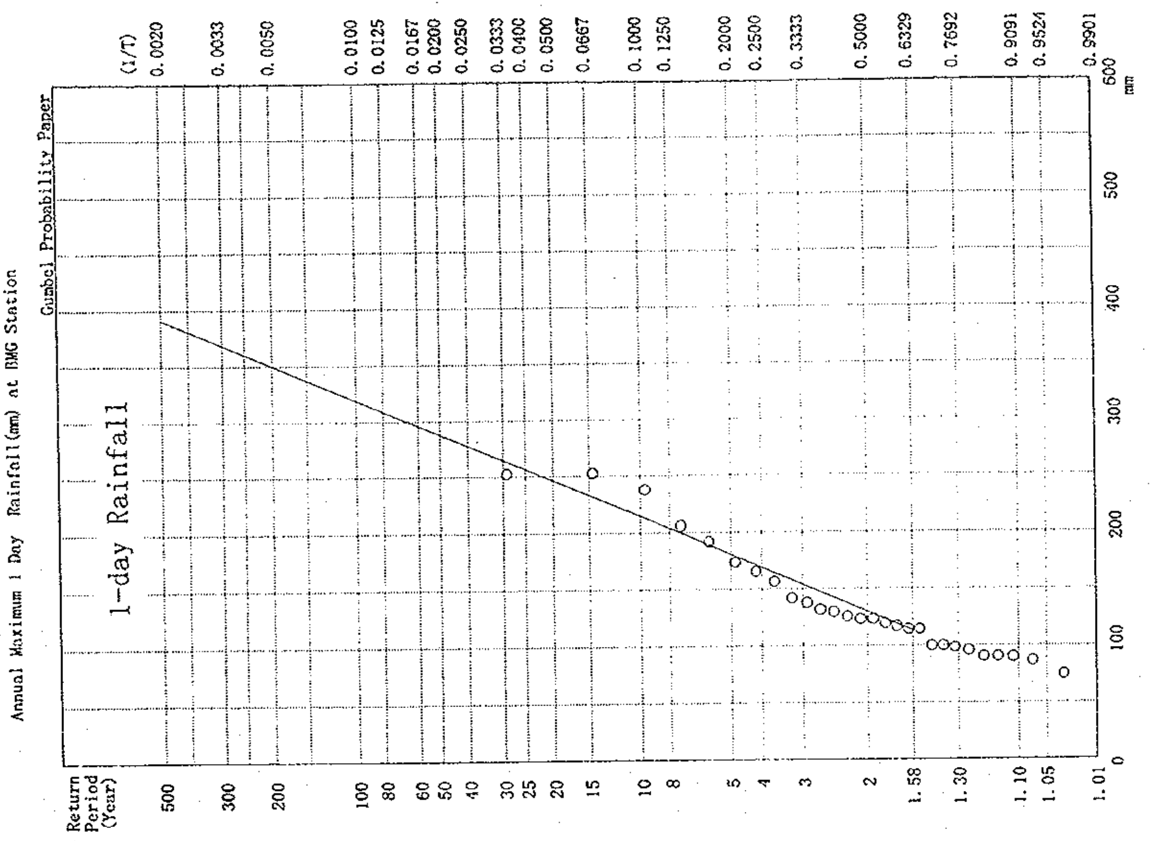
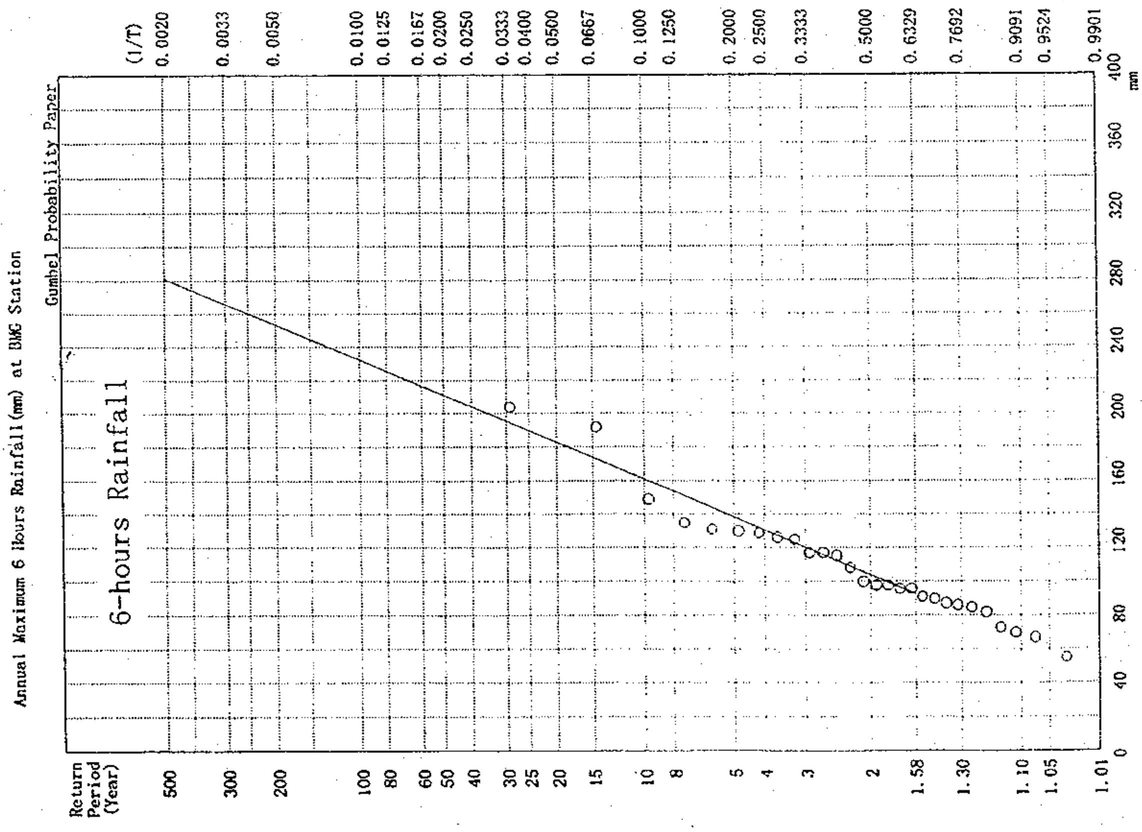




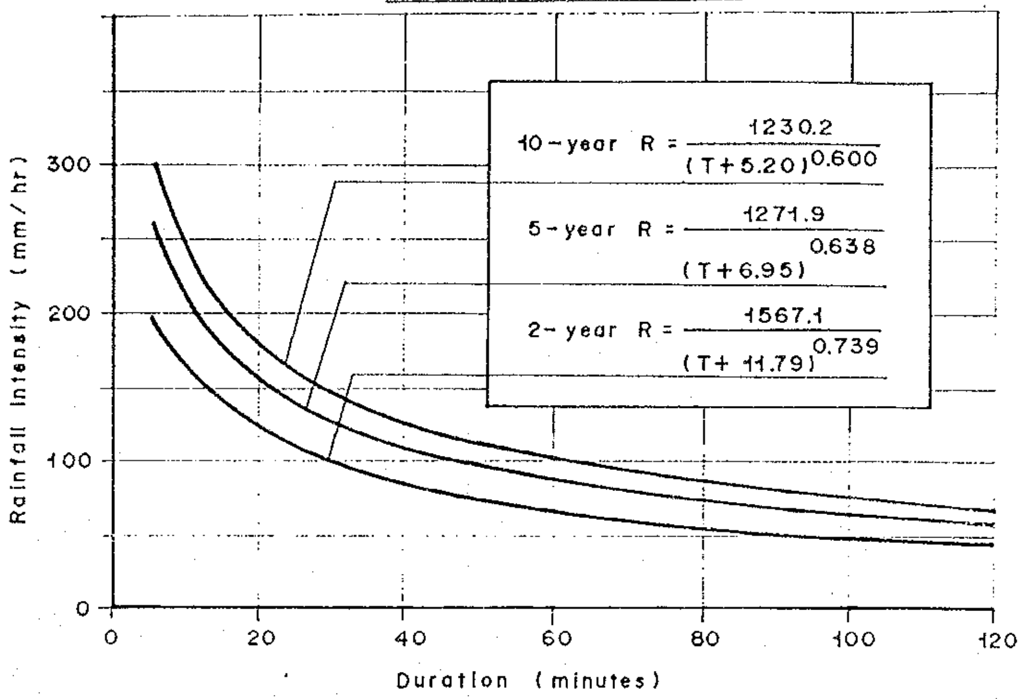
THE DETAILED DESIGN OF FLOOD CONTROL, URBAN DRAINAGE AND WATER RESOURCES DEVELOPMENT IN SEMARANG IN THE REPUBLIC OF INDONESIA	Fig. 4.2.1 PROBABLE RAINFALL IN 10 AND 60 MINUTES
JAPAN INTERNATIONAL COOPERATION AGENCY	



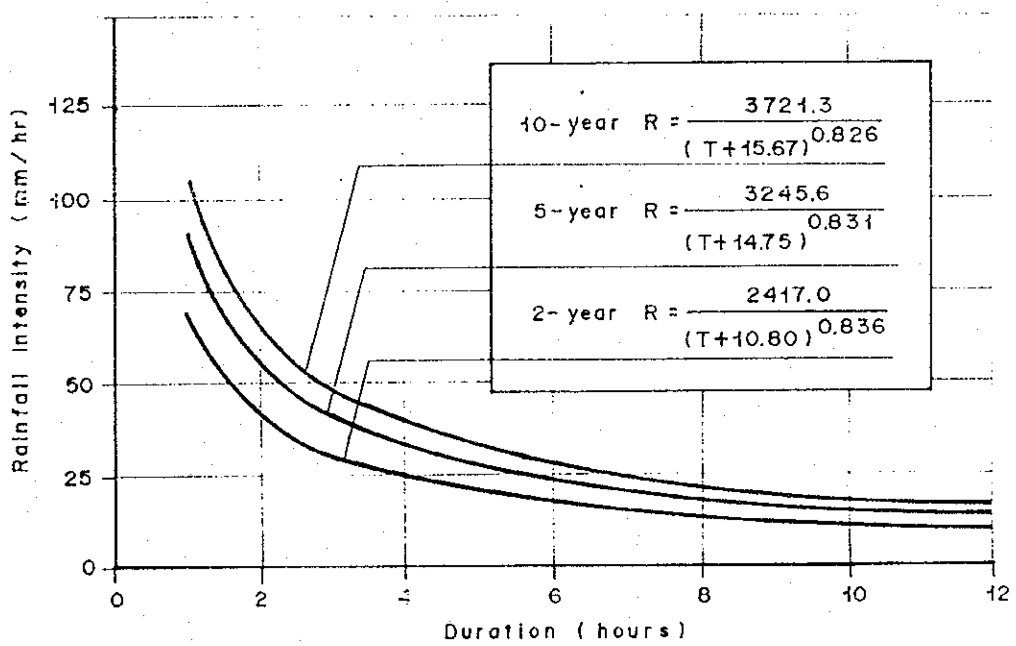


THE DETAILED DESIGN OF FLOOD CONTROL, URBAN DRAINAGE AND WATER RESOURCES DEVELOPMENT IN SEMARANG IN THE REPUBLIC OF INDONESIA	Fig. 4.2.2 PROBABLE RAINFALL IN 6 HOURS AND 1 DAY
JAPAN INTERNATIONAL COOPERATION AGENCY	

Short Duration (T<2 hours)



Long Duration (T>1 hour)



THE DETAILED DESIGN OF FLOOD CONTROL, URBAN DRAINAGE AND WATER RESOURCES DEVELOPMENT IN SEMARANG IN THE REPUBLIC OF INDONESIA	Fig. 4.2.3 RAINFALL INTENSITY CURVE
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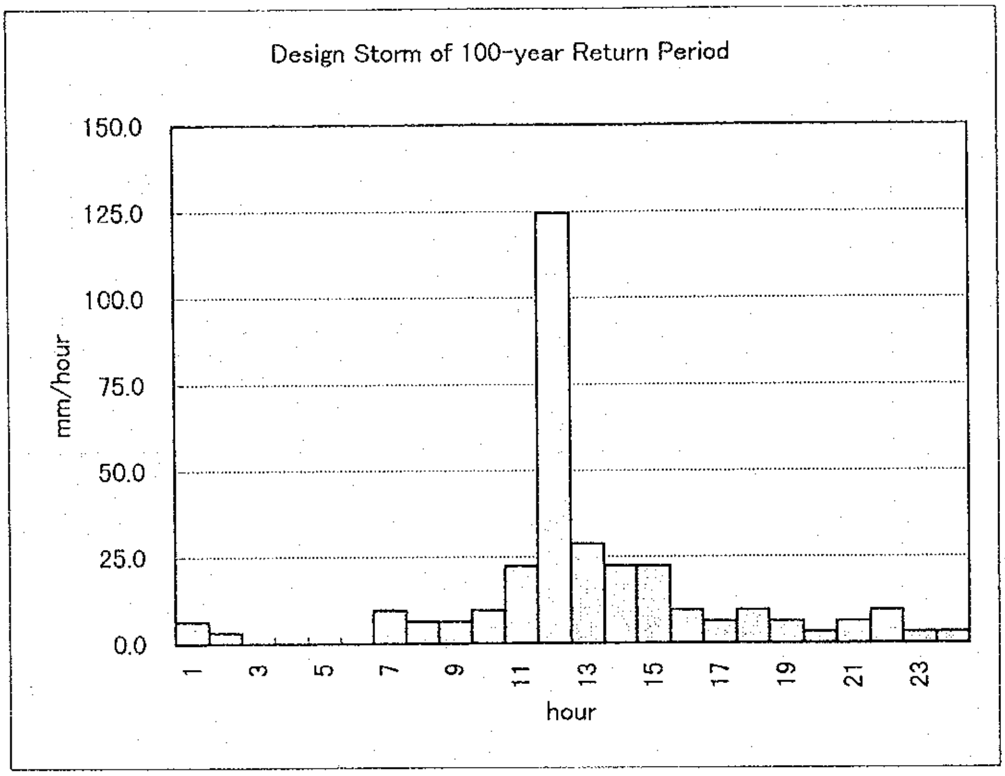

THE DETAILED DESIGN OF FLOOD CONTROL, URBAN DRAINAGE AND WATER RESOURCES DEVELOPMENT IN SEMARANG IN THE REPUBLIC OF INDONESIA

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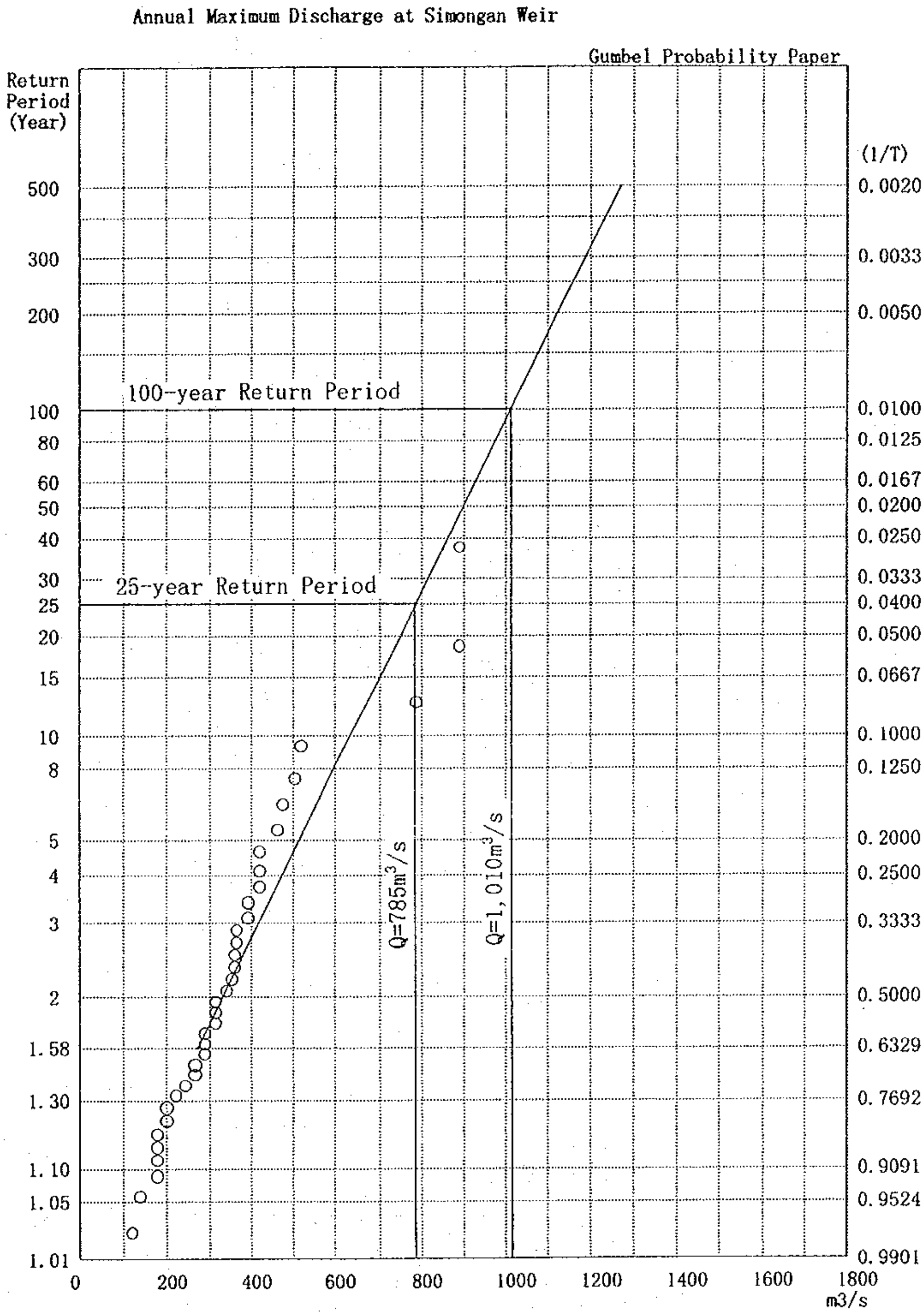
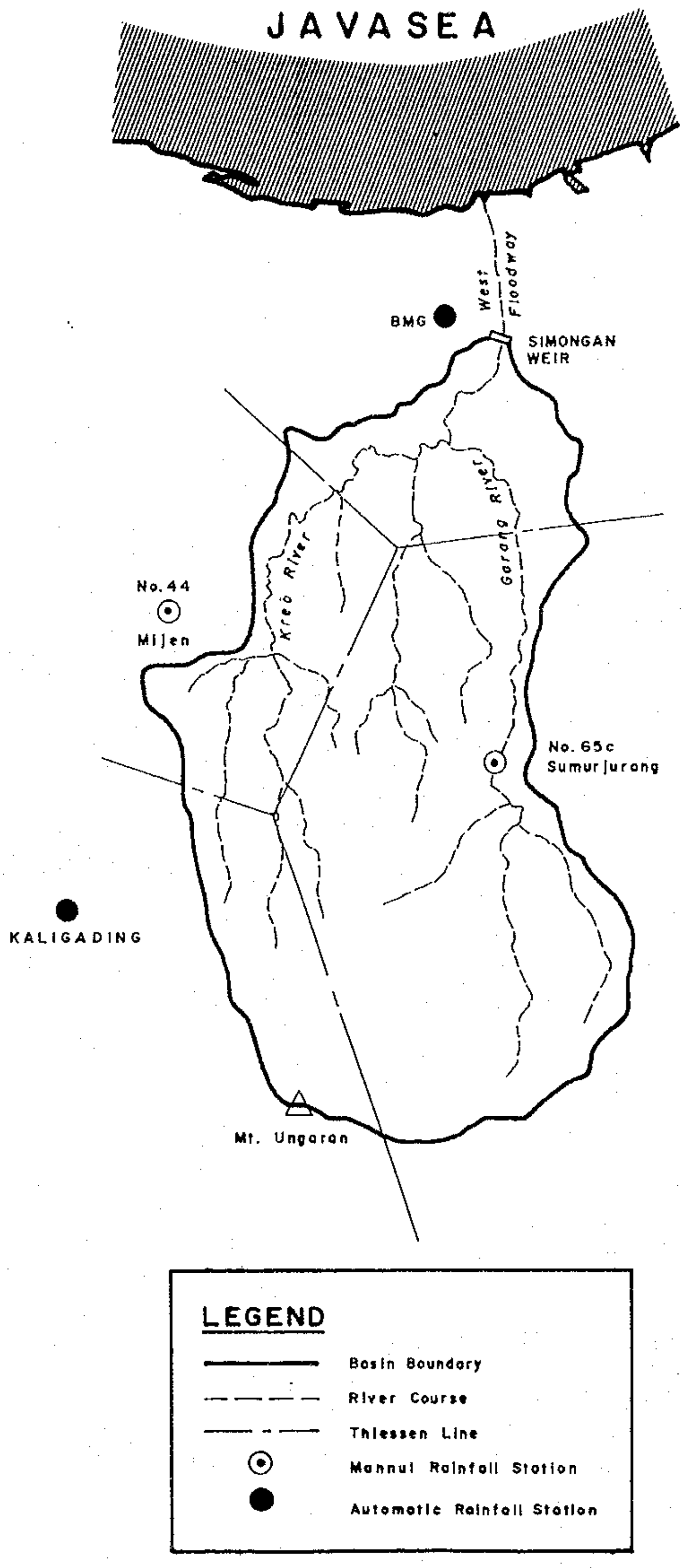
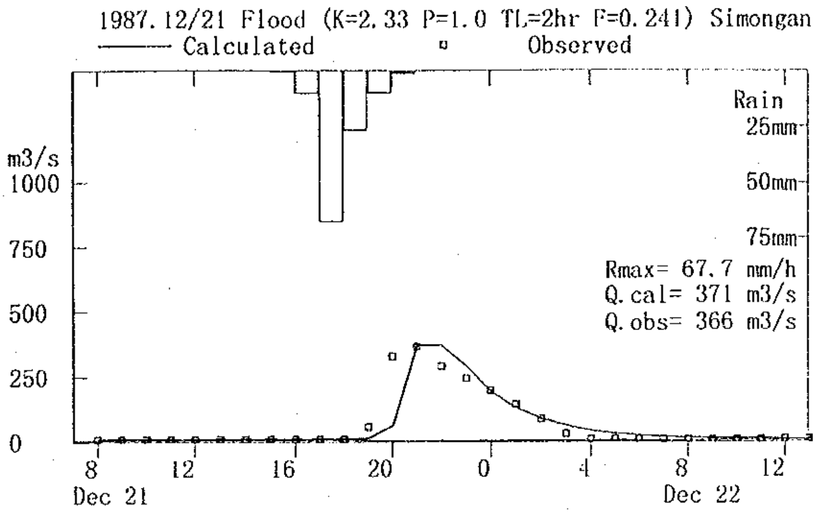
Fig. 4.2.4
DESIGN STORM OF 100-YEAR RETURN PERIOD

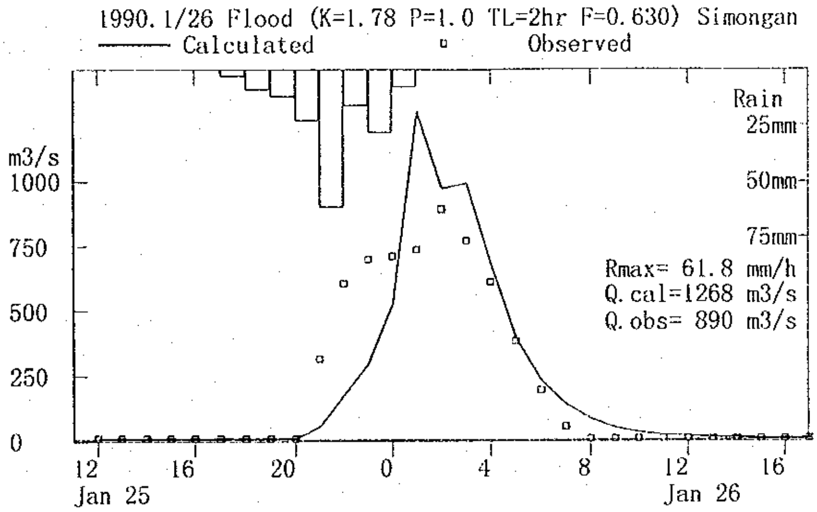


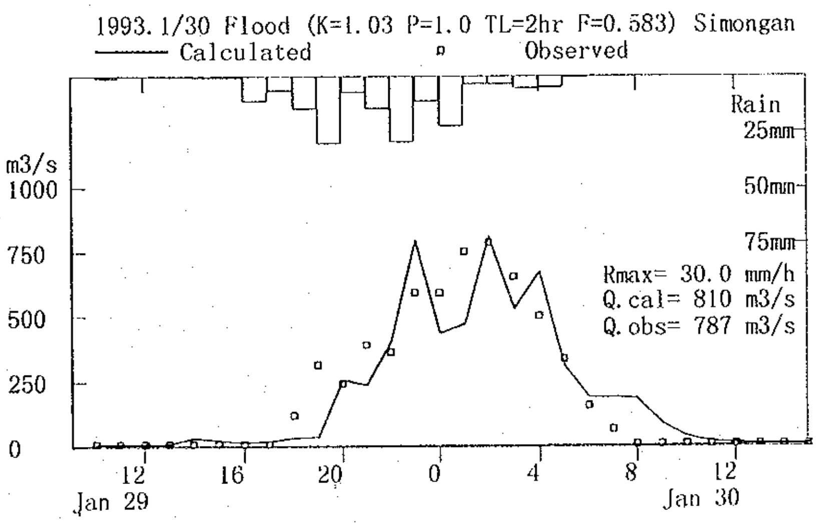
THE DETAILED DESIGN OF FLOOD CONTROL, URBAN DRAINAGE AND WATER RESOURCES DEVELOPMENT IN SEMARANG IN THE REPUBLIC OF INDONESIA	Fig. 4.2.5 PROBABLE DISCHARGE AT SIMONGAN WEIR
JAPAN INTERNATIONAL COOPERATION AGENCY	



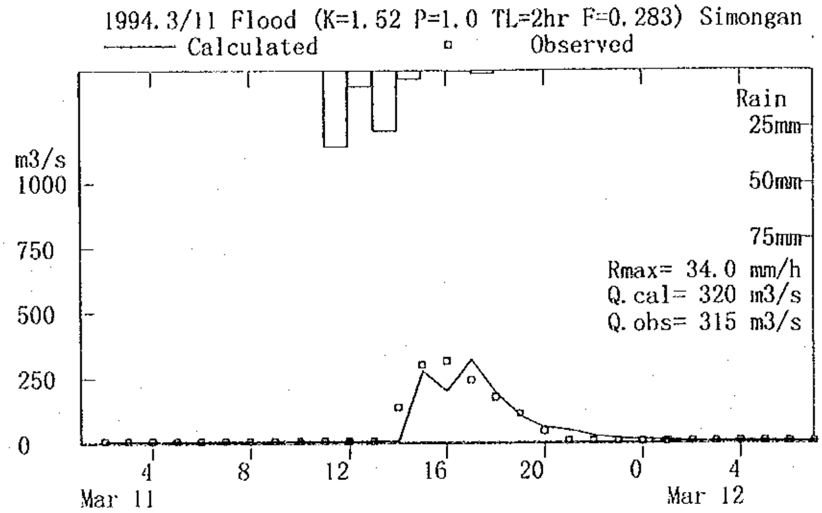
THE DETAILED DESIGN OF FLOOD CONTROL, URBAN DRAINAGE AND WATER RESOURCES DEVELOPMENT IN SEMARANG IN THE REPUBLIC OF INDONESIA	Fig. 4.2.6 THIESSEN POLYGON FOR FLOOD ANALYSIS
JAPAN INTERNATIONAL COOPERATION AGENCY	

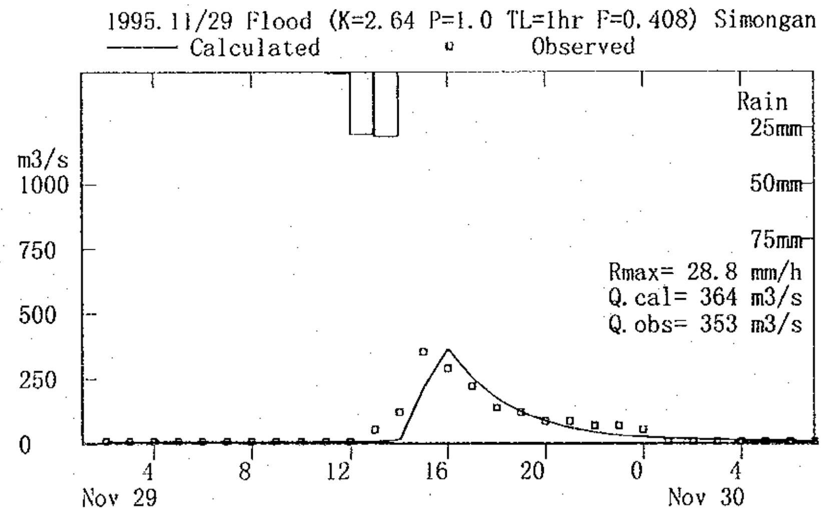




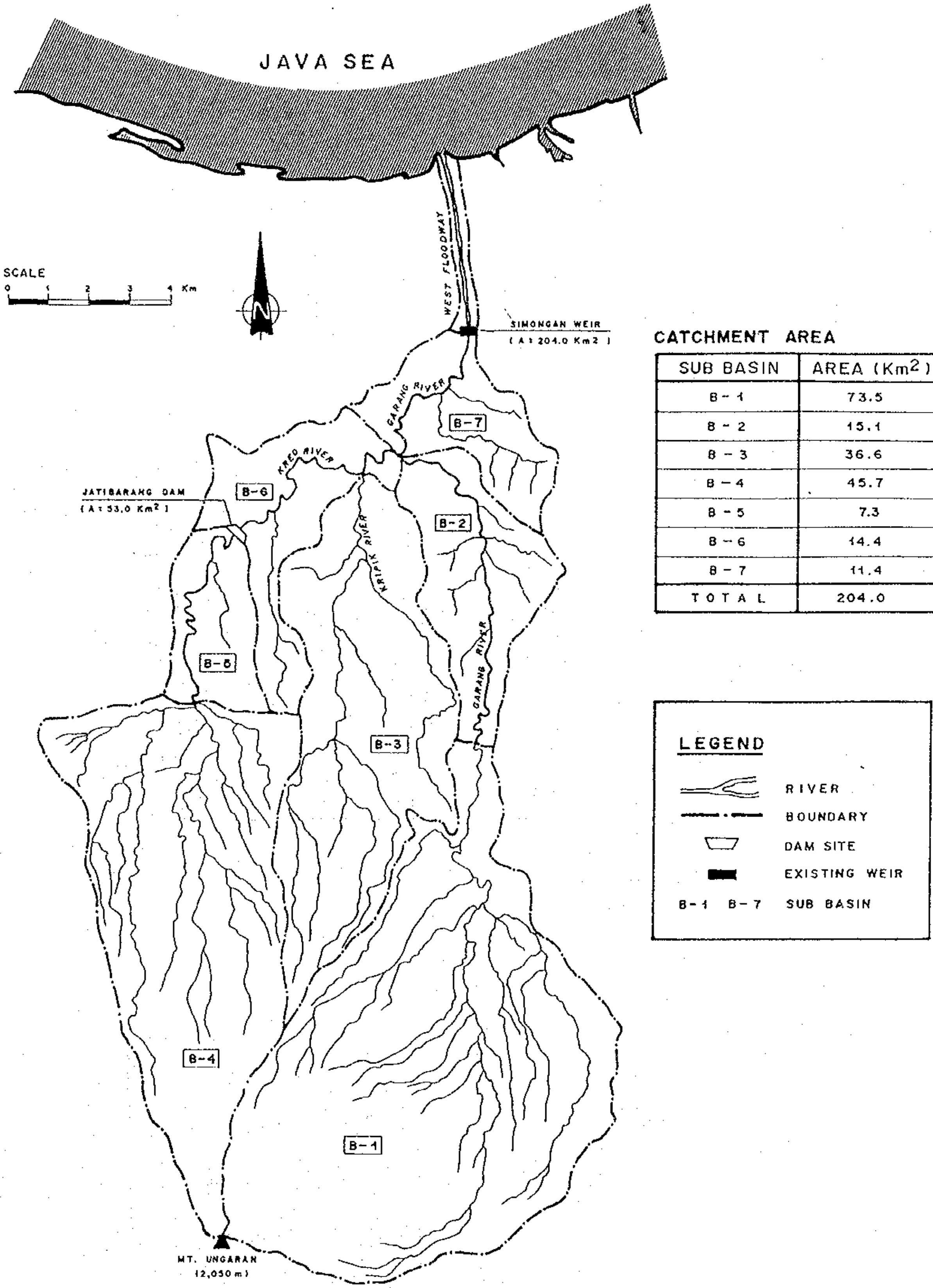


THE DETAILED DESIGN OF FLOOD CONTROL, URBAN DRAINAGE AND WATER RESOURCES DEVELOPMENT IN SEMARANG IN THE REPUBLIC OF INDONESIA	Fig. 4.2.7 (1/2) CALIBRATION OF FLOOD MODEL BY STORAGE FUNCTION METHOD
JAPAN INTERNATIONAL COOPERATION AGENCY	





THE DETAILED DESIGN OF FLOOD CONTROL, URBAN DRAINAGE AND WATER RESOURCES DEVELOPMENT IN SEMARANG IN THE REPUBLIC OF INDONESIA	Fig. 4.2.7 (2/2) CALIBRATION OF FLOOD MODEL BY STORAGE FUNCTION METHOD
JAPAN INTERNATIONAL COOPERATION AGENCY	



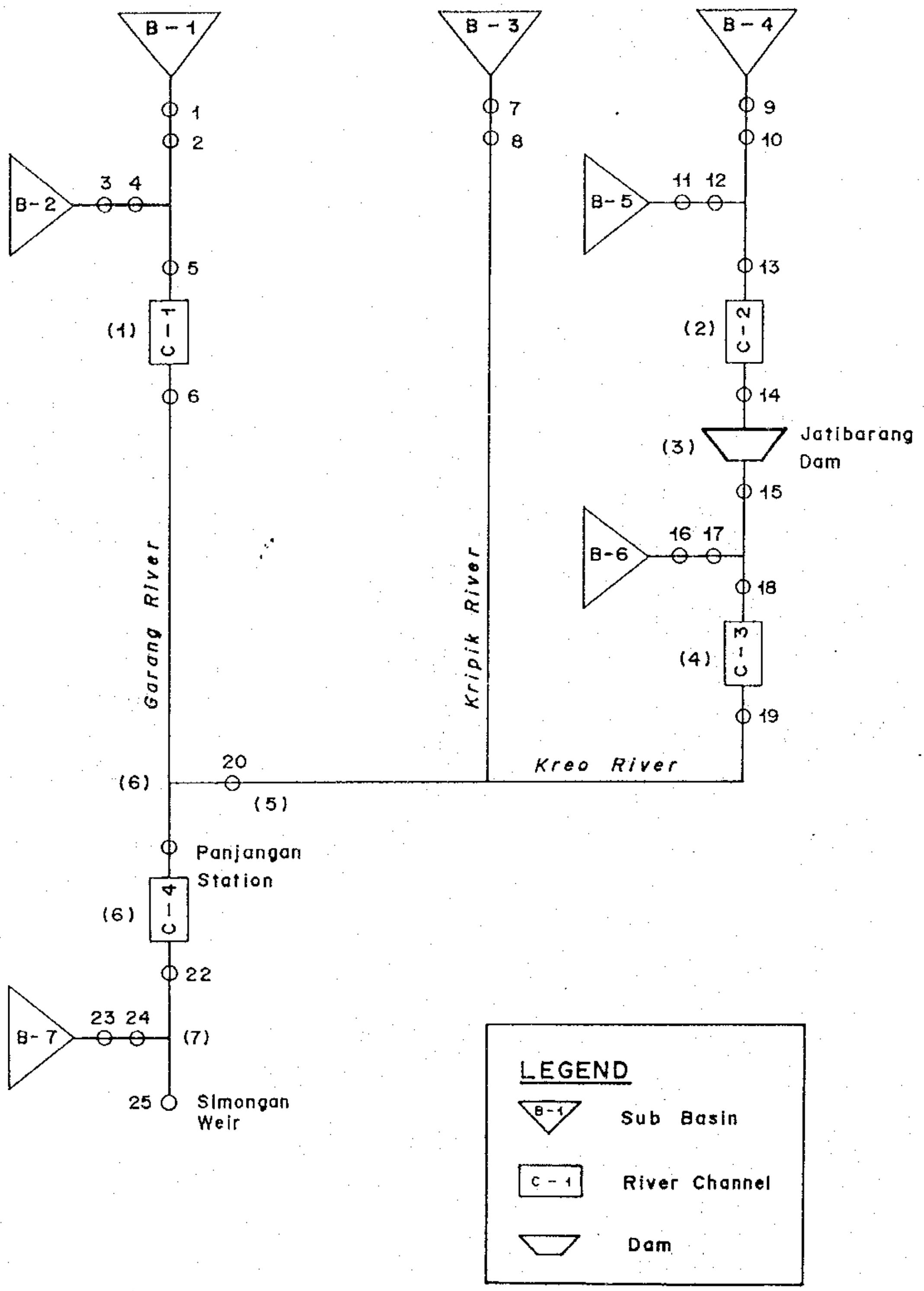
CATCHMENT AREA

SUB BASIN	AREA (Km²)
B - 1	73.5
B - 2	15.1
B - 3	36.6
B - 4	45.7
B - 5	7.3
B - 6	14.4
B - 7	11.4
TOTAL	204.0

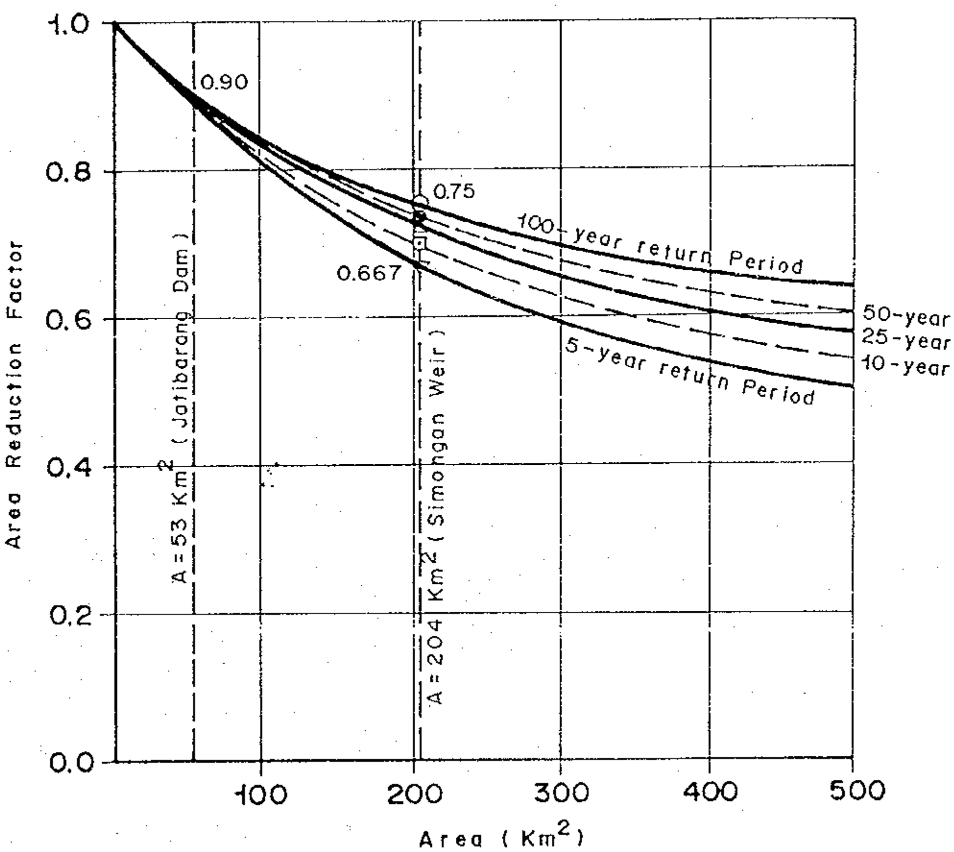
THE DETAILED DESIGN OF FLOOD CONTROL, URBAN DRAINAGE AND WATER RESOURCES DEVELOPMENT IN SEMARANG IN THE REPUBLIC OF INDONESIA

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. 4.2.8
SUB-BASIN DIVISION FOR FLOOD MODEL



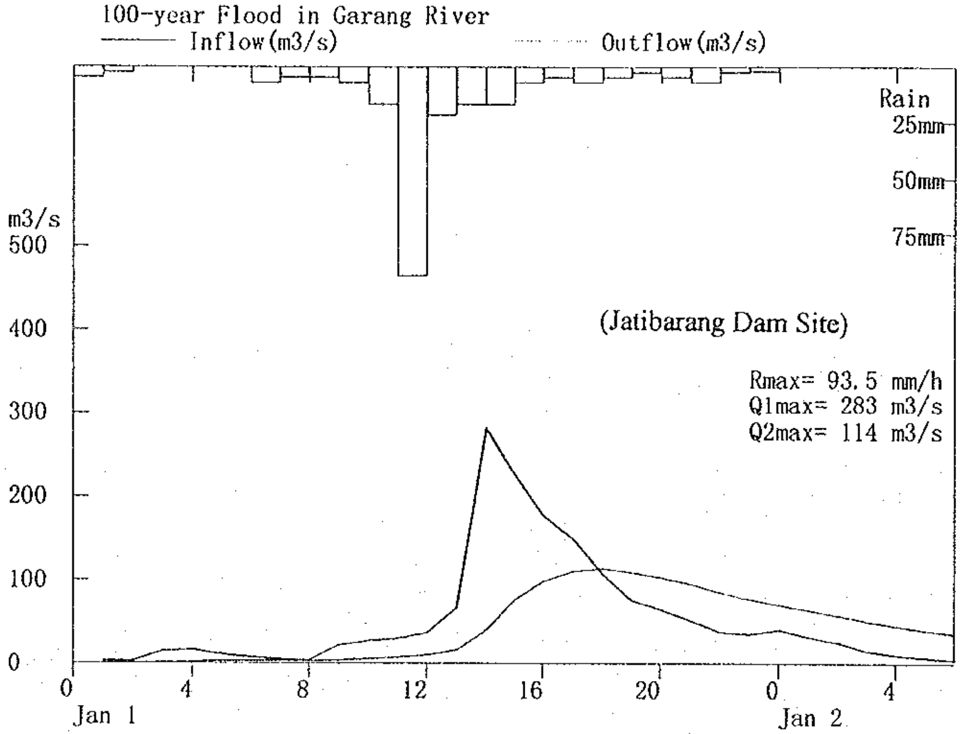
THE DETAILED DESIGN OF FLOOD CONTROL, URBAN DRAINAGE AND WATER RESOURCES DEVELOPMENT IN SEMARANG IN THE REPUBLIC OF INDONESIA	Fig. 4.2.9 MODEL DIAGRAM FOR FLOOD CALCULATION
JAPAN INTERNATIONAL COOPERATION AGENCY	

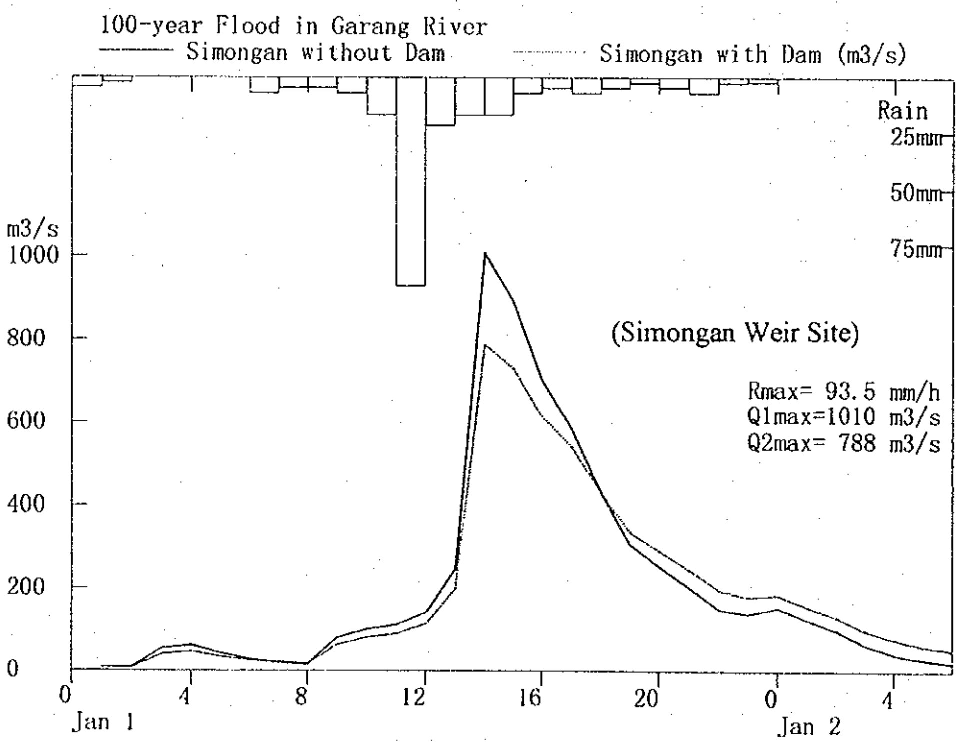


THE DETAILED DESIGN OF FLOOD CONTROL, URBAN DRAINAGE AND WATER RESOURCES DEVELOPMENT IN SEMARANG IN THE REPUBLIC OF INDONESIA

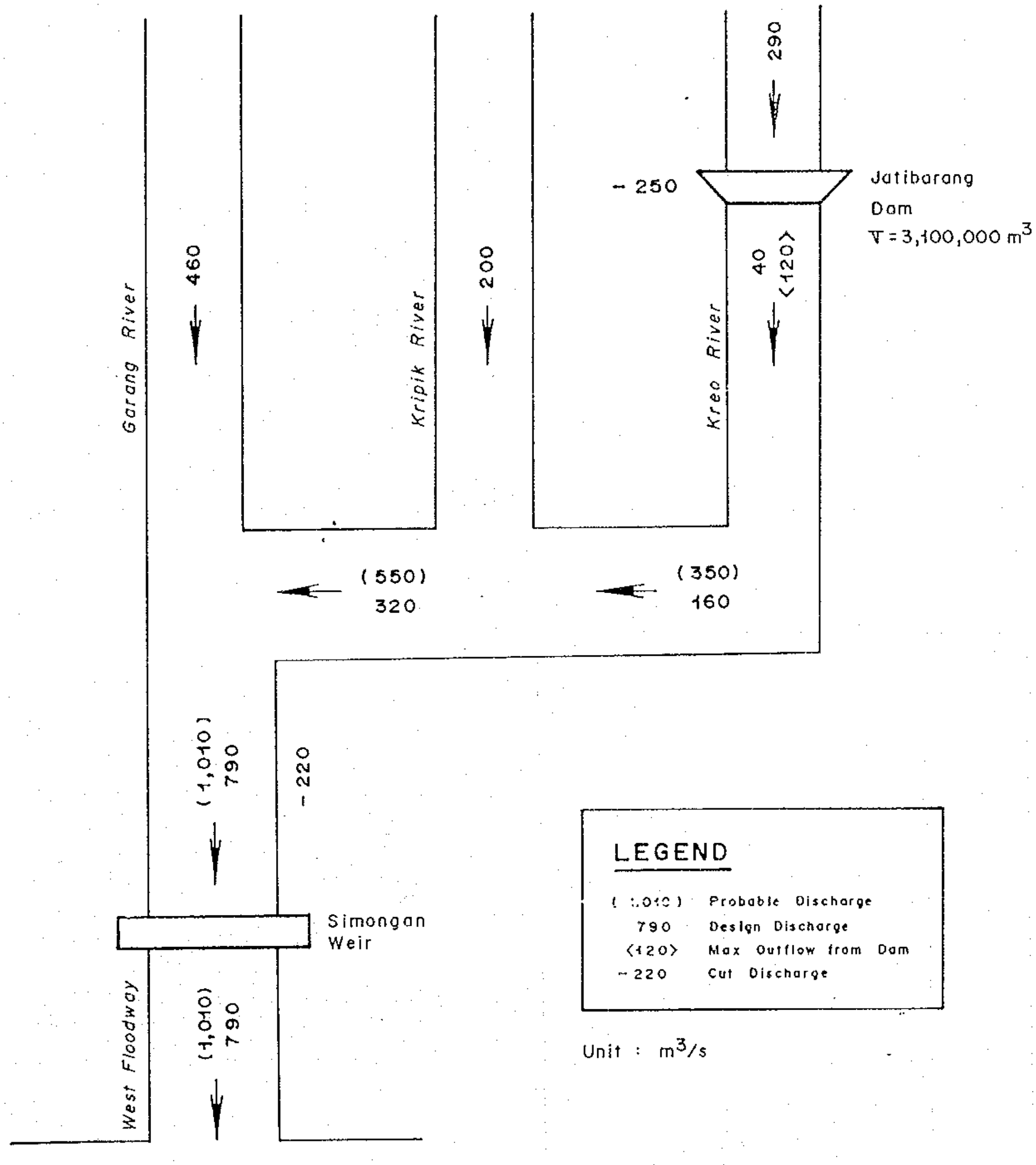
JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. 4.2.10
AREA REDUCTION FACTOR FOR GARANG RIVER BASIN

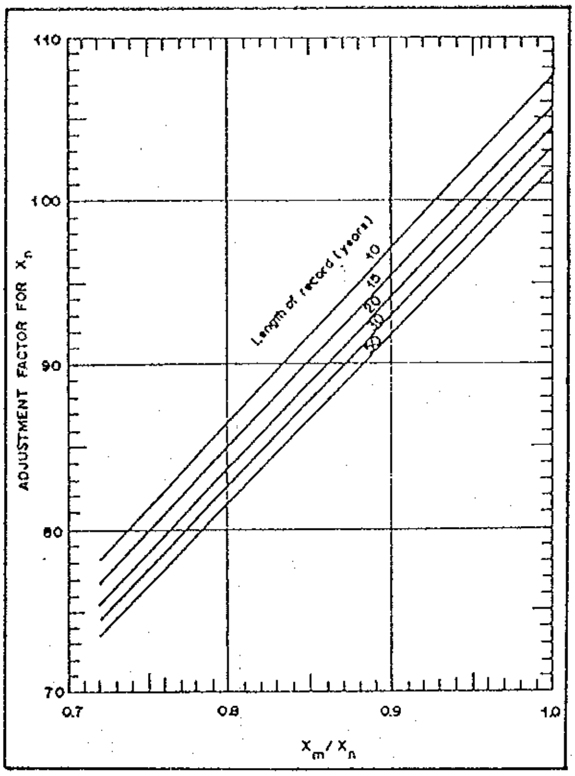


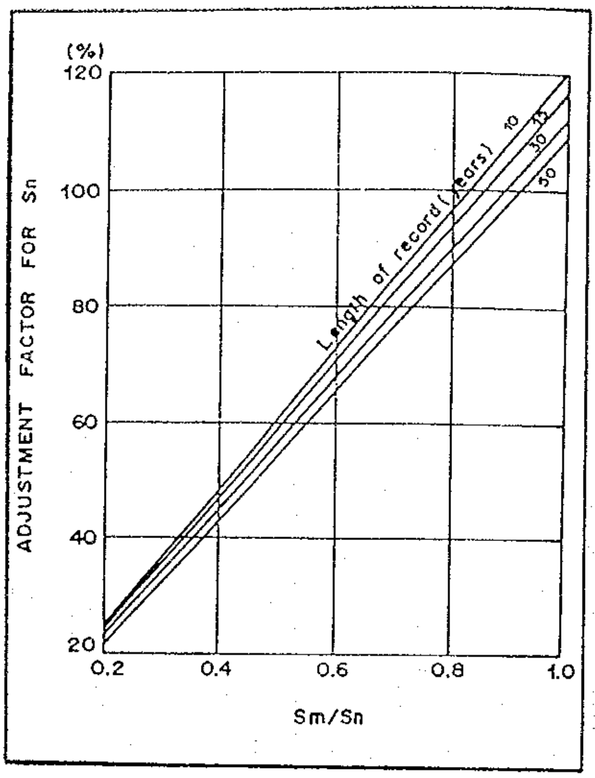


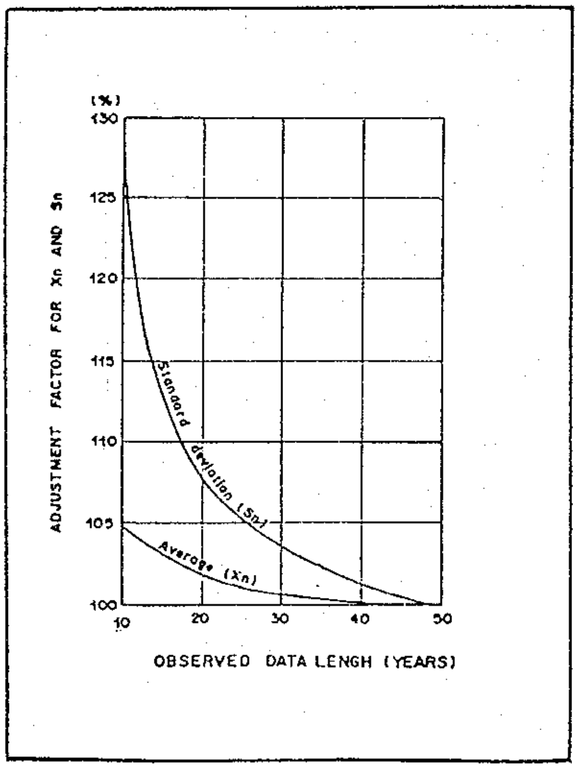
THE DETAILED DESIGN OF FLOOD CONTROL, URBAN DRAINAGE AND WATER RESOURCES DEVELOPMENT IN SEMARANG IN THE REPUBLIC OF INDONESIA	Fig. 4.2.11 FLOOD CONTROL EFFECT BY JATIBARANG DAM
JAPAN INTERNATIONAL COOPERATION AGENCY	



THE DETAILED DESIGN OF FLOOD CONTROL, URBAN DRAINAGE AND WATER RESOURCES DEVELOPMENT IN SEMARANG IN THE REPUBLIC OF INDONESIA	Fig. 4.2.12 DISTRIBUTION OF DESIGN FLOOD DISCHARGE IN GARANG RIVER
JAPAN INTERNATIONAL COOPERATION AGENCY	







NOTE:

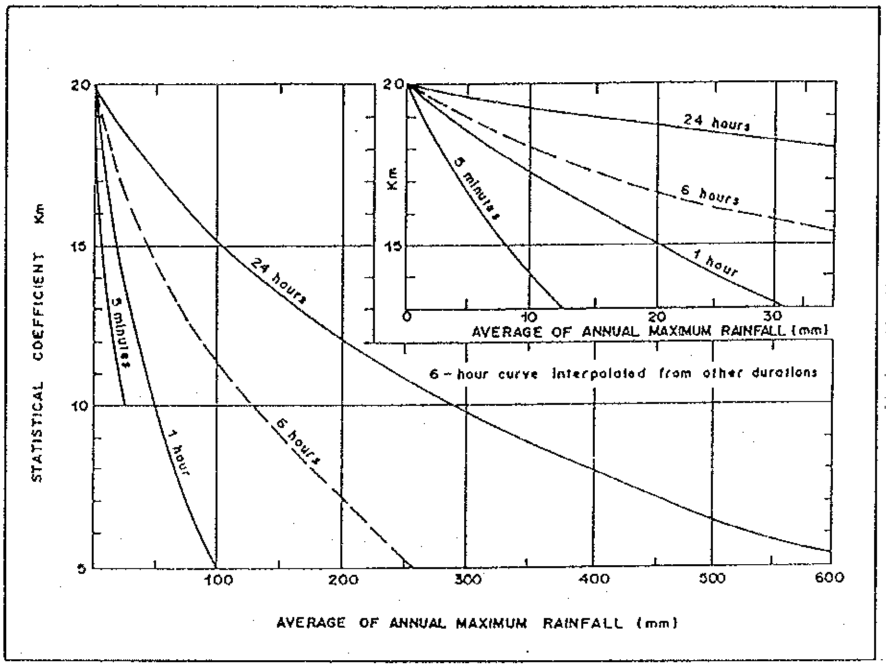
Xn : Unadjusted average of a series of annual maximum precipitation.

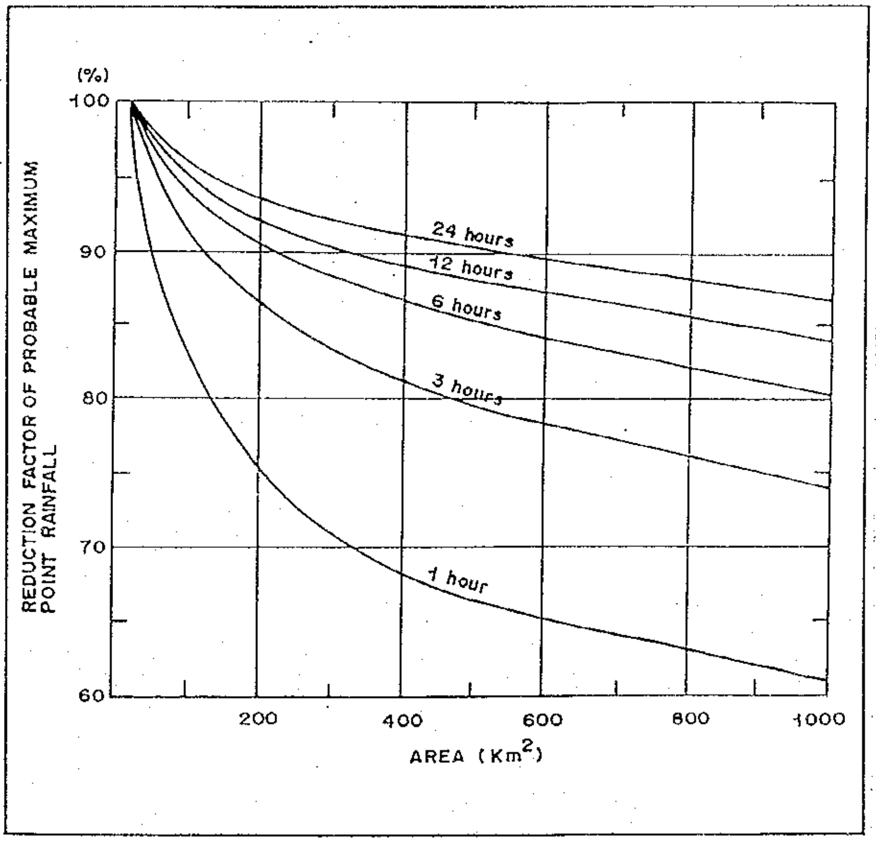
Xm : Unadjusted average of a series of annual maximum precipitation excluding the highest value.

Sn : Unadjusted standard deviation of a series of annual maximum precipitation.

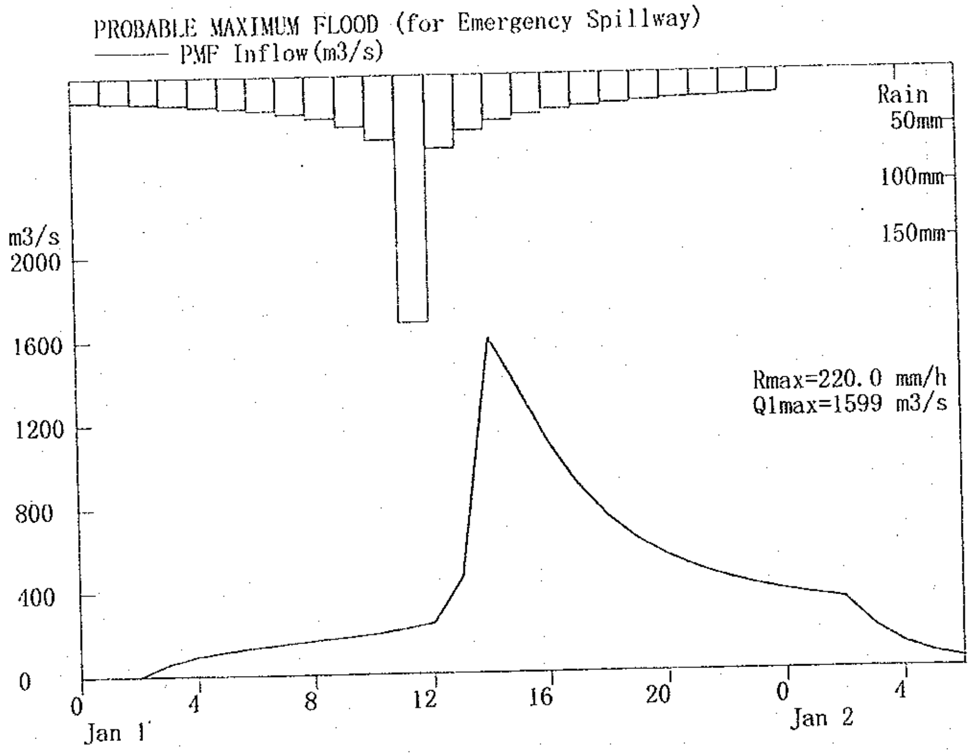
Sm : Unadjusted standard deviation of a series of annual maximum precipitation excluding the highest value.

THE DETAILED DESIGN OF FLOOD CONTROL, URBAN DRAINAGE AND WATER RESOURCES DEVELOPMENT IN SEMARANG IN THE REPUBLIC OF INDONESIA	Fig. 4.2.13 ADJUSTMENT FACTORS DEVELOPED BY HERSHFIELD FOR ESTIMATION OF PMP
JAPAN INTERNATIONAL COOPERATION AGENCY	

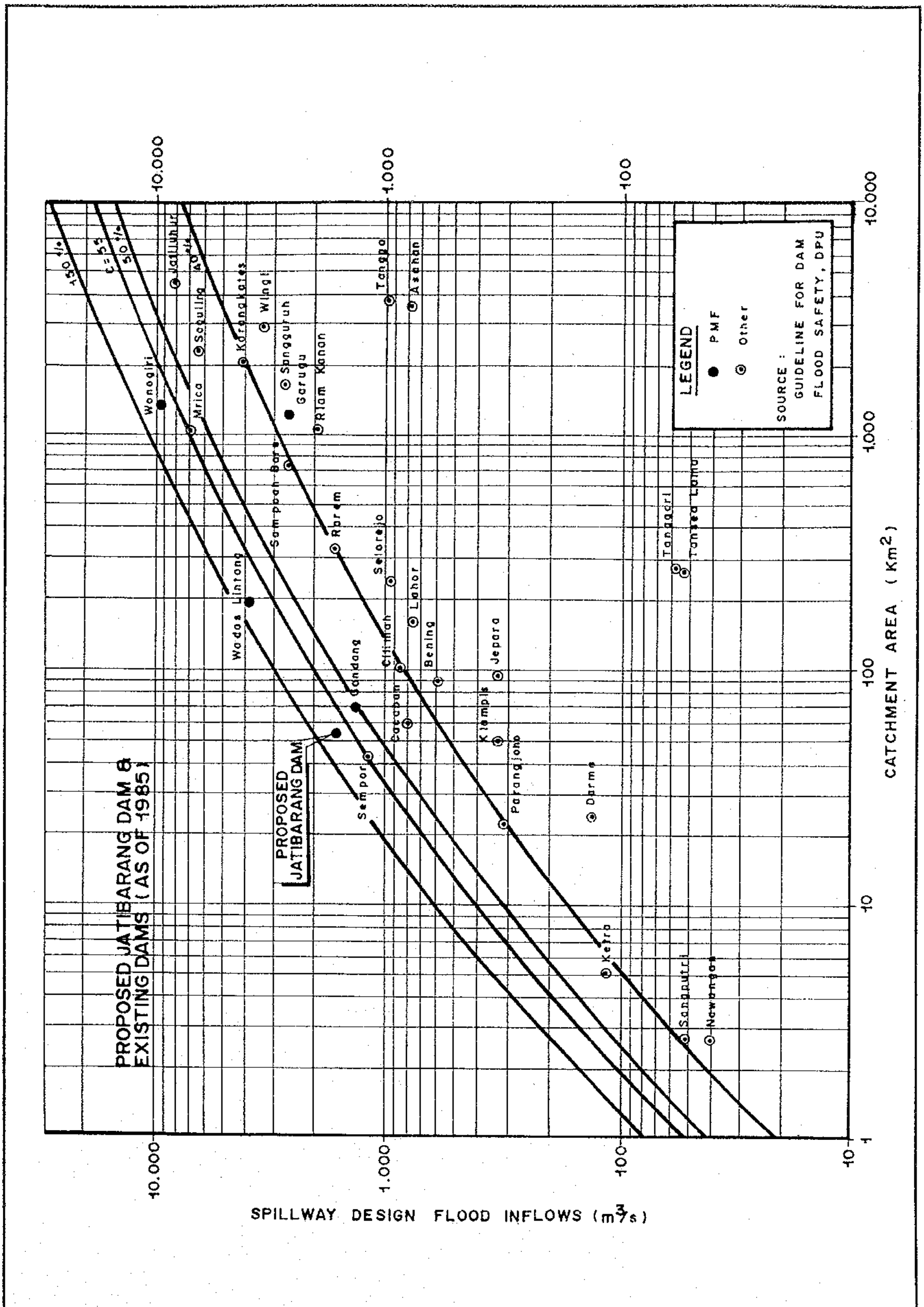




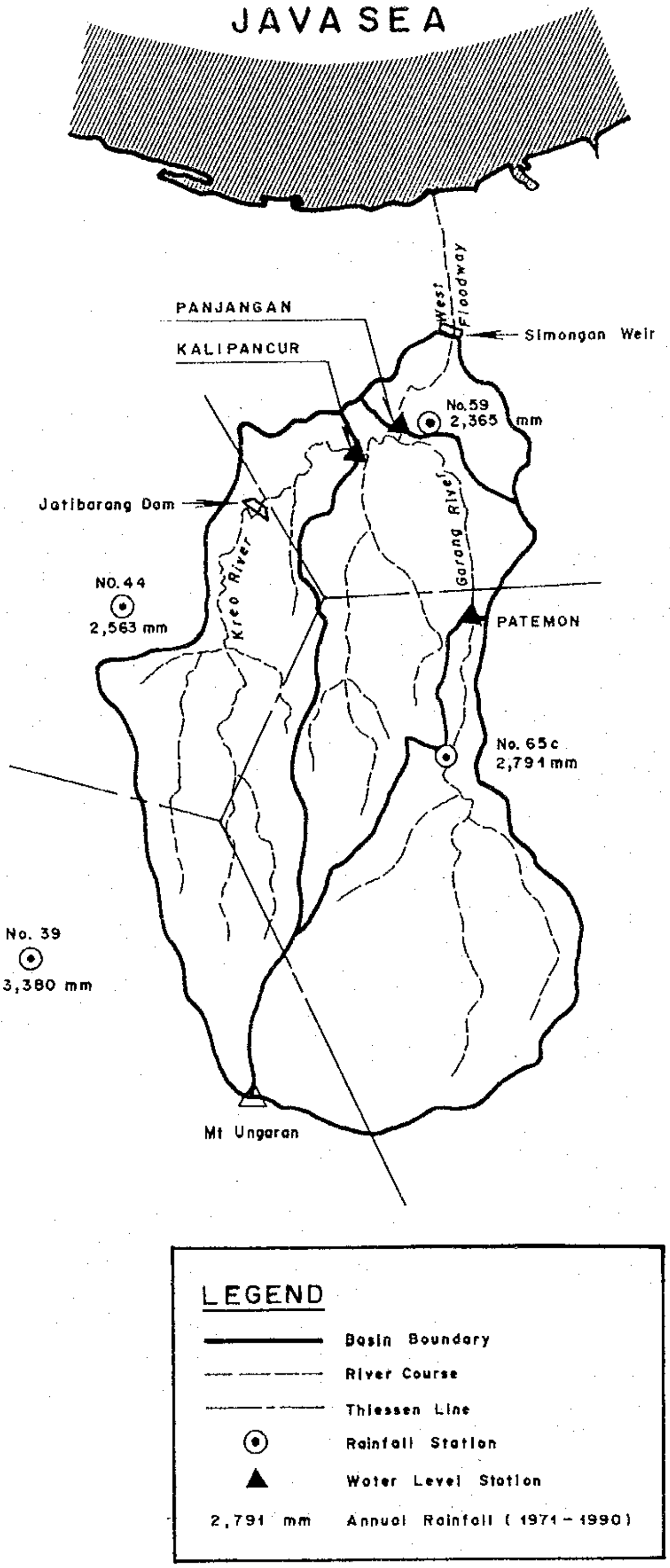
THE DETAILED DESIGN OF FLOOD CONTROL, URBAN DRAINAGE AND WATER RESOURCES DEVELOPMENT IN SEMARANG IN THE REPUBLIC OF INDONESIA	Fig. 4.2.14 STATISTICAL COEFFICIENT AND AREA REDUCTION FACTOR BY HERSHFIELD FOR ESTIMATION OF PMP
JAPAN INTERNATIONAL COOPERATION AGENCY	



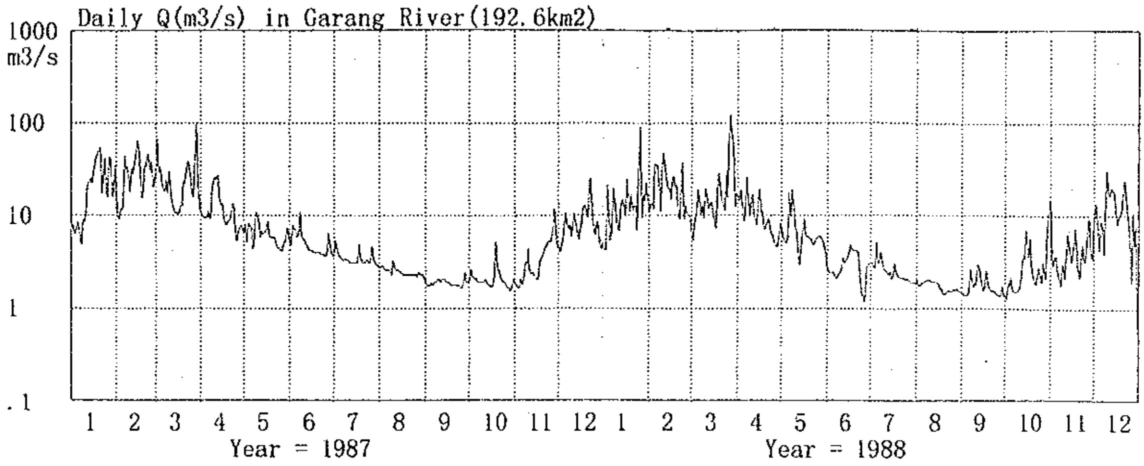
THE DETAILED DESIGN OF FLOOD CONTROL, URBAN DRAINAGE AND WATER RESOURCES DEVELOPMENT IN SEMARANG IN THE REPUBLIC OF INDONESIA	Fig. 4.2.15 MODEL HYETOGRAPH OF PMP AND HYDROGRAPH OF PMF INLFOW FOR JATIBARANG DAM
JAPAN INTERNATIONAL COOPERATION AGENCY	

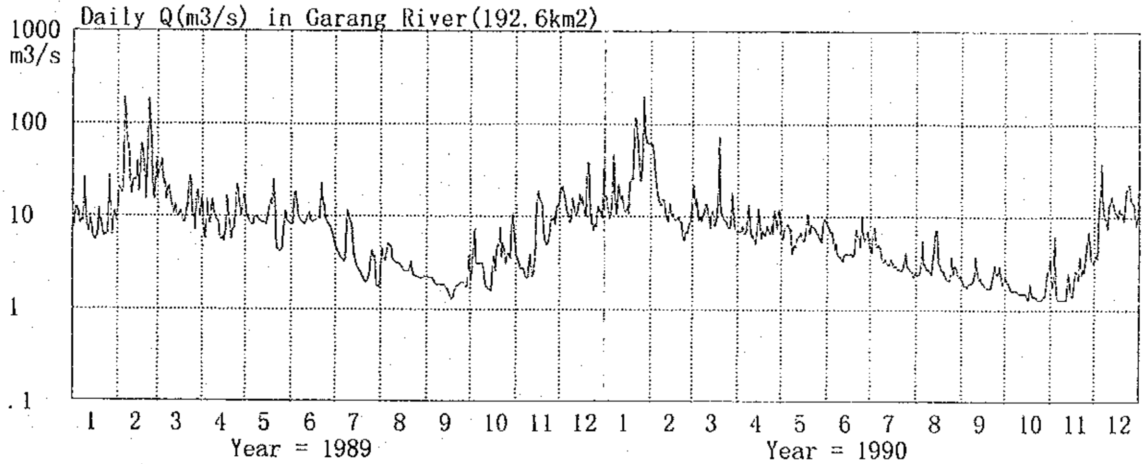


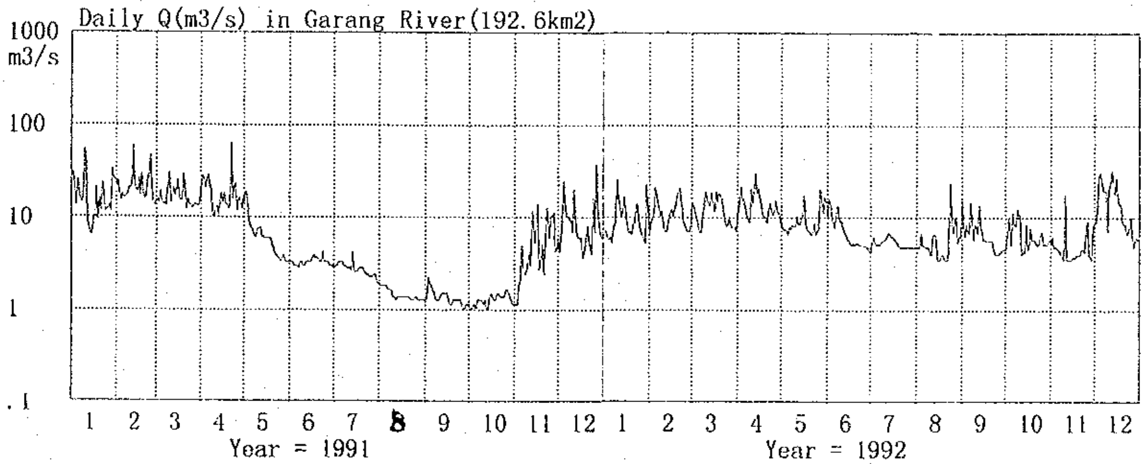
THE DETAILED DESIGN OF FLOOD CONTROL, URBAN DRAINAGE AND WATER RESOURCES DEVELOPMENT IN SEMARANG IN THE REPUBLIC OF INDONESIA	Fig. 4.2.16 DESIGN PEAK INLFOW DISCHARGE OF EMERGENCY SPILLWAY FOR DAMS IN INDONESIA
JAPAN INTERNATIONAL COOPERATION AGENCY	



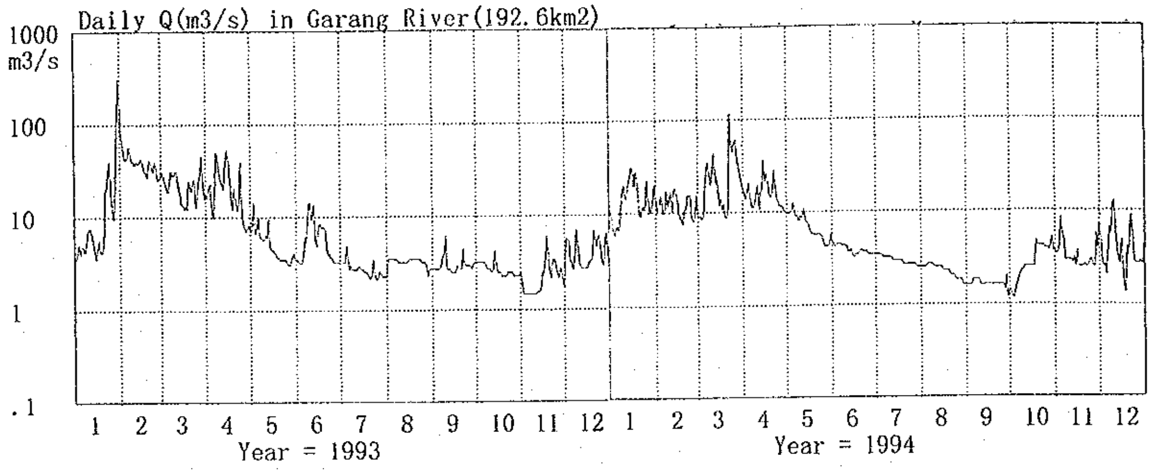
THE DETAILED DESIGN OF FLOOD CONTROL, URBAN DRAINAGE AND WATER RESOURCES DEVELOPMENT IN SEMARANG IN THE REPUBLIC OF INDONESIA	Fig. 4.3.1 THIESSEN POLYGON FOR LOW FLOW ANALYSIS
JAPAN INTERNATIONAL COOPERATION AGENCY	

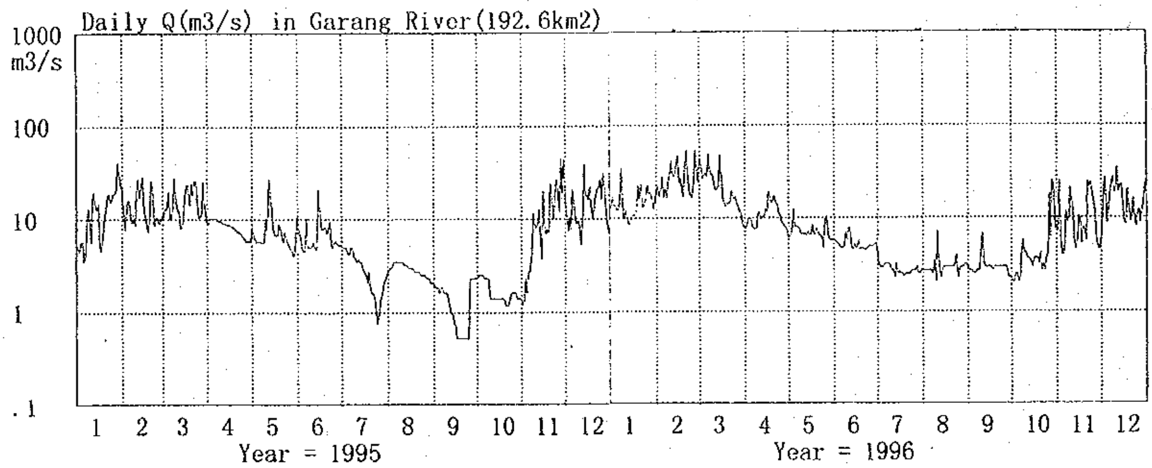






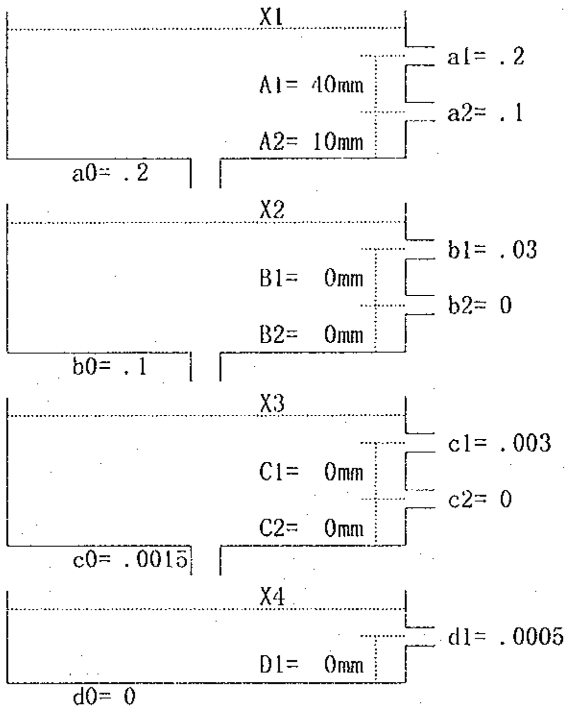
THE DETAILED DESIGN OF FLOOD CONTROL, URBAN DRAINAGE AND WATER RESOURCES DEVELOPMENT IN SEMARANG IN THE REPUBLIC OF INDONESIA	Fig.4.3.2 (1/2) DAILY DISCHARGE CHART OBSERVED AT PANJANGAN STATION IN GARANG RIVER
JAPAN INTERNATIONAL COOPERATION AGENCY	





THE DETAILED DESIGN OF FLOOD CONTROL, URBAN DRAINAGE AND WATER RESOURCES DEVELOPMENT IN SEMARANG IN THE REPUBLIC OF INDONESIA	Fig.4.3.2 (2/2) DAILY DISCHARGE CHART OBSERVED AT PANJANGAN STATION IN GARANG RIVER
JAPAN INTERNATIONAL COOPERATION AGENCY	

Tank-Model of Garang River (A=192.6km2)



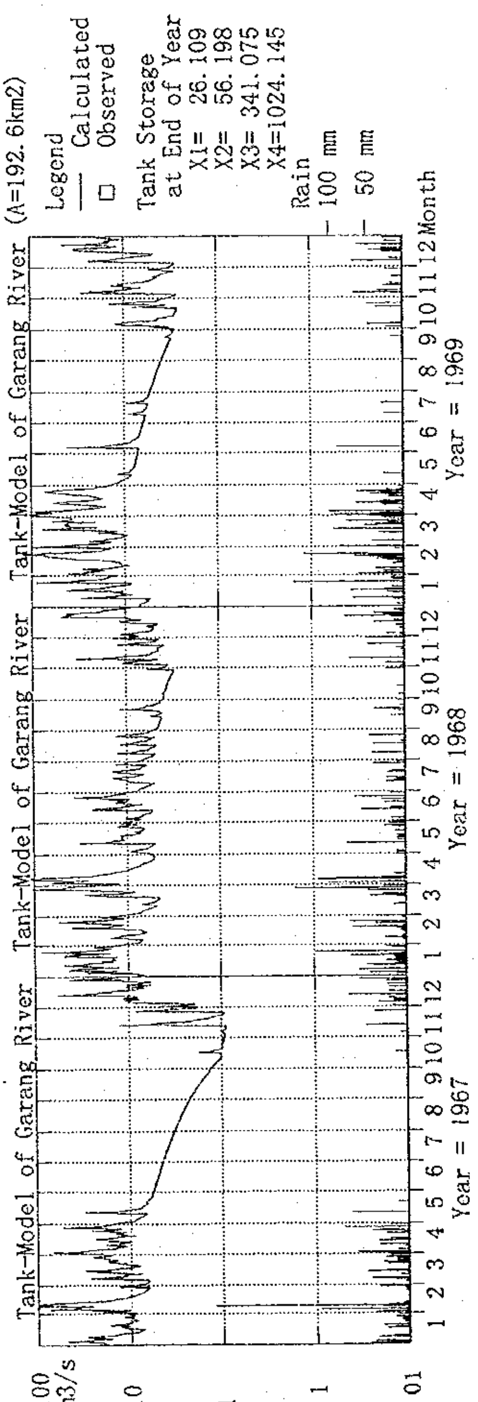
Catchment Area = 192.6 km2
Lag Time(Rain vs Q) = 0 day

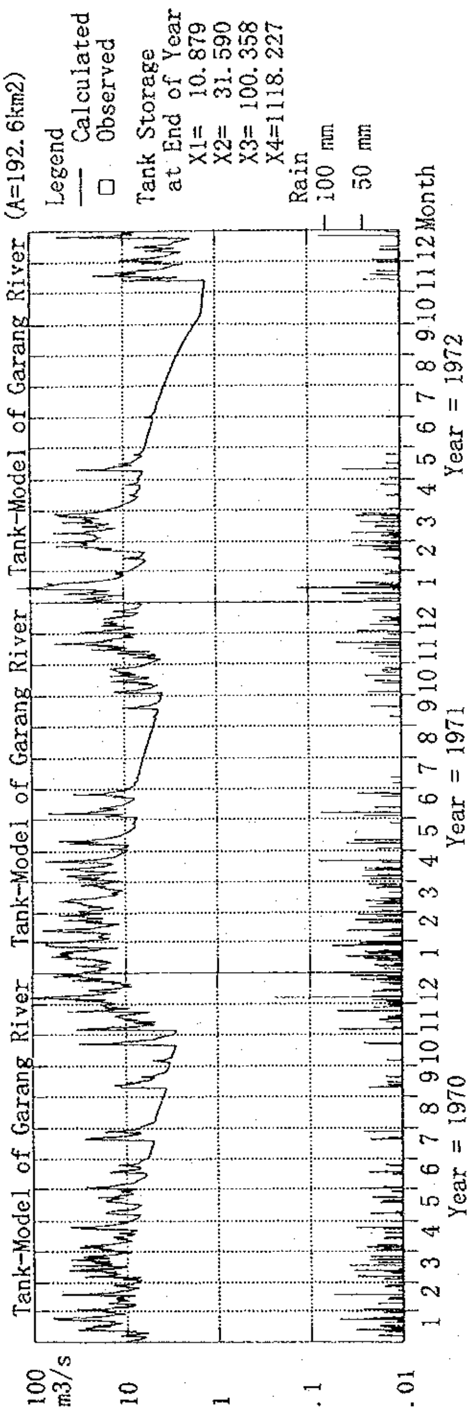
Initial Storage (Year=1967)
X1= 0 mm
X2= 90 mm
X3= 330 mm
X4= 840 mm

Month	Evaporation
1	2.42 mm/d
2	2.72 mm/d
3	2.79 mm/d
4	2.94 mm/d
5	3.16 mm/d
6	3.30 mm/d
7	3.52 mm/d
8	3.74 mm/d
9	4.18 mm/d
10	4.04 mm/d
11	3.23 mm/d
12	2.79 mm/d

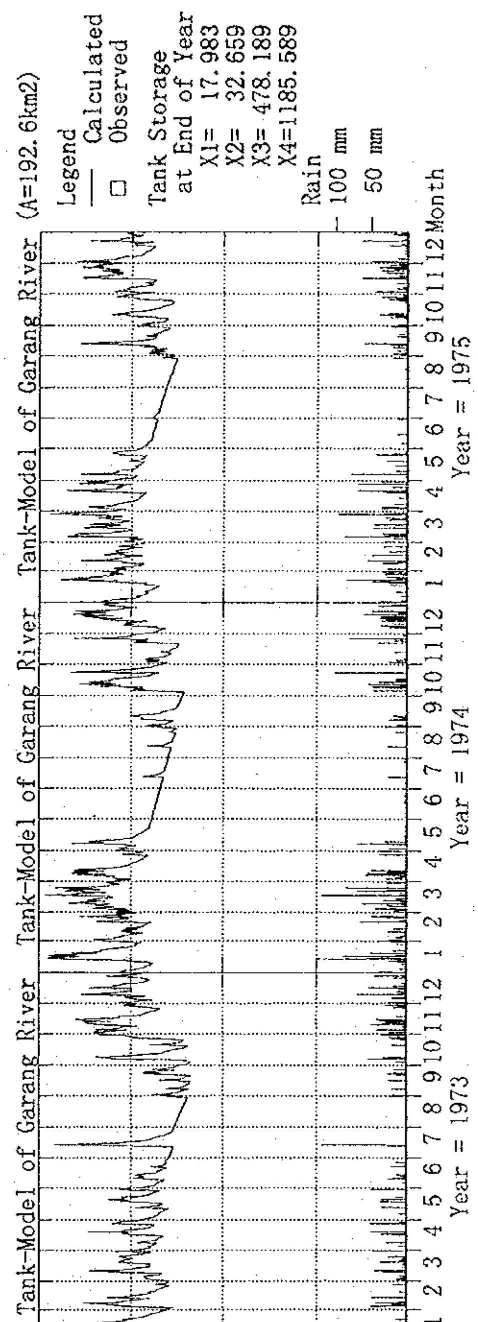
Rainfall = Input * 0.99

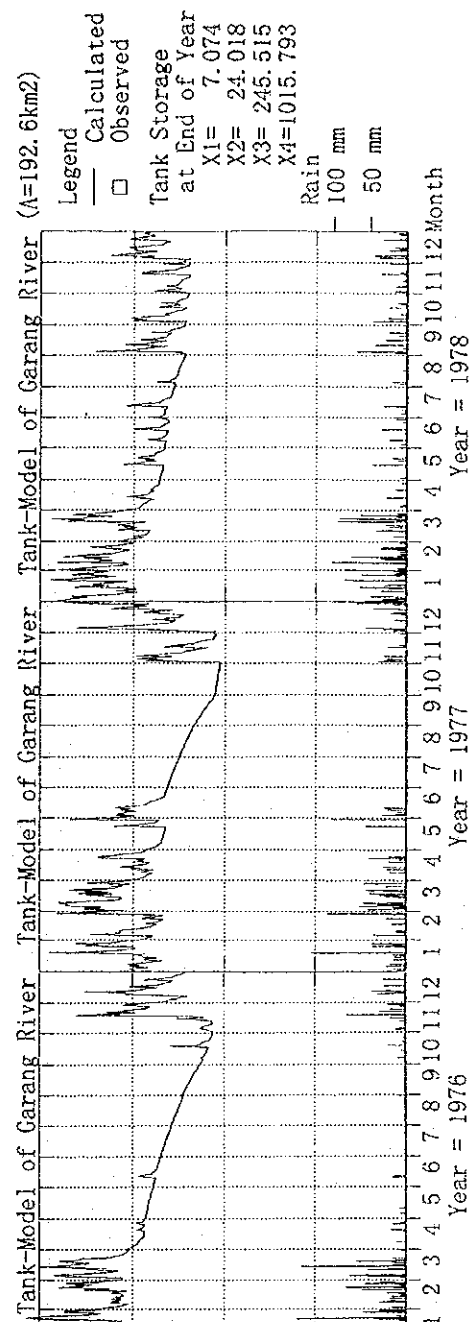
Fig.4.3.3 TANK MODEL PARAMETERS



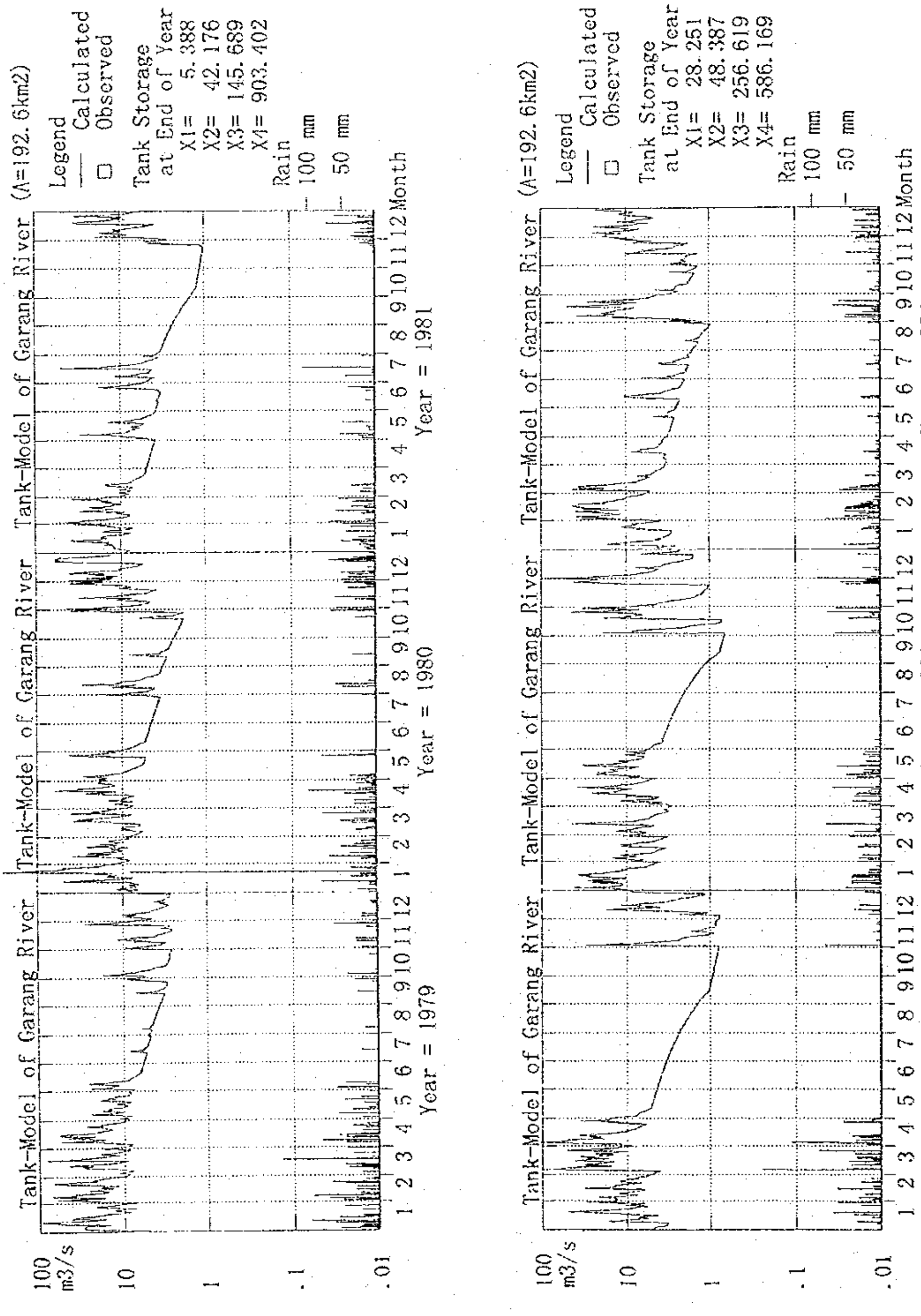


THE DETAILED DESIGN OF FLOOD CONTROL, URBAN DRAINAGE AND WATER RESOURCES DEVELOPMENT IN SEMARANG IN THE REPUBLIC OF INDONESIA	Fig. 4.3.4 (1/5) RESULT CHART OF TANK MODEL SIMULATION
JAPAN INTERNATIONAL COOPERATION AGENCY	

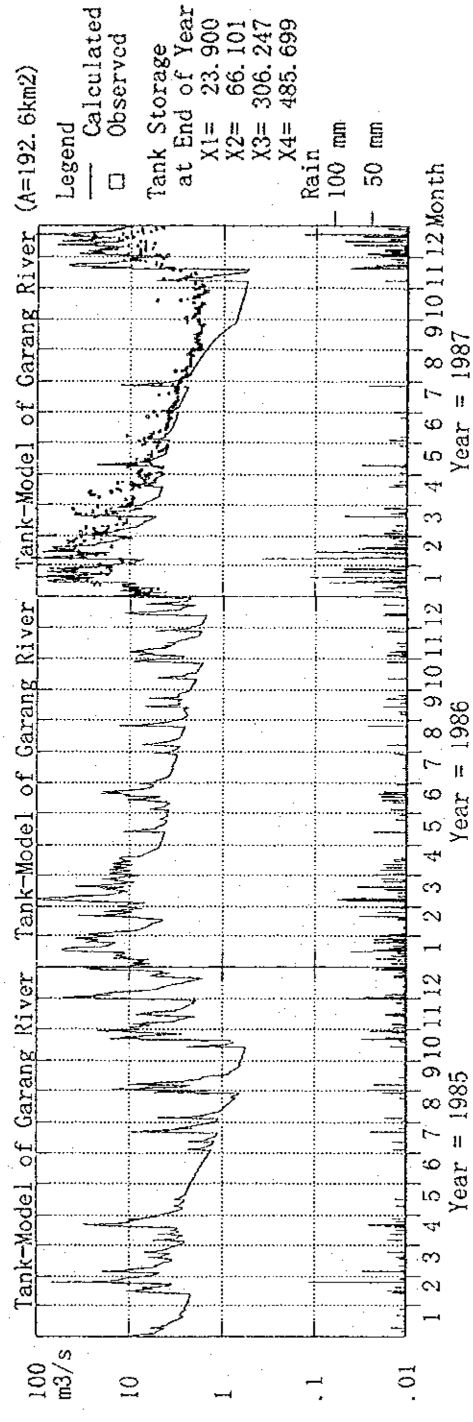


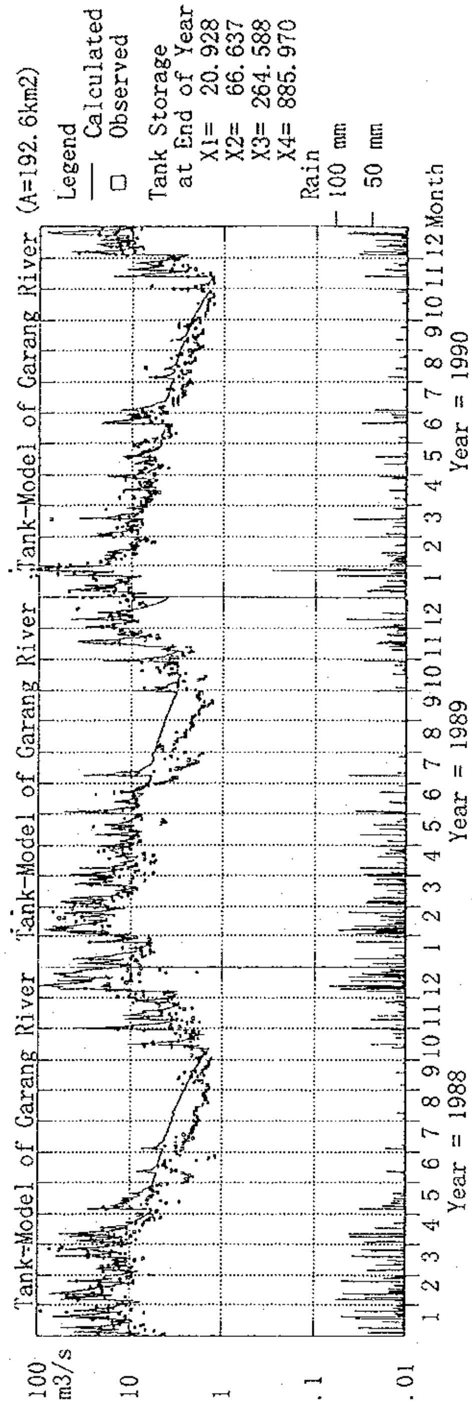


THE DETAILED DESIGN OF FLOOD CONTROL, URBAN DRAINAGE AND WATER RESOURCES DEVELOPMENT IN SEMARANG IN THE REPUBLIC OF INDONESIA	Fig. 4.3.4 (2/5) RESULT CHART OF TANK MODEL SIMULATION
JAPAN INTERNATIONAL COOPERATION AGENCY	

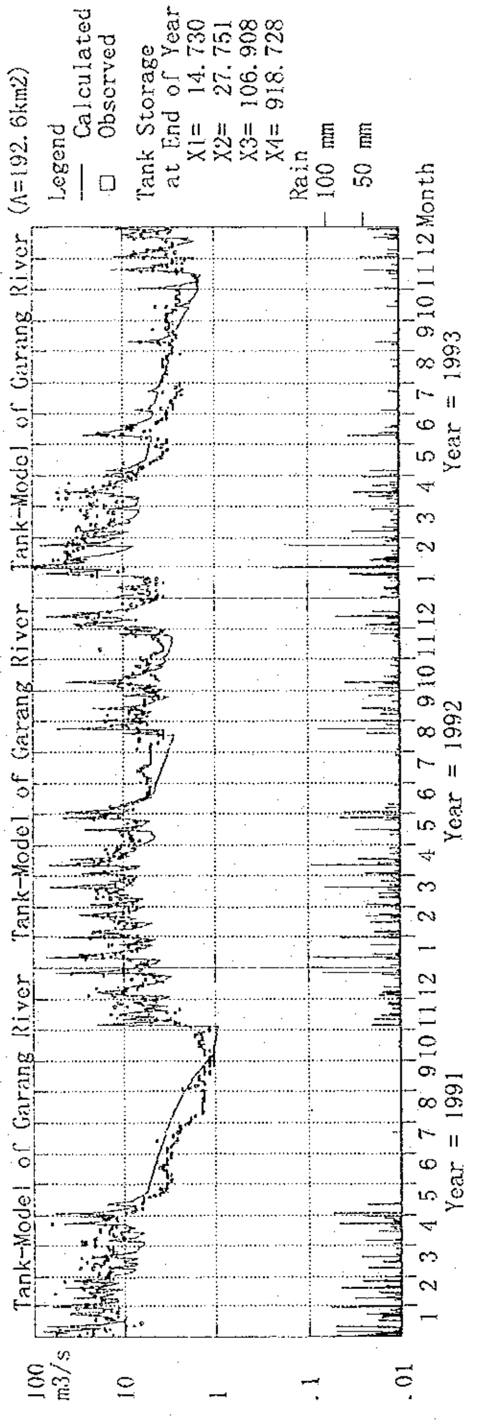


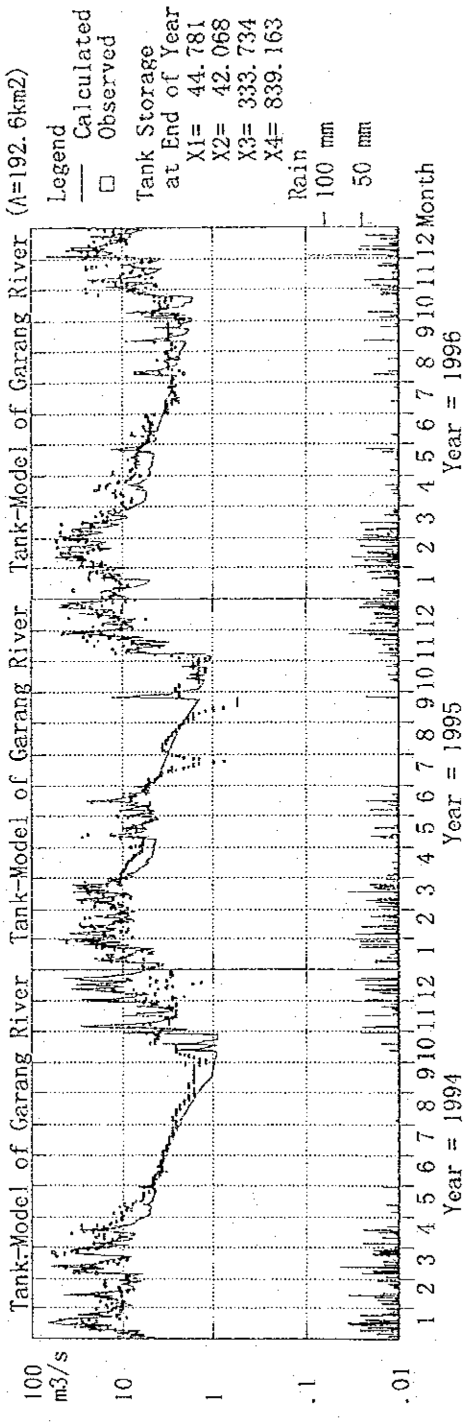
THE DETAILED DESIGN OF FLOOD CONTROL, URBAN DRAINAGE AND WATER RESOURCES DEVELOPMENT IN SEMARANG IN THE REPUBLIC OF INDONESIA	Fig. 4.3.4 (3/5) RESULT CHART OF TANK MODEL SIMULATION
JAPAN INTERNATIONAL COOPERATION AGENCY	



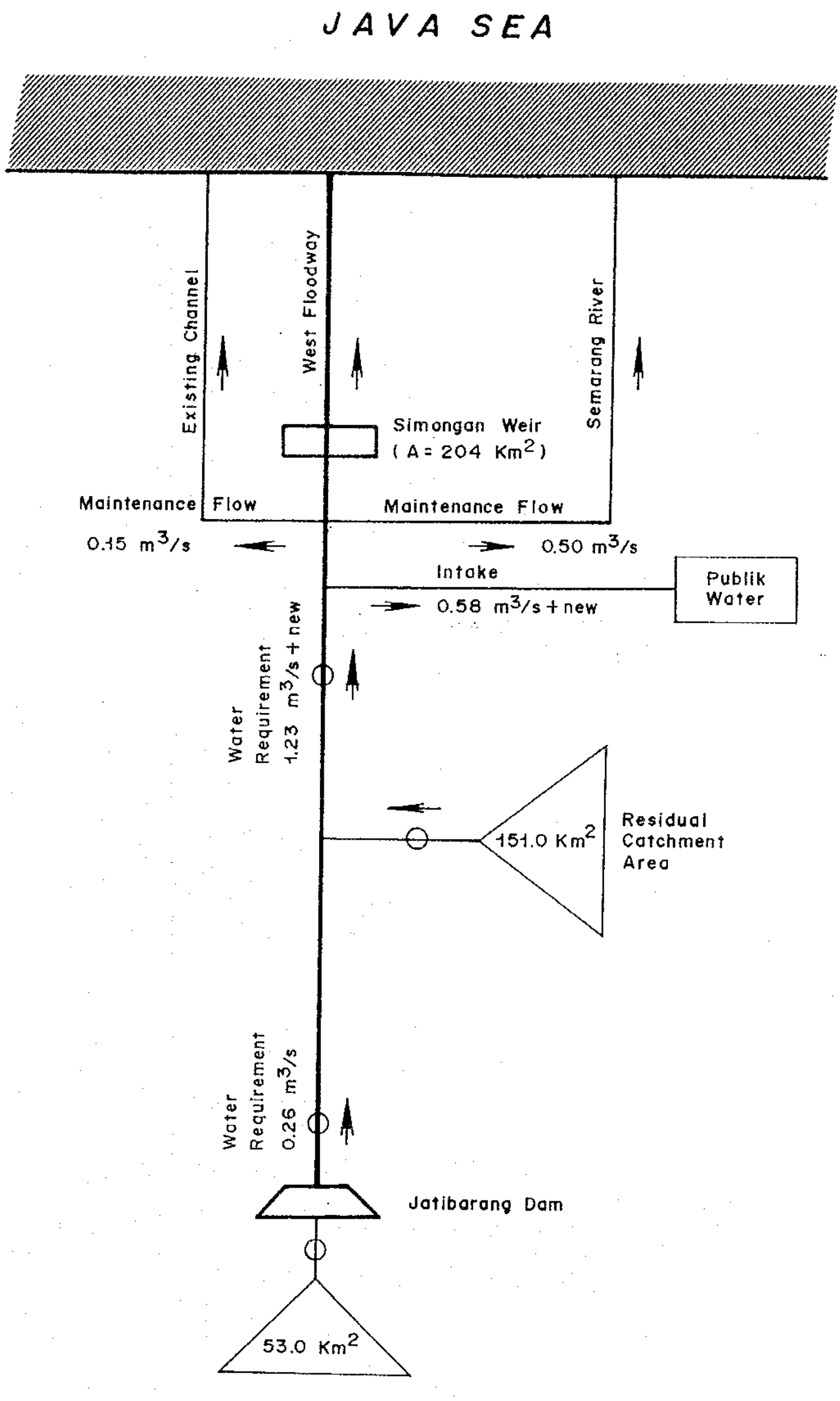


THE DETAILED DESIGN OF FLOOD CONTROL, URBAN DRAINAGE AND WATER RESOURCES DEVELOPMENT IN SEMARANG IN THE REPUBLIC OF INDONESIA	Fig. 4.3.4 (4/5) RESULT CHART OF TANK MODEL SIMULATION
JAPAN INTERNATIONAL COOPERATION AGENCY	

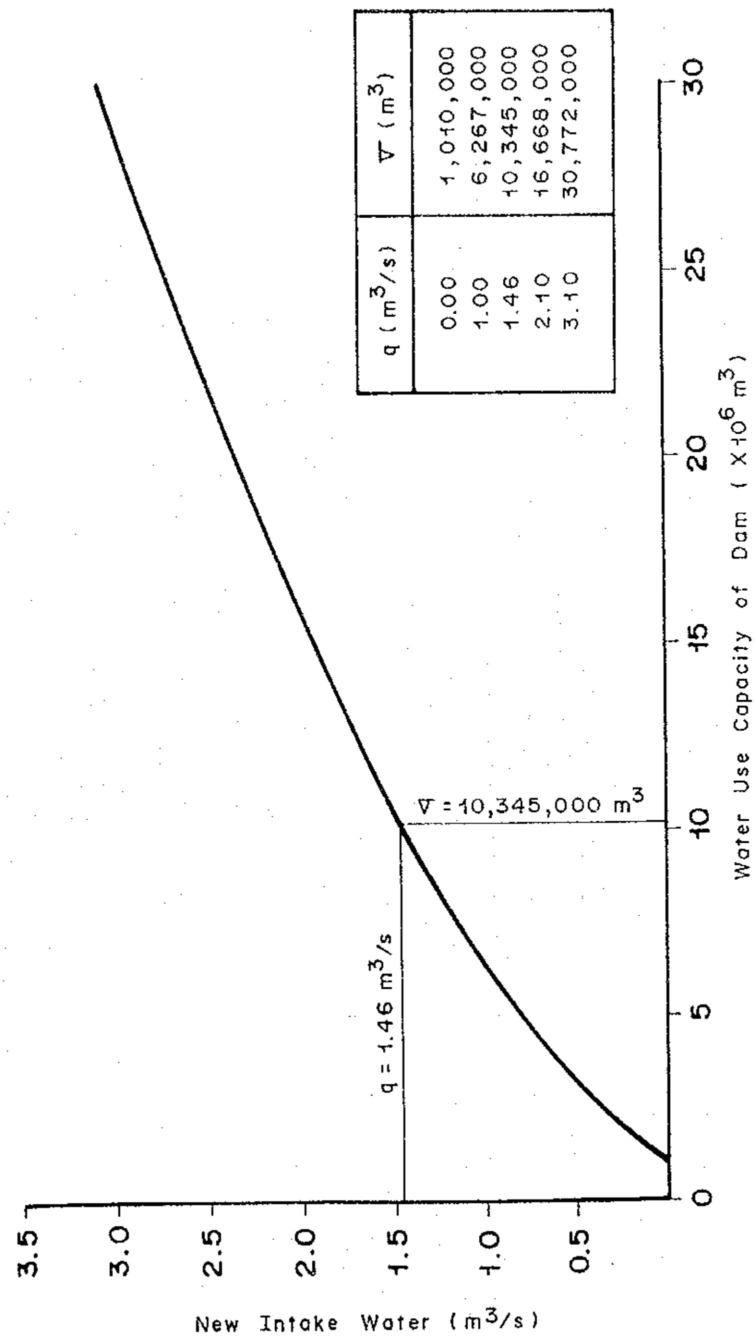




THE DETAILED DESIGN OF FLOOD CONTROL, URBAN DRAINAGE AND WATER RESOURCES DEVELOPMENT IN SEMARANG IN THE REPUBLIC OF INDONESIA	Fig. 4.3.4 (5/5) RESULT CHART OF TANK MODEL SIMULATION
JAPAN INTERNATIONAL COOPERATION AGENCY	



THE DETAILED DESIGN OF FLOOD CONTROL, URBAN DRAINAGE AND WATER RESOURCES DEVELOPMENT IN SEMARANG IN THE REPUBLIC OF INDONESIA	Fig. 4.3.5 SCHEMATIC DIAGRAM FOR WATER USE SIMULATION
JAPAN INTERNATIONAL COOPERATION AGENCY	

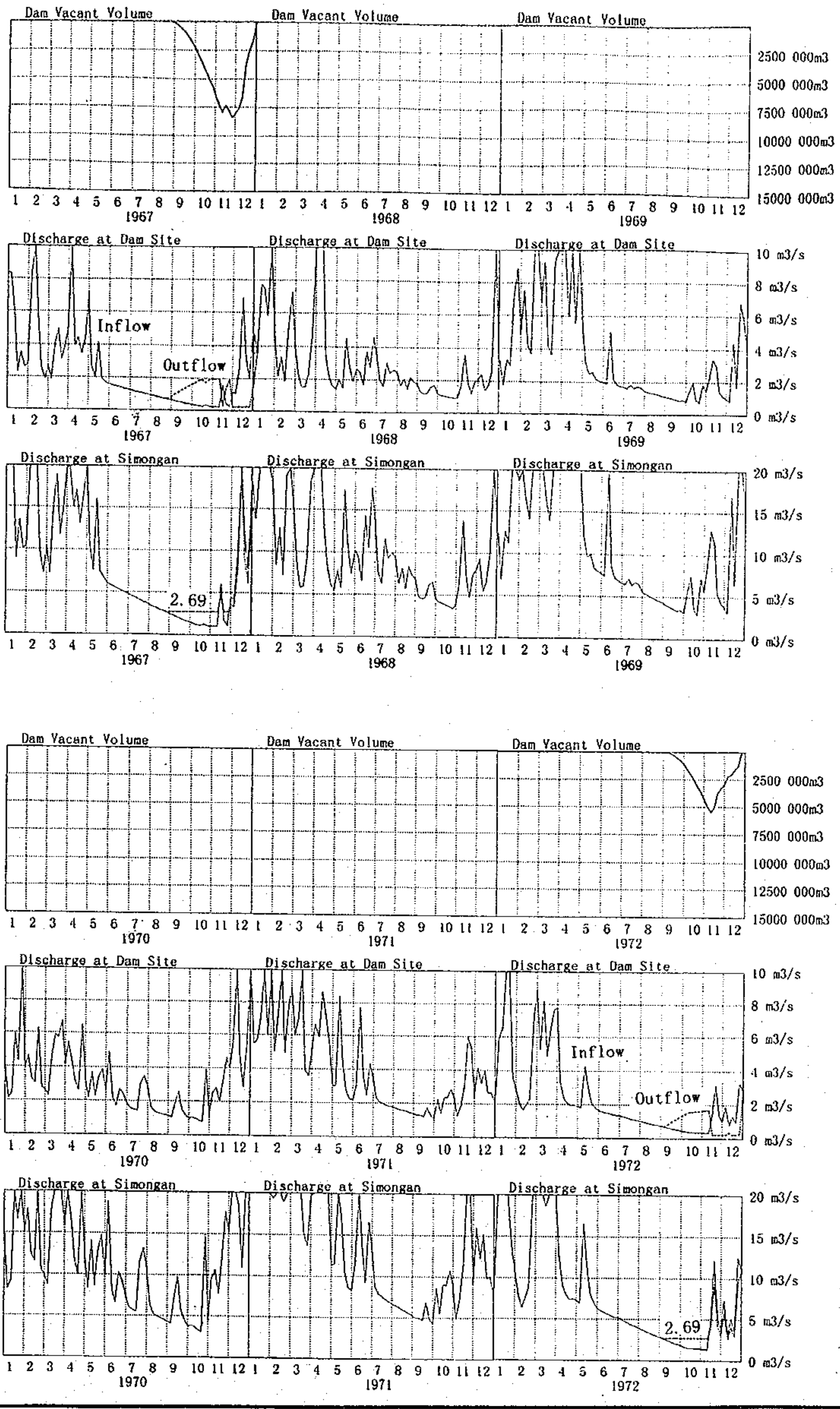


q (m^3/s)	∇ (m^3)
0.00	1,010,000
1.00	6,267,000
1.46	10,345,000
2.10	16,668,000
3.10	30,772,000

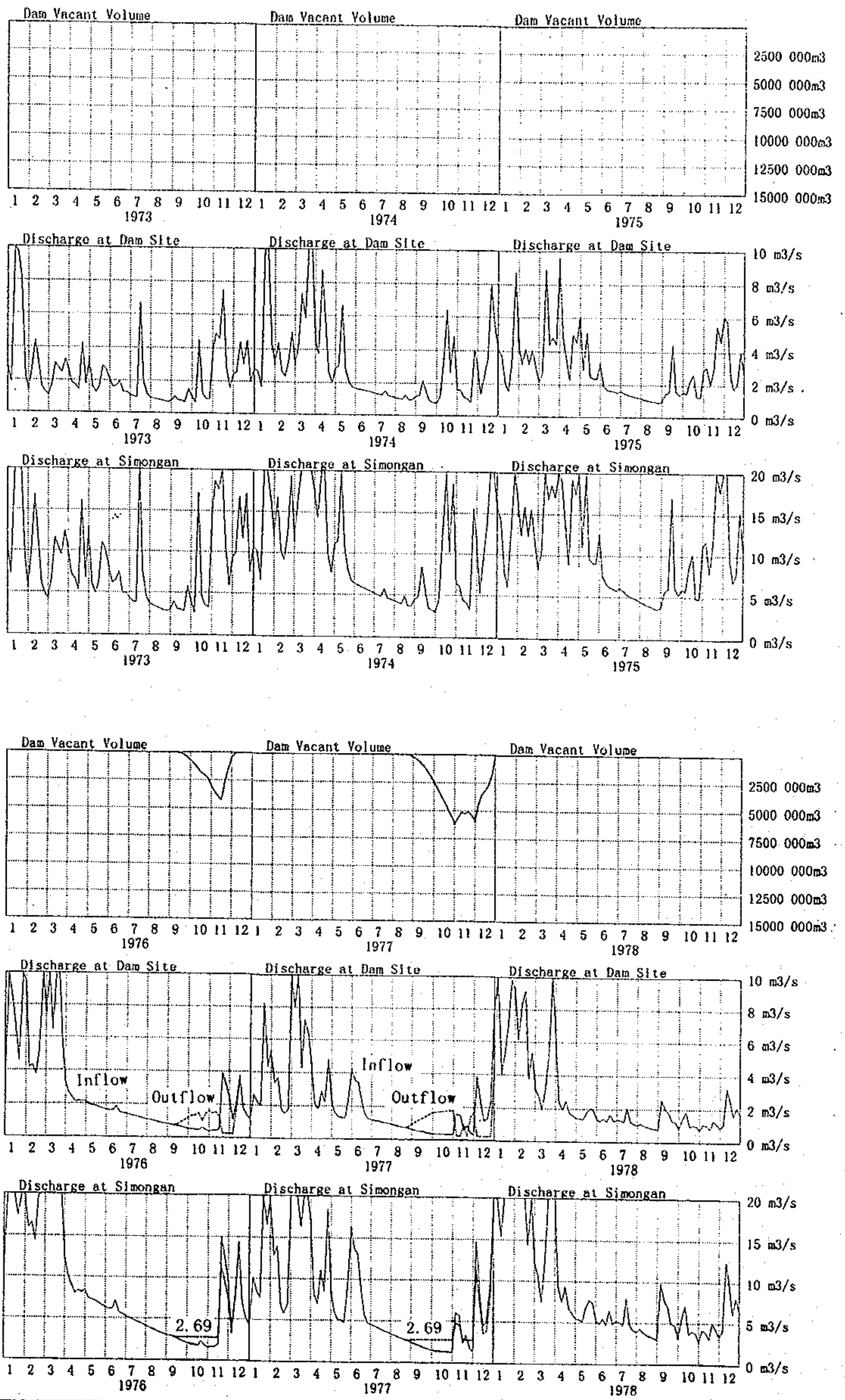
THE DETAILED DESIGN OF FLOOD CONTROL, URBAN DRAINAGE AND WATER RESOURCES DEVELOPMENT IN SEMARANG IN THE REPUBLIC OF INDONESIA

JAPAN INTERNATIONAL COOPERATION AGENCY

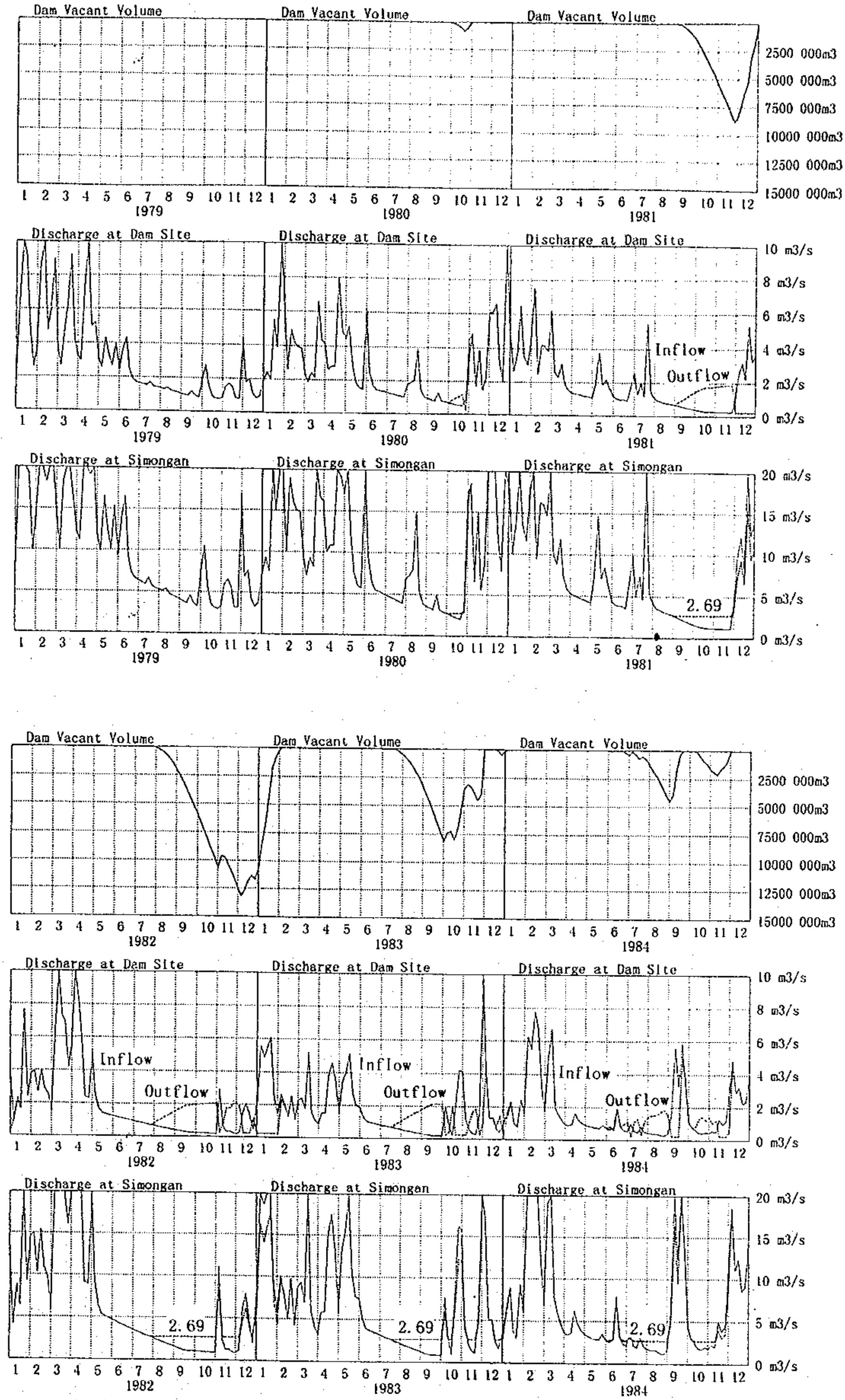
Fig. 4.3.6
RELATION BETWEEN NEW INTAKE WATER QUANTITY AND WATER USE CAPACITY OF DAM



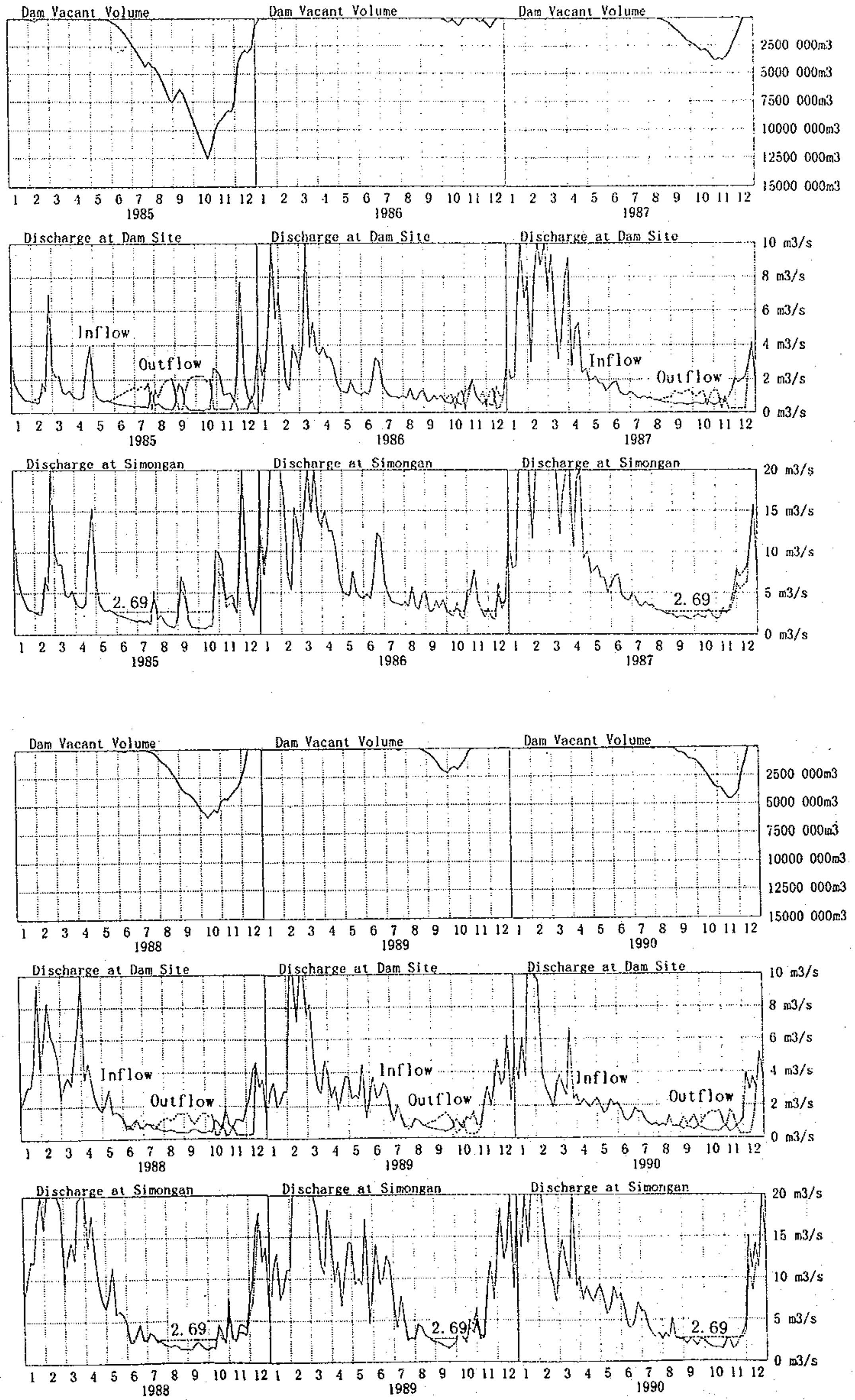
THE DETAILED DESIGN OF FLOOD CONTROL, URBAN DRAINAGE AND WATER RESOURCES DEVELOPMENT IN SEMARANG IN THE REPUBLIC OF INDONESIA	Fig. 4.3.7 (1/5) JATIBARANG DAM RESERVOIR OPERATION AND FLOW CONDITIONS
JAPAN INTERNATIONAL COOPERATION AGENCY	



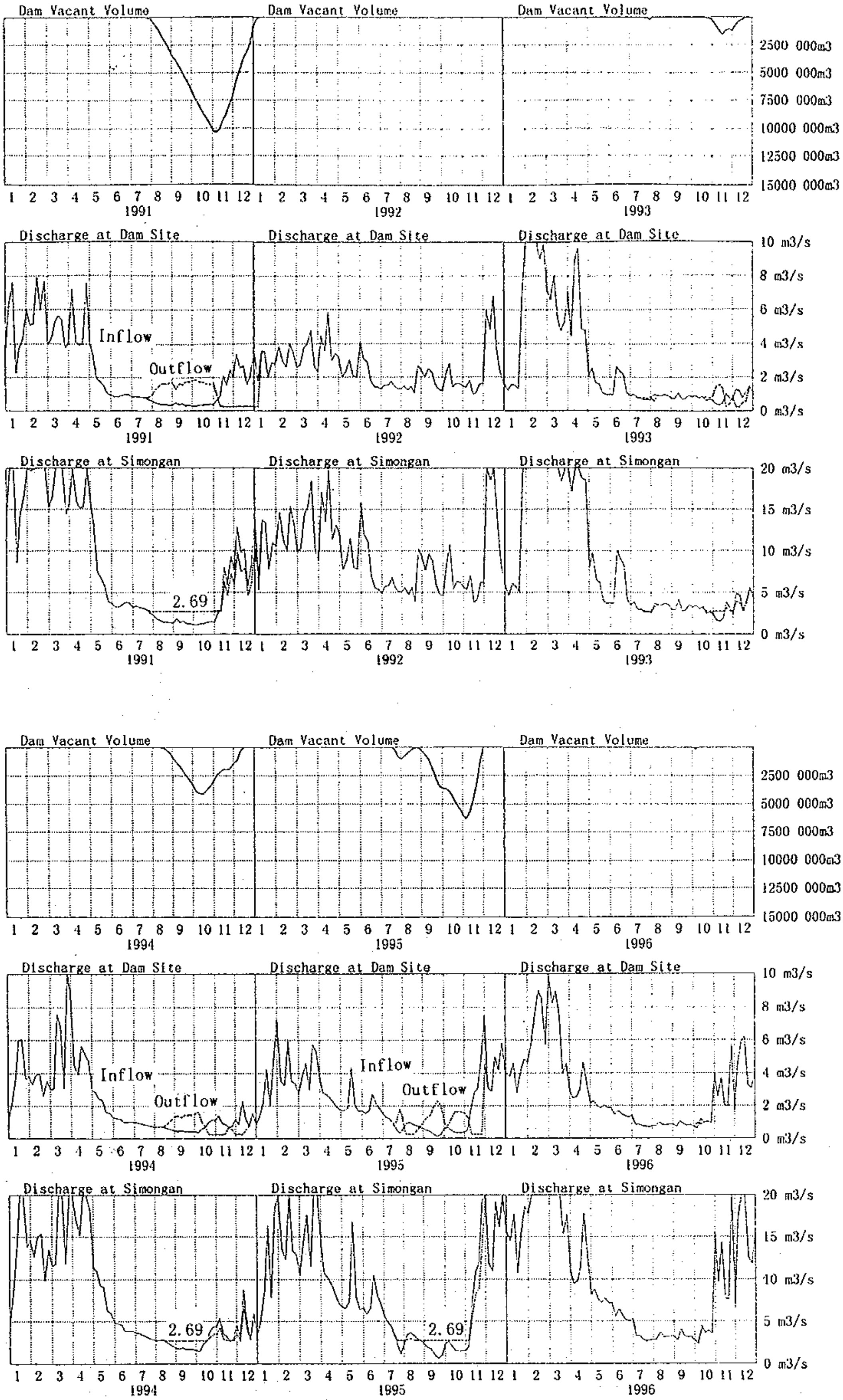
THE DETAILED DESIGN OF FLOOD CONTROL, URBAN DRAINAGE AND WATER RESOURCES DEVELOPMENT IN SEMARANG IN THE REPUBLIC OF INDONESIA	Fig. 4.3.7 (2/5) JATIBARANG DAM RESERVOIR OPERATION AND FLOW CONDITIONS
JAPAN INTERNATIONAL COOPERATION AGENCY	



THE DETAILED DESIGN OF FLOOD CONTROL, URBAN DRAINAGE AND WATER RESOURCES DEVELOPMENT IN SEMARANG IN THE REPUBLIC OF INDONESIA	Fig. 4.3.7 (3/5) JATIBARANG DAM RESERVOIR OPERATION AND FLOW CONDITIONS
JAPAN INTERNATIONAL COOPERATION AGENCY	



THE DETAILED DESIGN OF FLOOD CONTROL, URBAN DRAINAGE AND WATER RESOURCES DEVELOPMENT IN SEMARANG IN THE REPUBLIC OF INDONESIA	Fig. 4.3.7 (4/5) JATIBARANG DAM RESERVOIR OPERATION AND FLOW CONDITIONS
JAPAN INTERNATIONAL COOPERATION AGENCY	



THE DETAILED DESIGN OF FLOOD CONTROL, URBAN DRAINAGE AND WATER RESOURCES DEVELOPMENT IN SEMARANG IN THE REPUBLIC OF INDONESIA	Fig. 4.3.7 (5/5) JATIBARANG DAM RESERVOIR OPERATION AND FLOW CONDITIONS
JAPAN INTERNATIONAL COOPERATION AGENCY	

CHAPTER 5

GEOLOGY

CHAPTER 5 GEOLOGY

5.1 Geology of Damsite

5.1.1 Geological Survey

In the Phase 1 period from August 1997 to March 1998, the study on the definitive plan of Jatibarang Multipurpose Dam was conducted as the concrete gravity type, which was a conclusion of the Feasibility Study conducted in 1993. However, as a result of the geological survey conducted in the Phase 1 study period, it was concluded that the gravity type dam could not be applied for Jatibarang Multipurpose Dam because of the lack of the shear strength of the foundation rock. Then the dam type was changed to a rockfill type. Therefore the additional geological study of Jatibarang Multipurpose Dam for the type of rockfill dam was carried out in the Phase 2 period from July 1998 to January 1999.

The work items and quantities of geological survey at the dam in the study periods of Phase 1 and 2 are shown in the following table.

Study Period	Work Item	Work Quantities
Phase 1	Core Drilling (D=66mm)	24 holes, 1,699 m in total
	Lugeon Test in Borehole	19 holes, 245 times in total
	Trench Excavation	3 trenches, 638 m in total
	Adit Excavation	4 adits, 110 m in total
	In-situ Shearing Test	2 adits, 8 times in total
	Seismic Prospecting	2 lines, 1,300 m in total
	Laboratory Test of Core Sample	24 samples
Phase 2	Core Drilling (D = 66mm)	6 holes, 505 m in total
	Lugeon Test in Borehole	3 holes, 52 times in total
	Loading Test in Borehole	2 holes, 20 times in total
	In-situ Plate Loading Test (at the existing adits)	3 adits, 6 times in total

The locations of bore holes, trenches and adits are shown in Fig. 5.1.1 and the details of each investigation items and their work quantities are shown in Table 5.1.1.

5.1.2 Stratigraphy

The foundation rock of the Jatibarang damsite consists of Damar formation from the latter period of Tertiary to Quaternary. Riverbed deposit and Talus deposit are distributed as secondary sediment. The geological strata at the damsite is shown in Table 5.1.2, and the geological map and the geological profile at the damsite are shown in Figs. 5.1.1 and 5.1.2.

Damar formation at the damsite formulate alternation which consists of various sedimentary rocks and pyroclastic rocks, however it can be divided into two (2) strata of pyroclastic rock units and three strata of sedimentary rock units. They show almost level geological structure.

Pyroclastic rock unit mainly consists of volcanic breccia, and contains mafic tuff and andesite lava partly. Volcanic breccia composed of main part of the geological stratum is homogeneous. Crack hardly develops into the mother-rock, and hardness of rock is comparatively high. However, lower pyroclastic rock unit contains organic material partly, degree of cementation of the volcanic breccia is different at upper and lower pyroclastic rock units.

Sedimentary rock unit is formed by complicated alternation which consists of conglomerate, conglomeratic sandstone, tuffaceous sandstone and sandstone mainly. The thickness of each bed is ranged between several cm and several m, and the faces changes laterally remarkably. The hardness of the mother-rock is different at upper, middle and lower sedimentary rock units. Lower sedimentary rock unit which exists 40 m deep below the riverbed is soft and shows low degree of cementation, and existence of confined groundwater is confirmed.

5.1.3 Dam Foundation Rock Conditions

Strength of Foundation Rock

(1) Rock Classification

The foundation rock distributed at the damsite consists of soft rock. It is classified into four (4) classes based on their hardness and cementation, namely D class as the worst class, CL class, CM-L class and CM-H class as the best class. The rock classification of the damsite is shown in the table below. Rock classes of the dam axis section are shown in Fig. 5.1.3.

Classification	Characteristics
D	Completely weathered to a reddish soil without remains of mother-rock texture. Very low cementation part observed in the boring core, which is observed like sand and gravel. Core recovery ratio ranges from 0 % to 50 %.
CL	Moderately to highly weathered to a brown soil although remains of mother-rock texture. The mineral grains are not decomposed. Low cementation part observed in a boring core, which is observed like sand and gravel. Core recovery ratio is more than 70 %.
CM-L	Slightly weathered to fresh rock that consists of mainly tuff, tuffaceous sandstone, sandstone and conglomerate. Rock fragment is slightly soft. This class with moderate degree of cementation contains the fresh part and relatively weathered part, but they have the almost same hardness.
CM-H	Slightly weathered to fresh rock that consists of mainly volcanic breccia, conglomeratic sandstone and volcanic conglomerate. Rock fragment is relatively hard. This class with high degree of cementation contains the fresh part and the relatively weathered part, but they have almost same hardness.

(2) Mechanical Property of Rocks

The laboratory tests and the in-situ shearing tests were carried out in the Phase 1 stage, and the in-situ plate loading tests and the loading tests in boreholes were carried out in the Phase 2 stage.

The results of unconfined compression tests of boring core samples are shown together with physical properties in Table 5.1.3 and Figs. 5.1.4 and 5.1.5. The results of in-situ shearing tests in adits T-1 and T-3 are shown in Table 5.1.4 and Figs. 5.1.6 and 5.1.7. The results of in-situ plate loading tests in adits T-1, T-3 and T-4, and the results of loading tests in boreholes are shown in Tables 5.1.5 and 5.1.6.

The results of the tests are summarized in the following table.

Geological Unit	Rock Classi-fication	Rock	Ratio*1 (%)	Result of Tests: Unconfined Compressive Strength (kgf/cm^2)	Elastic Modulus (Deformation Modulus): Boreholes (kgf/cm^2)	Elastic Modulus (Deformation Modulus): Adits (kgf/cm^2)	Shear Strength (tf/m^2) and (f)
Upper Sedimentary Rock EL.180m ~ EL.120m	CL	Tuff		6	600-5,800 (400-3,400)		
		Tuffaceous Sandstone			1,100-1,400 (500-700)	2,900-8,600 (2,100-6,300)	20-50 (0.7)
		Sandstone			1,500 (600)	4,200-7,500 (1,800-2,000)	
	CM-L	Tuff		14 – 27	7,500 (2,800)		
		Conglomeratic Sandstone		43			
		Conglomerate		56			
		Sandstone			17,200 (9,700)		
Upper Pyroclastic Rock EL120m ~ EL.105m	CM-H	Volcanic Breccia		66 – 92	4,700-8,000 (2,700-4,300)		
Middle Sedimentary Rock EL.105m ~ EL.80m	CM-L	Tuff	20	13 – 27			
		Tuffaceous Sandstone	5	8 – 35			
		Sandstone	45	19	4,300-50,000 (2,700-15,000)	9,300 (5,600)	33 (0.8)
		Conglomeratic Sandstone	5	10 – 65			
		Conglomerate		56			
	CM-H	Conglomeratic Sandstone	25	43 – 64		13,200 (6,100)	85 (0.8)
Lower Pyroclastic Rock EL.80m ~ EL.50m	CM-H	Volcanic Breccia		22 – 29	2,200-11,000 (1,500-7,600)		

Note : *1 : The existing ratio of every rock in the middle sedimentary rock was estimated by boring data.
τ_0 : Shear strength
f : Coefficient of internal friction

As described before, the foundation rock is classified into four (4) classes. However, the mechanical property test results of each rock class are scattered in a wide range and the test results do not always represent the rock class. This comes from the complicated alternate layers which consist of different single layers in terms of gradation and compressive strength. Rock classification is made to express an average compressive strength of complicated alternate layers.

Judging from test results, compressive strength of CL class rock which is highly weathered is considered to be gradually increased as the depth become deeper. The test results of CM-L and CM-H classes show the original strength of the mother-rock which is not weathered.

The test results of the both CM-L and CM-H classes show only slight difference. However, CM-H class consists of alternate layer composed of single layers of relatively large compressive strength or homogeneous and massive thick layers. Therefore, CM-H class is judged to have bigger strength than CM-L class, however there is no remarkable difference of strength.

(3) Estimated Shear Strength, Elastic Modulus and Deformation Modulus

The estimated shear strength and modulus of each rock unit were decided by the results of the rock tests, in-situ shearing tests and loading tests for each geological unit and rock class. The estimated values are summarized bellow.

Geological Unit	Rock Class	Elastic Modulus	Deformation Modulus	Estimated Shear Strength
Upper Sedimentary Rock	CL class	3,000 – 5,000 kgf/cm^2	1,500 – 3,000 kgf/cm^2	τ_o=30 tf/m^2 (f=0.7)
	CM-L class	9,000 – 12,000 kgf/cm^2	5,000 – 7,000 kgf/cm^2	τ_o=45 tf/m^2 (f=0.8)
Upper Pyroclastic Rock	CM-H class	-	-	τ_o=50 tf/m^2 (f=0.8)
Middle Sedimentary Rock	CM-L class and CM-H class	-	-	τ_o=45 tf/m^2 (f=0.8)
Lower Pyroclastic Rock	CM-H class	-	-	τ_o=50 tf/m^2 (f=0.8)

Note f : coefficient of internal friction

The background of the decision is described hereinafter.

① From the results of unconfined compressive strength tests, the shear strength of the lower pyroclastic rock unit belonging to CM-H class was presumed to be between the values of fine sandstone belonging to CM-L class and conglomeratic sandstone belonging to CM-H class. The shear strength of lower pyroclastic rock unit is set at τ_o = 50 tf/m^2, coefficient of internal friction f = 0.8 on the presumption that it is directly proportional to their unconfined compressive strength test results.

② In the middle sedimentary rock unit, complicated alternation of CM-L class and CM-H class is observed. From the results of unconfined compressive strength tests, shear strength of CM-L class rock, which mainly consists of tuff, tuffaceous sandstone and sandstone, can be set at τ_o = 33 tf/m^2, coefficient of internal friction f = 0.8 (test results of fine sandstone), and shear strength of CM-H class rock, which consists of conglomeratic sandstone, is set at τ_o = 85 tf/m^2, coefficient of internal friction f=0.8 (test results of conglomeratic sandstone). The constituent ratio of each rock class is acquired from boring data, then the average shear strength of the unit is calculated and set at τ_o = 45 tf/m^2, coefficient of internal friction f = 0.8.

③ The unconfined compressive strength of upper pyroclastic rock unit is apparently bigger than the one of sedimentary rocks which belongs to CM-L class and conglomeratic sandstone which belongs to CM-H class. But there is no difference on the results of loading tests. Then the shear strength of the unit is set at same value of the one in the lower pyroclastic rock unit.

④ The shear strength of CM-L class rock in the upper sedimentary rock unit is set at the same value of the one in the middle sedimentary rock unit.

⑤ The shear strength of CL class rock, which consists of tuffaceous sandstone, in the upper sedimentary rock unit is set at τ_o = 30 tf/m^2, coefficient of internal friction f = 0.7 considering the average of the test results.

⑥ The test results of the elastic modulus and deformation modulus of CM-L class and CM-H class are ranged widely at each geological unit and the values of the test results are overlapped each other. Therefore, it is judged that there is no difference of elastic modulus and deformation modulus between CM-L class and CM-H class. The elastic modulus and deformation modulus are represented by the values of 9,000 to 12,000 kgf/cm^2 for elastic modulus and 5,000 to 7,000 kgf/cm^2 for deformation modulus considering the average of the test results.

⑦ The elastic modulus of CL class in the upper sedimentary rock unit shows 2,900 to 8,600 kgf/cm^2 by the plate loading tests and 600 to 5,800 kgf/cm^2 by the loading test in boreholes. Though the result show different values by the test methods, the values of both test results show smaller than that of the ones of CM-L and CM-H classes. The average values of the elastic modulus ranges 3,000 to 5,000 kgf/cm^2 and deformation modulus ranges 1,500 to 3,000 kgf/cm^2 in CL class of upper sedimentary rock unit.

Permeability

The results of lugeon tests in boreholes are shown in Table 5.1.7 and Figs. 5.1.8 and 5.1.9. The distribution of lugeon value at the dam axis section is shown Fig. 5.1.10.

The permeability of the foundation rock at the damsite is described below based on the lugeon test results.

(1) Riverbed

The permeability of the lower pyroclastic rock unit that exists between EL. 50 m and EL. 80 m is small. Especially, the lugeon value of this rock unit between EL. 50 m and EL. 60 m is less than 5 lugeon.

The permeability of the lower sedimentary rock unit that exists deeper than EL. 50 m shows more than 20 lugeon and confined groundwater with maximum pressure of 2.0 kgf/cm^2 is confirmed.

(2) Left and Right Abutments

The permeability of the upper pyroclastic rock unit and middle sedimentary rock unit which are distributed between EL. 80 m and EL. 120 m shows less than 10 lugeon. Seventeen percent of the lugeon test results in them shows average critical pressure of 6 kgf/cm^2.

At the left and right abutments, less pervious rock zone with less than 5 lugeon rises up to as high as the normal water level of EL. 148.9 m. The groundwater level at the right abutment has observed higher than the normal water level. However, the groundwater level has not yet observed at the level higher than the normal water level at the left abutment.

Conclusion

From the riverbed to the halfway up to the slope at the damsite, fresh to slightly weathered rock which belongs to CM-L or CM-H classes is distributed at the depth less than several meters from the ground surface and the rock is judged to have enough bearing capacity as the foundation rock for the rockfill type dam with the height of 80 m.

There exist weathered rock which is classified CL class with thickness of maximum 30 m from the ground surface at the upper slope of both abutments. The surface layer of abutment with 10 m deep which is highly weathered shall be removed, and the layer below highly weathered rock is judged to have enough bearing capacity as the foundation rock of the dam abutments.

At the riverbed of the damsite with the elevation of about EL. 75 m, sedimentary rock with low solid state of around 3 m deep exists within the lower pyroclastic rock unit. This sedimentary rock which is classified as CL class locally distributed horizontally and shows high permeability of more than 20 lugeon, but it is judged to have enough bearing capacity. Therefore, such water sealing works as replacement of the layer or grouting with fine cement shall be applied as a foundation treatment.

Sedimentary rock unit consists of alternate layers with faces extremely change. Since continuous cracks which might be water passages do not developed, permeability of the unit is considered to be low. However, layers with high permeability exist locally. These layers with low cementation are considered to be developed by repeated horizontal swelling and shrinkage and to be become water passages. Since the unit consists of soft rock, introduction of excessive pressure at grouting works may be cause collapse of foundation rock, introduction of pressure at grouting works should be done below a critical point.

5.2 Geology of Reservoir Area

5.2.1 Geological Survey

A saddle portion whose lowest elevation is EL. 161.69 m exists at the right bank of the reservoir. Since the creep length of the saddle portion at the Surcharge Water Surface (EL. 151.8 m) is about 150 m, leakage from the saddle portion can be presumed depending on the condition of the geology.

Gentle slopes and isolated hills are scattered at the right bank of the reservoir and the distribution of the alternate layers consist of some sedimentary rocks which covers bedrock of siltstone is considered to be topographically irregular. There is a possibility that this irregularity of the topography was caused by landslide and the area has possibility to cause landslide after impounding water in the reservoir.

The same topography is recognized at the left bank of the reservoir. However, since bedrock of siltstone is outcropped over a wide area, it is judged that this area has low possibility of landslide.

Considering above topographical conditions, geological survey with borings and excavation of trenches was conducted at the saddle portion and gentle slopes in the right bank of the reservoir.

The work items and quantities of geological survey in the reservoir area are shown in the following table.

Work Item	Work Quantities
Core Drilling (D=66 mm)	16 holes, 475 m in total
Lugeon Test in Bore Hole	5 holes, 28 times in total
Standard Penetration Test	11 holes, 310 times in total
Trench Excavation	7 trenches, 766 m in total

The locations of boreholes and trenches are shown in Fig. 5.2.1 and the details of each investigation items and their work quantities are shown in Table 5.2.1.

5.2.2 Topography and Stratigraphy

Geological map and strata at the reservoir area are shown in Table 5.2.2 and Fig. 5.2.2. Topography and stratigraphy of the reservoir area are described hereunder.

Topography

Kreo River on which Jatibarang Multipurpose Dam is planned forms a narrow valley at the damsite and flows toward the direction of south-southwest to north-northeast. However, alluvial terrace with the width of about 500 m is developed along Kreo River and the valley is wide at the upper stretch from about 400 m upstream of the damsite.

Mountains around the reservoir form topographical features of relatively flat plateaus with the elevation of about 200 m and the relative height between the riverbed and the top of the surrounding mountains ranges 80 to 100 m.

The slope gradient of the surrounding mountainside of the reservoir is relatively gentle except the slopes of both banks at the damsite. Gentle slopes and isolated hills are scattered at the halfway of the surrounding mountains. Mountains at the right bank of the reservoir forming a border with the watershed of Cebong Rive which is a tributary of Kreo River have narrow ridges at the elevation of 165 m and form saddle portion with the length of about 200 m.

Stratigraphy

The bedrock of the reservoir area consists of Kerek Formation which belongs to Miocene to Pliocene in Tertiary, Damar Formation and Kaligetas Formation which belong to Pliocene in Tertiary to Pleistocene in Quaternary. Each formation mainly consists of sedimentary rocks and quality of rock is soft.

Damar Formation is distributed at the northern side of a fault which exists 400 m upstream (southern side) from the damsite and Kerek and Kaligetas formations are distributed at the southern side of the fault.

Riverbed and terrace deposits which are secondary deposit in Holocene in Quaternary are distributed along the existing riverbed, and talus deposit is distributed at the skirt of the surrounding mountains.

(1) Bedrock

Kerek Formation consists of siltstone and includes limestone in an appearance of lens with some tens of cm in length. Siltstone is soft and tends to be deteriorated by slaking.

Kaligetas Formation consists of sedimentary rock and forms alternate layers which mainly consist of conglomerate, conglomeratic sandstone, tuffaceous sandstone and sandstone. Kaligetas Formation is distributed above the halfway of the mountainside, and generally weathered with reddish brown color and gradually transforming to soil.

Damar Formation consists of alternate layers of sedimentary rock unit, which mainly consists of tuffaceous sandstone, conglomeratic sandstone and volcanic conglomerate, and pyroclastic rock unit, which mainly consists of volcanic breccia.

The geological ages of Kerek Formation and Kaligetas Formation are different and the relationship of the geological structure of both formations is unconformity. However, according to boring and trench excavation survey, the basal conglomerate of Kaligetas Formation or the weathered zone underneath the surface of unconformity is not recognized. The surface of unconformity at the mountains on the right bank of the reservoir is undulated and does not form dip slope which may become slip surface.

(2) Fault

The fault mentioned above is recognized by boreholes and excavated trenches at the saddle portion of the right bank of the reservoir. The fault recognized at the saddle portion is small scale with the width of less than 10 cm and consists of foliated fault gouge developed with slickenside. The fault is inclined toward southwest with the angle of about 60° at the trench FS-1 and about 70° at the boring RA-5.

Kerek Formation which is distributed on the southern side of the fault is older than Damar Formation which is distributed on the northern side of the fault. The fault is presumed to be a reverse fault with upheaval of the southern side.

Kerek Formation which is expected to exist under Damar Formation is not confirmed by boreholes with the depth of 100 m (EL. 0 m) at the damsite located northern side of the fault. From this survey result, it is presumed that the possible displacement of the fault will be more than hundreds of meters.

(3) Hydrogeological Structure

Sedimentary rock unit of low degree of cementation with lugeon value of more than 20 is recognized deeper than EL. 50 m at the damsite. However, since this layer is considered not to be continued to the southern side of the fault and no permeable layers are confirmed along the fault, the possibility of seepage from the upstream of the dam to the downstream through the sedimentary rock unit is none. Furthermore, this sedimentary rock unit accompanies confined groundwater and it is judged that the pyroclastic rock units which is distributed on the sedimentary rock unit form wide range of impermeable layer.

In considering the layer of confined groundwater, it is judged that Damar Formation has a homocline which incline gently or a fold which has a gentle wave tune in the wide area. Above mentioned geological structures are illustrated in Fig. 5.2.3.

5.2.3 Result of Geological Survey

Saddle Portion at Right Bank

The saddle portion at the right bank of the reservoir forms ridge which has the length of about 200 m at the elevation of lower than 165 m and at the lowest elevation of 161.69 m. On the other hand, the Surcharge Water Surface is EL. 151.80 m and the deference of the elevation of the saddle portion and the Surcharge Water Surface is 10 m at the minimum. The width of the saddle portion at the level of EL. 151.80 m is about 150 m.

The drilling works with seven (7) bore holes were conducted at the saddle portion along the both survey lines in parallel with (BP line) and crossing (AP line) the right bank ridge. The geological profiles along the survey lines are shown in Fig. 5.2.4, and the lugeon test results are tabulated in the Table 5.2.3. The survey results are briefly described hereinafter.

① A fault is developed at the northernmost end of the saddle portion and the bedrock of the saddle portion is siltstone which belongs to Kerek Formation at the southern side of the fault, alternate layers of sedimentary rock unit which belongs to Damar Formation at the northern side of the fault.

② Siltstone, which forms the saddle portion, is fresh and soft with N value of 30 to 50 at the depth of 15 to 25 m from the ground surface.

③ The impervious rock with the lugeon value of less than 5 exist deeper than 5 to 20 m from the ground surface at the saddle portion. However, the lugeon test results show very low critical pressure of 1.54 to 4.15 kgf / cm^2 at each test section. Around the saddle portion with the elevation of less than 165 m, the pervious rock with lugeon value of more than 20 is distributed at the depth less than 10 m deeper than the Surcharge Water Surface and the groundwater level is lower than the Surcharge Water Surface.

Surrounding Slopes of Reservoir

Landslide at a slope near the saddle portion makes the loss of the length of seepage path of the ridge and, as a result, there is a possibility of increment of leakage from the reservoir. Landslide at the slope of the opposite side of the entrance of the narrow valley of Kreo River may cause big wave which damage the dam directly. Considering these issues, drilling works and trench excavation on the four (4) survey lines (CP, DP, EP and FP lines) as shown in Fig. 5.2.1 were conducted. The geological cross sections of the four (4) survey lines are shown in Figs. 5.2.5 and 5.2.6 and the survey results are described hereinafter.

(1) CP Line

A gentle slope is developed below EL. 180 m and the bedrock above EL. 180 m consists of sedimentary rock unit which belongs to Kaligetas Formation and siltstone which belongs to Kerek Formation below EL. 180 m.

Secondary deposit, which was formulated by landslide or collapse, is not distributed on the gentle slope. The geological structure of the bedrock will not cause rockslide and slip surface is not recognized by boreholes.

(2) DP Line

A gentle slope is developed above EL. 170 m and the bedrock above EL. 155 to 165 m consists of alternate layer of sedimentary rock unit which belongs to Kaligetas Formation, siltstone which belongs to Kerek Formation below the elevation.

Weathering and silting of the sedimentary rock unit which belongs to Kaligetas Formation is progressed and N values of the unit are about 10. However, disorder of layers which will be caused by rockslide is not recognized and no slip surface is found at a portion where N value increases sharply.

(3) EP Line

A gentle slope is developed at around EL. 155 m and the slope above EL. 155 m is steep with horseshoe shape. The bedrock consists of the alternate layers of sedimentary rocks, which belong to Kaligetas Formation, and siltstone, which belongs to Kerek Formation. Since the surface of unconformity is undulated very much, sedimentary rock unit, which belongs to Kaligetas Formation, is distributed above the elevation of around 165 m and around the very gentle slope between EL.140 to 155 m.

Talus deposit or sliding block is not found at the drilling cores of RI-1 and RI-2. Talus deposit was found at the trench of SS-3 excavated at the gentle slope around EL. 155 m. Since the extent of talus deposit is limited and the thickness of the layer is presumed to be thin, it is judged that a big scale of landslide, which affects the function of the reservoir, will not take place.

(4) FP Line

A gentle slope and isolated hills are developed between EL. 150 m and the terrace plain. The bedrock consists of alternate layers of sedimentary rocks, which belong to Kaligetas Formation, and siltstone, which belongs to Kerek Formation. The surface of unconformity is undulated very much and Kaligetas Formation is generally distributed at the surface of the slope. Talus deposit or sliding block is not distributed in the area between the gentle slope and the isolated hills.

Conclusion

(1) Saddle Portion on Right Bank

At the saddle portion on the right bank of the reservoir, pervious rock with lugeon value of more than 20 is distributed with 150 m in length and 10 m deeper than the Surcharge Water Surface (EL. 151.80 m). The elevation of the groundwater is lower than the Surcharge Water Surface at this area. The necessity of the leakage protection works to mitigate leakage after filling water in the reservoir will be studied in CHAPTER 7.

(2) Fault

From the results of the limited number of core drillings and trench excavations, the fault developed at the boundary of Damar and Kerek formations was judged small scale with the width of less than 10 cm without permeable layers. During the construction stage, the fault shall be exposed by removing topsoil. Based on its scale and permeability confirmed, the possibility of seepage from the upstream of the dam to the downstream through the fault shall be studied again.

(3) Slopes around Reservoir

Gentle slopes and isolated hills are scattered on the right bank of the reservoir. The possibility of landslide is considered at this area by the topographical features. However, since sliding block or slip surface are not recognized at the slopes where the geological survey was conducted, it is judged that the present topography is old dissected terrace or was formulated by differential erosion depending on the difference of geology. Therefore, there is no slope which may cause landslide by filling water in the reservoir and no protection works against landslide is required.

5.3 Construction Material

5.3.1 Construction Material Survey

The following materials are required for the embankment of rockfill dam and the concrete for appurtenant facilities.

- Aggregate
- Impervious Material
- Semi-pervious Material
- Pervious Material

To collect such necessary information and data on aggregate for concrete and embankment material as quality and available quantity, drilling works, seismic prospecting works, test pit excavation and soil and rock mechanical laboratory tests were executed. The work items and quantities of construction material survey in the study period of Phase1 and Phase 2 are shown in the following table.

Aggregate

Study Period	Work Item	Work Quantities
Phase 1	Core Drilling (D=66 mm)	13 holes, 710 m in total
	Seismic Prospecting	5 lines, 1,700 m in total
	Laboratory Test of Core Sample	20 samples
	Alkali-Aggregate Reaction Test	5 samples

Non-mixed Impervious Material

Study Period	Work Item	Work Quantities
Phase 2	Core Drilling (D=66 mm)	20 holes, 200 m in total
	Test Pit	12 pits
	Physical Property Test including tests of specific gravity, water content, liquid limit, plastic limit and particle-size analysis	24 samples
	Moisture-Density Relation Test (D=100 mm)	24 samples
	Consolidated Undrained Triaxial Compression Test (D=100 mm)	24 samples
	Permeability Test (D=100 mm)	24 samples

Mixed Impervious Material

Study Period	Work Item	Work Quantities
Phase 2	Physical Property Test including tests of specific gravity, absorption, water content, liquid limit, plastic limit and particle-size analysis	5 samples
	Moisture-Density Relation Test (D=100 mm)	5 samples
	Consolidated Undrained Triaxial Compression Test (D=100 mm)	5 samples
	Permeability Test (D=100 mm)	4 samples

Non-mixed Semi-pervious Material

Study Period	Work Item	Work Quantities
Phase 2	Core Drilling (D=66 mm)	10 holes, 100 m in total
	Test Pit	8 pits
	Physical Property Test including tests of specific gravity, absorption, water content and particle-size analysis	8 samples

Mixed Semi-pervious Material

Study Period	Work Item	Work Quantities
Phase 2	Relative Density Test (D=100 mm)	4 samples
	Consolidated Drained Triaxial Compression Test (D=100 mm)	4 samples
	Permeability Test (D=100 mm)	4 samples

Pervious Material

Study Period	Work Item	Work Quantities
Phase 2	Sampling including sieving works	1 sample
	Physical Property Test including tests of specific gravity, absorption and particle-size analysis	7 samples
	Large Scale Relative Density Test (D=300 mm)	1 samples
	Large Scale Consolidated Drained Triaxial Compression Test (D=300 mm)	4 samples
	Large Scale Permeability Test (D=300 mm)	3 samples

The details of investigation items and quantities are shown in Tables 5.3.1 and 5.3.2.

5.3.2 Concrete Aggregate

The geological survey of quarry site for aggregate was carried out in the Phase 1 stage.

Selection of Quarry Site

Sedimentary rock which is distributed within the area with the radius of 5 km centered at the damsite is formulated in the period between Miocene in Tertiary and Pleistocene in Quaternary and mainly consists of conglomerate, sandstone and mudstone. In the sedimentary rock, pyroclastic rock which consists of tuff breccia and tuff exist partly in stratum. The sedimentary rock in this area is too soft to be utilized as pervious material.

As shown in Fig. 5.3.1, volcanic rocks which formulate Mt. Ungarang located 17 km south of the damsite and intrusive rocks which scattered around Mt. Ungarang exist around the damsite. However the volcanic rocks mainly consist of low cemented rocks such as welded tuff or tuff breccia. Lava which is usually used as aggregate or pervious material is not distributed in wide range. Only the intrusive rocks can be used as aggregate and pervious material in this region.

Mt. Mergi located 17 km southeast of the damsite which was considered suitable as a quarry of aggregate for the dam concrete in the Phase 1 stage is formulated with intrusive sheet of andesite. Therefore, Mt. Mergi was selected as the quarry of aggregate and pervious material.

Topography and Stratigraphy

Topography and stratigraphy of Mt. Mergi and its around area which is selected as a quarry site for the concrete aggregate for the appurtenant facilities and the pervious material of embankment of rockfill dam are described hereunder.

(1) Topography

Mt. Mergi, the quarry site, is located on a plateau of about EL 400 m which extend in the east area of Mt. Ungaran. Mt. Mergi of which elevation is EL. 570 m has the height of around 170 m from its foot. Mt. Mergi extends about 1.5 km in the east-west direction and 0.8 km in the north-south direction.

(2) Stratigraphy

The location and geological profile at the quarry site is shown in Fig. 5.3.2, and the geological constitution at the quarry site is tabulated below.

<table>
<tr><th colspan="2">Age</th><th colspan="2">Formation and Stratum Name</th><th>Symbol</th><th>Description</th></tr>
<tr><td rowspan="2">Tertiary ~ Quaternary</td><td rowspan="2">Pliocene ~ Pleistocene</td><td colspan="2">Andesite</td><td>An</td><td>Andesite consists of sheet or dike, which is mainly composed of Plagioclase, Pyroxene and Ore Minerals, and shows dark gray. But it was changed in quality partly by the hydrothermal alteration, and secondary minerals that consist of Chlorite, Mordinite and Illite were formed, and show greenish light gray. The hardness of rock is comparatively high, and the bedrock has cracks with the interval of 10 to 200 cm.</td></tr>
<tr><td>Kaligetas</td><td>Pyroclastic Rock</td><td>Kp</td><td>Pyroclastic rock mainly consists of volcanic breccia, and partly contains mafic tuff and andesite lava. Volcanic breccia contains fragments of andesite and pumice, and matrix consists of mafic tuff. Rocks are weathered strongly, so the hardness of rocks is very soft. This stratum is covered by andesite sheet.</td></tr>
</table>

The plateau around Mt. Mergi consists of sedimentary rock and pyroclastic rock which was formulated between Pliocene in Tertiary and Pleistocene in Quaternary and volcanic rock which was formulated by the volcanic activities of Mt. Ungaran in Quaternary. While andesite of Mt. Mergi consists of intrusive rock. The difference of topography mentioned above is caused by differential erosion by the difference of geological constitution.

The form of the intrusive rock of andesite at Mt. Mergi is not clarified yet. At the southeast end of Mt. Mergi where core drilling works were conducted, intrusive rock is distributed. However, it is clarified by the drilling works that there exist pyroclastic rock below EL. 470 m to 480 m and the intrusive rock covers the pyroclastic rock. Therefore, intrusive rock is judged to form sheet.

The andesite include phenocryst of Plagioclase and Pyroxene with the diameter less than 3 mm, and partially secondary mineral, such as Chlorite, is formed by hydrothermal alteration. Therefore, soft and porous portions by hydrothermal alteration exist in the boring cores.

Conditions of Rock for Aggregate

(1) Rock classification

Cracks with same directions, which may possibly cause flatness of aggregate, do not developed in the andesite at the quarry site. Since the interval of cracks of the andesite is 10 to 200 cm, the maximum aggregate size of 15 cm is supposed to be available. The andesite of the quarry site is classified into four (4) classes, D class, CL class, CM class and CH class based on the hardness of the rock and the characteristics of four (4) classes are summarized in the table below.

Rock Classification	Characteristics
D	･Boring cores could be drilled without water like soil. ･Rocks are deeply weathered and very soft. ･Discrimination of cracks is impossible.
CL	･Boring cores mainly consists of shorter length of cores than its diameter or separated conditions. ･Cores are deeply weathered, discolored to brown or reddish brown and soft. ･Brown colored weathering is developed at the cracks and mostly include weathered material. ･Column cores which was softened by hydrothermal alteration belong to this class.
CM	･Mainly consisting of column core with the length of 10 to 50 cm. ･Almost of cores are fresh and hard. ･Cracks show brown colored weathering or fresh.
CH	･Mainly consisting of column core with the length of 10 to 50 cm. ･Rocks are composed of finer grain mineral and it is harder than CM class. ･Cracks show brown colored weathering or fresh.

(2) Laboratory Test Results

The results of rock laboratory tests are shown in Table 5.3.3 and Fig. 5.3.3 and 5.3.4.

The summary of the results of physical property tests and unconfined compressive tests are tabulated below.

Class	Physical Properties		Mechanical Property
	Averages Saturated Surface-dry Density (g/cm^3)	Averages Absorption Ratio (%)	Averages Unconfined Compressive Strength (kgf/cm^2)
CL	2.26	9.6	203
CM	2.62	2.9	566
CH	2.70	1.8	743

(a) Physical Property and Unconfined Compressive Test

(i) CL Class

The saturated surface-dry density and absorption ratio of all CL class samples are less than 2.5 gf/cm^3 and more than 3 % respectively. Further more unconfined compressive strength shows only 200 kgf/cm^2 on an average. From the result, CL class is judged to be unsuitable for concrete aggregate.

(ii) CM class

The saturated surface-dry density and absorption ratio of almost of all CM class samples show the values of more than 2.5 gf/cm^3 and less than 3 % respectively except for few samples. The unconfined compressive strength shows 560 kgf/cm^2 on an average. Therefore, CM class can be utilized as concrete aggregate.

(iii) CH class

The saturated surface-dry density and absorption ratio of all CH class samples are more than 2.5 gf/cm^3 and less than 3 % respectively. The unconfined compressive strength shows 740 kgf/cm^2 on an average. CH class has best property as concrete aggregate.

(b) Alkali-Aggregate Reaction Test

The test samples for alkali-aggregate reaction test were taken from two quarry sites at MT. Mergi which are under operation. Three samples were collected, namely two of them, sample Nos. A1 and A2, were from a quarry located east end of Mt. Mergi and the rest, sample No. B was from a stock yard of another quarry located at the west end of Mt. Mergi.

Alkali-aggregate reaction test was done by chemical method and the test results and the standard for judging harmfulness of samples are shown in Fig. 5.3.5. from this figure, all samples are judged not to be harmful.

Conclusion

From the results of the laboratory tests, it become clear that CM and CH class of andesite can be utilized as aggregate of concrete. However, it is necessary to confirm durability of CM class rock as concrete aggregate by means of abrasion test and soundness test.

At the east side of Mt. Mergi where the drilling works were conducted, pyroclastic rock which is classified D class is confirmed to exist at the depth deeper than EL. 460 m. Andesite, which is classified CL class and is judged not suitable for concrete aggregate, exists at the bottom of andesite sheet with the thickness of 10 to 20 m.

Therefore, available andesite is judged to exist above EL. 480 m. However, the available volume of andesite much exceed the required volume for aggregate.

Weathered and deteriorated layer of andesite is as thin as less than 10 m from the ground surface. Available andesite exists at the depth not deeper than 10 m from the ground surface. However, deteriorated andesite by hydrothermal alteration is partly distributed. The distribution of deteriorated zones is not confirmed in detail yet except a zone which was formed on the line S-2 of seismic prospecting and confirmed as CL class rock by boring core.

5.3.3 Embankment Material

The survey for embankment material of rockfill type dam was conducted at the beginning of Phase 2 study stage.

Selection of Survey Area

To mitigate the hauling cost of the impervious and semi-pervious materials, the objective survey areas for the impervious and semi-pervious material to be surveyed are limited to the areas near the damsite. Since the objective survey area for the suitable pervious material dose not exist near the damsite, a quarry site, which was selected for aggregate material of concrete for the concrete gravity type dam in the phase 1 stage, was selected as the area to be surveyed.

(1) Impervious Material

A wide flat plain which is called erosional low-relief surface is developed on the left bank of the damsite and it was judged that completely weathered foundation rock which can be utilized as impervious material is formulated. Therefore, the left bank ridge located at the immediate downstream of the damsite (Borrow Area D), the hilly area on the left bank of the damsite (Borrow Area B) and the hilly area at the south of the reservoir (Borrow Area A) were selected as survey sites for the impervious material. (refer to Fig. 5.3.6)

(2) Semi-pervious Material

Sandstone in low solid state that may be used as semi-pervious material exists partly in stratum in the sedimentary rock that formulates the hilly area near the damsite. However, the available quantity is considered to be limited and selection of the material will be costly. Therefore, only the riverbed deposits of Garang River or Kreo River are the possible materials though quality and available quantity is limited because both rivers are medium and small-scale rivers.

However, rather wide alluvium plain is formulated in the area about 0.5 km to 2.0 km upstream from the damsite on Kreo River and semi-pervious material is expected to be distributed in this area at certain available amount. This area is selected as the objective survey area for semi-pervious material (Borrow Area C). (refer to Fig 5.3.6)

Survey and Laboratory Test Results

(1) Non-mixed Material

The field survey and laboratory tests for the three embankment materials, impervious, semi-pervious and pervious material, were conducted without mixing each material. The results of survey and the tests are described hereinafter.

(a) Impervious Material

(i) Results of Drilling Works and Test Pits Excavation

Location map and geological profile at Borrow Area A and B are shown in

Figs. 5.3.7 and 5.3.8.

The soil condition of Borrow Areas A, B and D are shown in the table below, and it was observed that significant difference in terms of soil conditions does not exist among the three areas.

Division		Symbol	Characteristics
Weathered Soil Zone	Topsoil	St	Topsoil mainly consists of clay and silt, and there is no original structure of mother-rock. The soil has loose condition, and contains roots of plants and organic material.
	Reddish Soil	Sr	Reddish soil mainly consists of clay and silt, and there is no original structure of mother-rock, no fragment which composes rock. Therefore it consists of clay and silt mainly. The soil has high plasticity, but moisture content is not so high.
	Brownish Soil	Sb	Brownish soil mainly consists of clay and silt, and there is no original structure of mother-rock. But the fragments are recognized partly or generally as the mass of clay mineral and the quality of fragments is very soft. The soil has high plasticity like Reddish Soil, but moisture content is slightly high.
High Weathered Rock Zone		Rw	About half of the rock material has been weathered to clay minerals, and is converted to soil partly. But it is possible to classify the original rock

In all three areas, geological strata consist of weathered soil zone as upper layer and highly weathered rock zone underneath. The weathered soil zone consists of topsoil, reddish soil and brownish soil. Topsoil layer is loose with the thickness of tens of cm including organic material. While reddish and brownish soils consist of silt and clay with high plasticity and almost of weathered soil zone is occupied by reddish and brownish soil. The thickness of each layer is shown in the table below and reddish and brownish soils are considered to be able to use as impervious material.

Borrow Area	Reddish Soil		Brownish Soil	
	Thickness (m)	Depth from Ground Surface (m)	Thickness (m)	Depth from Ground Surface (m)
A	2.0~3.5	0.1~3.9	1.6~5.4	2.5~9.0
B	2.0~6.8	0.2~7.1	1.8~5.3	2.3~9.6
D	2.8~3.0	0.2~3.2	2.8 over	3.2 over

(ii) Results of Laboratory Tests

The laboratory tests have been conducted for reddish and brownish soils in

Borrow Areas A, B and D, and the test results are shown in Table 5.3.4 and Figs. 5.3.9 to 5.3.12.

All samples include more than 90 % of clay and silt and they are classified as CH and MH with fine grained soil and high plasticity except two (2) samples in Borrow Area A. The moisture content of brownish soil is several % higher than the one of reddish soil, however there is no significant difference as impervious material.

In general, earth material including 20 % of clay and silt is used as impervious material, since such material has necessary permeability with less consolidation settlement expected.

The impervious material which can be obtained from all three borrow areas contains more than 90 % of clay and silt and it was judged that the material can not be used as impervious material without mixing with coarse material.

(2) Semi-pervious Material

(i) Results of Drilling Works and Test Pits Excavation

Geological strata of Borrow Area C where riverbed deposit and terrace deposit are distributed as semi-pervious material is presented in Table 5.3.5. Location map and geological profile at Borrow Area C is shown in Fig. 5.3.13.

Riverbed deposit is distributed along the existing riverbed and flood plain. It contains 5 % of siltstone gravel, which is soft and easily pulverized by slaking. Thickness of riverbed deposit was confirmed by drilling and test pit excavation at 1.0 m to 4.2 m with average thickness of 2.3 m.

Terrace deposit is divided into two layers, upper and lower layers. The upper and lower layers consist of silt and gravel respectively. However, there is an area on the terrace at the right bank of Kreo River where the lower layer does not exist and the upper layer covers the base rock directory. The thickness of the upper layer is confirmed at 0.8 m to 3.4 m and the one of the lower layer is 1.3 m to 2.8 m. The lower later of terrace deposit includes 5 to 10 % of siltstone gravel same as the riverbed deposit.

(ii) Results of Laboratory Tests

The laboratory tests for the riverbed and terrace material were carried out and the results of the tests are presented in Table 5.3.6 and Fig. 5.3.14.

The river deposit consists of gravel-sand mixtures with maximum grain size of 77 mm and the proportion of grain size consists of 71 % of gravel, 23 % of sand and 6 % of silt. The natural moisture content is about 10 %.

The lower layer of terrace deposit consists of gravel-sand mixtures to silty sand with maximum grain size of 145 mm and the proportion of grain size of 37 to 74 % of gravel, 11 to 42 % of sand and 15 to 21 % of silt and clay. The natural moisture content ranges between 20 and 26 %.

Since the riverbed deposit includes 6 % of silt and clay and 5 % of siltstone gravel, it is judged that the river bed material can not be used as semi-pervious material.

As for the terrace deposit, it includes 15 to 20 % of silt and clay and 5 to 10 % of siltstone gravel which may cause slaking. Therefore, it also dose not have permeability required as semi-pervious material.

As a conclusion of the laboratory tests, they can not be utilized for semi-pervious material without mixing with other material. Further more, since the total available amount for the semi-pervious material is limited, the above material can not be utilized for semi-pervious material. Therefore, as semi-pervious material, other sources shall be applied.

It can not be considered that the above materials are applied to the mixing material for the impervious material from the economical viewpoint, because massive amount of the upper terrace layer shall be removed and to reduce the natural moisture contents of 20 %.

(c) Pervious Material

(i) Results of Drilling Works and Laboratory Tests

The survey area for pervious material is decided at Mt. Mergi located 17 km southeast of the damsite. Andesite which is distributed at Mt. Mergi is considered to be 'sheet' and available andesite is confirmed to exist above

EL 470 m at the east slope of Mt. Mergi from the results of drilling works executed in the Phase 1 study stage.

Andesite at Mt. Mergi, which has cracks with the interval of 10 to 200 cm, is considered to be able to use as pervious material. Since weathered or deteriorated zone is supposed to be thin, the available andesite is considered to exist not deeper than 10 m from the ground level.

The andesite is classified into four classes, D, CL, CM and CH. The class D shall not be used and other three classes are available. The physical property of the available andesite is summarized in the table below.

Rock Classification	Average Saturated Surface-dry Density	Average Absorption Ratio (%)
CH	2.70	1.8
CM	2.62	2.9
CL	2.26	9.6

From figures in the table above and their hardness, it is concluded that CH and CM classes can be utilized as pervious material for any zone and CL class can be utilized in an inner zone.

(ii) Results of Laboratory Tests

The laboratory tests for pervious material were executed for two (2) kinds of samples with adjusted gradation. The results are presented in Table 5.3.7 and Figs. 5.3.15 and 5.3.16, and are summarized in the table bellow.

Sample	Test Conditions		Shear Strength		Permeability k (cm/sec)
	Adjustment of Gradation	Void Ratio	C (kgf/cm^2)	ϕ (°)	
CASE-1	63.0 - 53.0 mm 5% 53.0 - 37.5 mm 10% 37.5 - 26.5 mm 10% 26.5 - 19.0 mm 7%	0.325	1.94	40.92	---
CASE-2	19.0 - 9.50 mm 18% 9.50 - 4.75 mm 18% 4.75 - 2.00 mm 22% 2.00 mm - 10%	0.283	1.97	41.38	7.13×10^{-2}
CASE-3	63.0 - 53.0 mm 5% 53.0 - 37.5 mm 10% 37.5 - 26.5 mm 10% 26.5 - 19.0 mm 7%	0.257	1.29	43.87	5.41×10^{-2}
CASE-4	19.0 - 9.50 mm 18% 9.50 - 4.75 mm 18% 4.75 - 2.00 mm 12% 2.00 mm - 20%	0.22	1.45	44.35	8.00×10^{-3}

(iii) Conclusion of Test Results

The permeability required for pervious material is more than 1×10^{-2} cm/sec in general. All test samples satisfy this requirement and are judged to be available as pervious material. The estimated physical and mechanical values of pervious material, which were derived from the laboratory test results, are shown in the table below.

Material (Test Sample Size)	Item	Symbol	Test Result	Estimated Value
Pervious (< 63.0 mm)	Bulk Specific Gravity	Gb	2.544	2.544
	Natural Moisture Content	w	0.80 %	1.00 % *
	Void Ratio	e	0.221 - 0.325	0.325 (maximum Value)
	Dry Density	γd	1.92 – 2.083 tf/m^3	1.92 tf/m^3 (minimum value)
	Shear Strength	c	1.29 - 1.97 kgf/cm^2	0.00 kgf/cm^2
		φ	40.92 ° - 44.35 °	more than 40 °
	Permeability	k	5.41×10^{-2} cm/sec - 8.00×10^{-3} cm/sec	not less than 1×10^{-2} cm/sec

* Note : The moisture content of samples at the time of test are presumed to be dryer than at the time mined. Therefore the estimated value of moisture content makes bigger than the test value.

(2) Mixed Material

Since it was concluded that the impervious and semi-pervious material, which is available near the damsite, can not be utilized without mixing material, laboratory tests on mixed material were carried out.

However, it has been become clear that the available amount of coarse material is limited near the damsite. Therefore, andesite crushed rock and crushed sand from the quarry at Mt. Mergi and river sand mined around Mt. Merapi which is sold in and around Semarang City are used as mixing material.

(a) Mixed Impervious Material

(i) Sample for Test

As a sample for impervious material, fine materials taken from test pits in Borrow Areas B and D which show typical moisture contents and grain size distribution were used.

(ii) Results of Laboratory Tests

The physical property test results of the coarse materials are presented in Table 5.3.6 and Fig 5.3.17. The test results of mixed material are shown in Table 5.3.8, and Figs. 5.3.18 and 5.3.19. The test results of mixed material are summarized in the table bellow.

<table>
<tr><th rowspan="2">Sample</th><th colspan="2">Test Conditions</th><th colspan="2">Shear Strength</th><th rowspan="2">Permeability
k (cm/sec)</th></tr>
<tr><th>Adjustment of
Gradation</th><th>Dry Density
(g/cm^3)</th><th>C
(kgf/cm^2)</th><th>ϕ (°)</th></tr>
<tr><td rowspan="2">CASE-1</td><td rowspan="2">Fine Material : 50%
(TPD-1U)
Coarse Material : 50%
(Crushed Rock : 20%)
19.0 - 4.75mm
(Natural Sand : 80%)</td><td>1.591
(D95:Wet Side)
w=24.3%</td><td>0.27</td><td>25.63</td><td>1.00×10^{-5}</td></tr>
<tr><td>1.673
(Dmax:OMC)
w=20.5%</td><td>-</td><td>-</td><td>2.29×10^{-7}</td></tr>
<tr><td rowspan="2">CASE-2</td><td rowspan="2">Fine Material : 35%
(TPD-1U)
Coarse Material : 65%
(Crushed Rock : 20%)
19.0 - 4.75mm
(Natural Sand : 80%)</td><td>1.703
(D95:Wet Side)
w=20.6%</td><td>0.12</td><td>31.37</td><td>1.92×10^{-5}</td></tr>
<tr><td>1.798
(Dmax:OMC)
w=16.5%</td><td>-</td><td>-</td><td>6.39×10^{-6}</td></tr>
<tr><td rowspan="2">CASE-3</td><td rowspan="2">Fine Material : 25%
(TPD-1U)
Coarse Material : 75%
(Crushed Rock : 20%)
19.0 - 4.75mm
(Natural Sand : 80%)</td><td>1.837
(D95:Wet Side)
w=17.0%</td><td>0.09</td><td>30.49</td><td>3.01×10^{-5}</td></tr>
<tr><td>1.934
(Dmax:OMC)
w=13.8%</td><td>-</td><td>-</td><td>1.04×10^{-5}</td></tr>
<tr><td rowspan="2">CASE-4</td><td rowspan="2">Fine Material : 27%
(TPD-1U)
Coarse Material : 73%
(Crushed Rock : 35%)
19.0 - 4.75mm
(Natural Sand : 65%)</td><td>1.771
(D95:Wet Side)
w=17.8%</td><td>0.39</td><td>20.39</td><td>2.47×10^{-5}</td></tr>
<tr><td>1.864
(Dmax:OMC)
w=14.8%</td><td>-</td><td>-</td><td>9.62×10^{-6}</td></tr>
<tr><td rowspan="2">CASE-5</td><td rowspan="2">Fine Material : 35%
(TPB-4U)
Coarse Material : 65%
(Crushed Rock : 20%)
19.0 - 4.75mm
(Natural Sand : 80%)</td><td>1.738
(D95:Wet Side)
w=18.9%</td><td>0.06</td><td>36.00</td><td>1.15×10^{-5}</td></tr>
<tr><td>1.830
(Dmax:OMC)
w=15.5%</td><td>-</td><td>-</td><td>3.56×10^{-6}</td></tr>
</table>

(iii) Conclusion of Test Results

Permeability required for the impervious material is less than 1×10^{-5} cm/sec in general. Among the five samples in the table above, CASE-3 shows permeability of more than 1×10^{-5} cm/sec under both optimum moisture content and D95 density (wet side), it is judged that CASE-3 sample can not applied as impervious material.

Some of other samples show the permeability of more than 1×10^{-5} cm/sec under D95 density. However, it will be possible to compact more by effective compaction equipment and effective supervision of the embankment works and required permeability is expected to be obtained at the construction site.

CASE-1 sample has possibility that fine material exceed 25 % of total weight and the optimum moisture content is 20.5 % which is bigger by 5 % than the one of other samples. In general, workability of impervious material with fine material of more than 25 % will be deteriorated and may cause subsidence. Therefore, CASE-1 sample is judged not suitable as an impervious material.

The samples of CASE-2, 4 and 5 are available as mixed impervious material. Estimated physical and mechanical properties are summarized in the table below and the range of full sized gradation of the actual mixed impervious material is shown in Fig. 5.3.20.

Material (Test Sample Size)	Item	Symbol	Test Results	Estimated Value
Impervious (<19.0 mm)	Apparent Specific Gravity	Gs	2.716 - 2.737	2.716 (minimum Value)
	Optimum Water Content	Wopt	14.8 - 16.5 %	15.50%
	D Value 95 Dry Density	γd	1.708 - 1.772 tf/m³	1.739 t/m³ (average)
	Shear Strength	C'	0.06 - 0.39 kgf/cm²	0.1 kgf/cm²
		φ'	20.4 - 36.0 °	25 °
	Permeability	k	9.62×10^{-6} - 3.56×10^{-6} cm/sec (Optimum Moisture Content : Max Dry Density)	less than 1×10^{-5} cm/sec

The values in table above are decided based on the lower limit or average value of the test results of the available three (3) samples. The test material of three (3) samples are adjusted to exclude the grain size bigger than 19.0 mm. Therefore, the actual range of gradation is set to include 20 to 50 % of coarse material with the size of more than 19.0 mm.

(b) Mixed Semi-pervious Material

(i) Sample for Test

Crushed rock of the quarry site at Mt. Mergi and purchased sand mined around Mt. Merapi are also used for mixing material for the laboratory test of mixed semi-pervious material.

(ii) Results of Tests

Test results of mixed semi-pervious material are presented in Table 5.3.9 and Figs. 5.3.21 and 5.3.22. The test results are summarized as bellow.

Sample	Test Conditions		Shear Strength		Permeability k (cm/sec)
	Adjustment of Gradation	Void Ratio	C (kgf/cm^2)	ϕ (°)	
CASE-1	Crushed Rock 19.0 - 4.75mm : 30% 4.75mm under : 70%	0.39	0.02	35.68	4.09×10^{-3}
CASE-2	Crushed Rock 19.0 - 4.75mm : 60% 4.75mm under : 40%	0.41	0.03	35.46	1.14×10^{-2}
CASE-3	Crushed Rock 19.0 - 4.75mm : 30% Natural Sand : 70%	0.23	0.02	34.24	1.36×10^{-4}
CASE-4	Crushed Rock 19.0 - 4.75mm : 60% Natural Sand : 40%	0.28	0.07	36.60	2.72×10^{-3}

(iii) Conclusion of Test Results

The permeability required for semi-pervious material is 1×10^{-2} to 1×10^{-4} cm/sec in general. All of the test samples presented in the table above almost satisfy the requirement and they are judged to be utilized as semi-pervious material. The estimated physical and mechanical properties are summarized in the table below and the range of full sized gradation of the semi-pervious material is shown in Fig. 5.3.20.

Material (Test Sample Size)	Item	Symbol	Test Results	Estimated Value
Semi-Pervious (<19.0 mm)	Bulk Specific Gravity	Gb	2.573 – 2.600	2.588 (average)
	Natural Water Content	W	0.39 - 0.75 %	2.00 %*
	Void Ratio	e	0.41 – 0.23	0.39
	Dry Density	γd	1.831 - 2.107 t/m^3	1.86 t/m^3
	Shear Strength	c	0.02 – 0.007 kgf/cm^2	0.00 kgf/cm^2
		ϕ	34.24 ° – 36.60 °	35 °
	Permeability	k	1.14×10^{-2} - 1.36×10^{-4} cm/sec	1×10^{-2} - 1×10^{-4} cm/sec

* Note : The moisture content of samples at the time of test are presumed to be dryer than at the time mined. Therefore the estimated value of moisture content makes bigger than the test value.

These values presented in the table above are decided based on the lower limit or average value of the test results of above samples. Since the test material of the samples are adjusted to exclude the grain size of bigger than 19.0 mm, the actual range of gradation is set to include 40 to 70 % of coarse material. In Case 3 in which natural sand is mixed, the permeability is 1.36×10^{-4} cm/sec, which is almost lowest limit for semi-pervious material. The sample was adjusted without the grain size bigger than 19.0 mm. Since Case-3 include fine grade of 3.6 %, the fine grade of 2 % will be the upper limit for the actual material.

Consideration

Prior to the embankment works of the dam body, the standard for the construction supervision shall be established upon the confirmation of mixing ratio or gradation of embankment material by trial embankment works. The following items shall be taken into consideration in establishment of the standard.

(1) Permeability of Mixed Impervious Material

Some of the samples of imperious material for the laboratory tests do not satisfy the permeability requirement of less than 1×10^{-5} cm/sec at the wet side of optimum moisture content. Therefore, the gradation or the allowable moisture content shall be decided through in-situ permeability tests by trial embankment.

(2) Adjustment of Moisture Content and Mixing Method of Mixed Impervious Material

The natural moisture content of weathered soil which is available near the damsite is as high as 40 % and when the weathered soil is mixed with coarse material, weathered soil may become hard pan. To prevent weathered soil from hard pan, suitable mixing method shall be selected through trial embankment.

Crushed rock or sand which are supposed to be used as coarse material for mixing have moisture content of only several %. It is confirmed that the mixed sample with the crushed rock and sand tend to have lower moisture content than the optimum moisture content. Therefore, adjustment method of moisture content of the mixed material using crushed rock and sand shall be studied through trial embankment.

(3) Sand Ratio in Semi-pervious Material

The natural sand which was used for mixing material in the laboratory tests are debris flow deposit mined around Mt. Merapi and the quality of the natural sand will fluctuate depending on the mining places. Therefore, when the natural sand is used as a mixing material, quality control of the sand is quite important.