CHAPTER 5 EXAMINATION OF BASIN DESIGN DISCHARGE VOLUME

5.1 Setup of Target Year

The target year for the comprehensive flood management is set up as for 2030 in consideration with the consistence with the spatial plans of related local governments.

Table 5.1-1 Spatial Plans of Related Local Governments

Local Government	West Java	DKI Jakarta	Depok City	Bogor Regency	Bogor City
Target Year	2010 - 2029	2011 - 2030	2011 - 2030	2006 – 2025 [2)]	2012 - 2031

Source: JICA Project Team

5.2 Setup of Future Prospective of River Basin

The assumption on the future prospective of the basin is consistent with the spatial plans of related local governments (the detail explanation refers to the report on spatial plan).

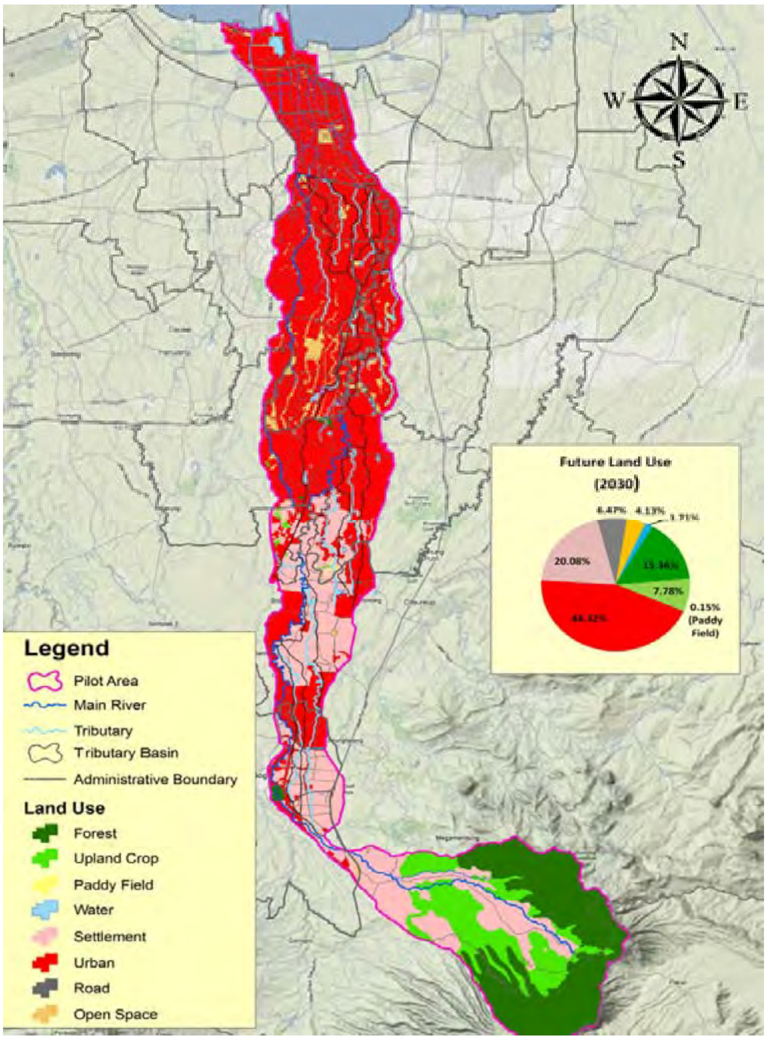


Figure 5.2-1 Future Land Use Condition of River Basin (2030)

5.3 Examination of Basin Design Discharge Volume

The flow chart for the examination of basin design discharge volume is shown in Figure 5.3-1.

The design scale is determined as 1/50. The design rainfall is described by the 3 elements consisting of the rainfall volume, time distribution of rainfall and area distribution of rainfall. The design rainfall is defined based on the extension of the mass of recorded rainfall (rainfall pattern).

Furthermore, regarding the basin high water, the hydrograph is prepared based on the defined design rainfall by flood inundation analysis model.

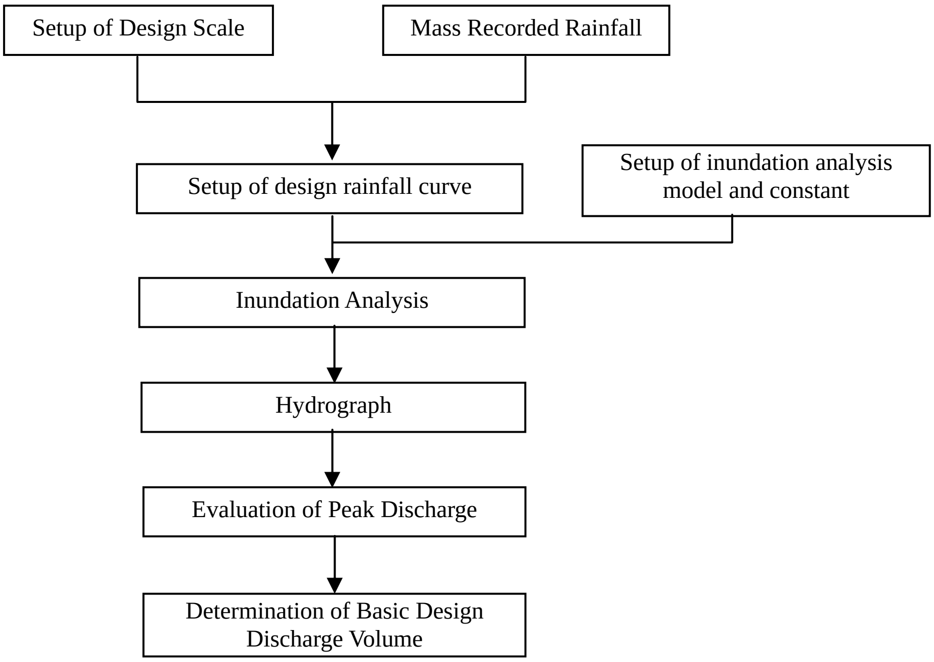


Figure 5.3-1 Flow Chart of Basin Design Discharge Volume Examination

5.3.1 Design Reference Point

The design reference point in the basin is the location to determine the design scale in the flood management area in the basin.

The design reference point is defined at 1 location in the basin where is the just upstream or the neighboring area of the most important urban area in the basin. It is decided by considering the distribution of population and asset in the inundation area, topographic feature, inundation pattern, etc. Moreover, it is selected in accordance with the data collectability on water level, discharge volume and so on.

Due to the following reasons, the Manggarai Point (Manggarai Water Level Gauging Station) is selected as the design reference point in Ciliwung river basin (see the following figure).

- It is located in the upstream of urban area where intensive population and assets are gathered in the inundation area.
- It is located in the upstream of capital city as economic and industrial center.
- It is the reference point as for the flood warning river.
- The sufficient hydrological data is available.
- It is located at the downstream of the river originated from the mountainous area.

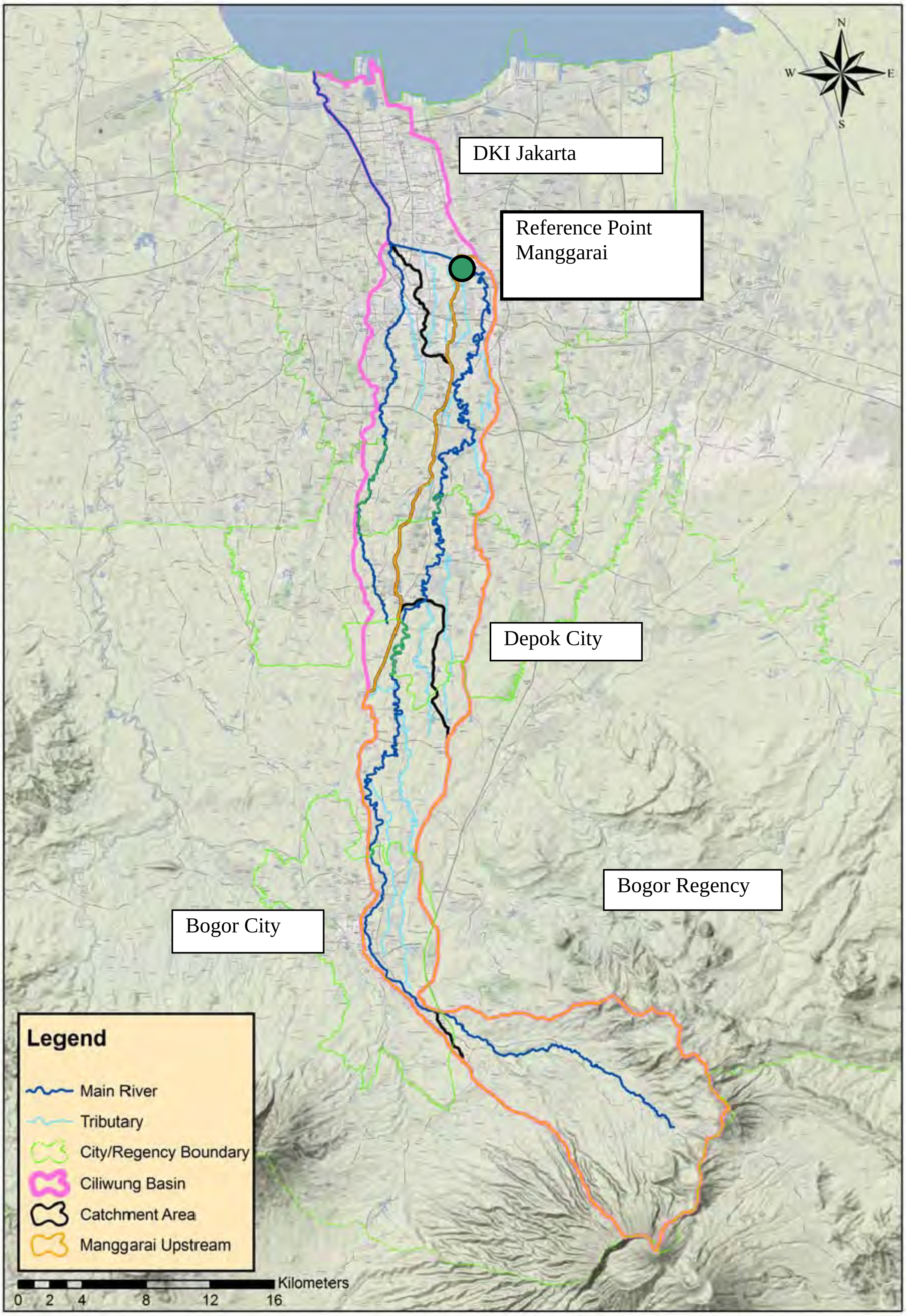


Figure 5.3-2 Location Map of Reference Point

5.3.2 Design Rainfall

(1) Design Scale

The design scale of comprehensive flood management is described as the safety level of flood control (construction level of flood management facility). It is decided by comprehensively considering the urbanization pattern in the basin, flood damage conditions, economic effects, and so on.

The design scale for comprehensive flood management in Ciliwung river is decided as W=1/50 by considering the followings.

- The safety level of flood control in Ciliwung river set as W=1/50.
- The probability of actual rainfall in 2007 flood as the largest flood in the record is evaluated as approximately 1/60.

As for the reference, the design scale of 8 river systems in 1997 master plan is shown in Table 5.3-1. Moreover, the design scale in several countries is summarized in Table 5.3-2.

Table 5.3-1 Design Scale of 1997 Master Plan

No.	River systems	Catchment Area (km^2)	Characteristics of Basins		Design scale (year)
			Topography	Present Landuse in Flood plain	
1	Cidurian	803	Mountainous	Rural	1/25
2	Cimanceuri	570	Mountainous	Rural	1/25
3	Cirarab	161	Hilly	Rural	1/25
4	Cisadane	1411	Mountainous	Urban + Rural	1/50
5	Cengkareng Floodway	459	Plain+Hilly	Urban	1/100
6	Western Banjir Canal	421	Plain + Hilly + Mountainous	Urban	1/100
	Ciliwung (upstream of manggarai)	337	Mountainous	Urban	1/100
7	Eastern Banjir Canal <proposed>	207	Plain + Hilly	Urban	1/100
8	C.B.L. Floodway	1326	Plain + Hilly + Mountainous	Rural + Urban	1/50

Table 5.3-2 Design Scale in Several Countries

COUNTRY	COMMERCIAL	INDUSTRIAL	RESIDENTIAL	RURAL	AGRICULTUR
Australia	50-100	50-100	50-100		5-50
Bulgaria	100-500			30-100	5-10
China (2)	200			100	
Czech Republic	100	50			7-10
Hong Kong	50-200	50-200	50-200	10-200	2-5
India (2)	50				25
Indonesia	**5-100**	**5-100**	**5-100**	**5-50**	**5-25**
Japan	10-200	10-200	10-200	10-200	10-200
Malaysia (3)	5-100	5-100	5-100	5-100	5-30
Philippines(2)	100			50-70	
Poland	1,000	500		100	20-100
Turkey	100-500	100-500			
Thailand	25-100	25-100	25-100		50-200
UK	10-100	10-100	10-100		1-10
USA	25-100	25-100	25-100		5-25

NOTES: Source: Manual ESCAP

(1)Standards refer to river training and flood control

(2)These standards are for levee design

(3)Designs also check that 100-year flood line is below ground line of buildings

(2) Duration of Design Rainfall

The duration of design rainfall is the flood arrival time affecting the peak discharge volume. It is determined as for 48 hours by examining the peak difference of rainfall and discharge volume (details are described in Chapter 2).

(3) Design Rainfall

The design rainfall is calculated based on the available hourly rainfall data from 1992 to 2008 with N=17 years hourly rainfall data in the basin. For the calculation of design rainfall, the Gunbel Probability Distribution Model is applied, which is commonly used in Indonesia.

- Rainfall in Duration of Design Rainfall 48 Hours Rainfall: 247mm (N=17, Gumbel Probability Distribution Model)

(4) Design Rainfall Curve

The 4 rainfall patterns (rainfall curve in 1994, 2001, 2006 and 2007) are applied as design rainfall curve. Those patterns are calculated by extending the rainfall curve of past major flood to the design rainfall volume and eliminating anomalous rainfall data in timely distribution and area distribution of rainfall data (details refer to Chapter 2).

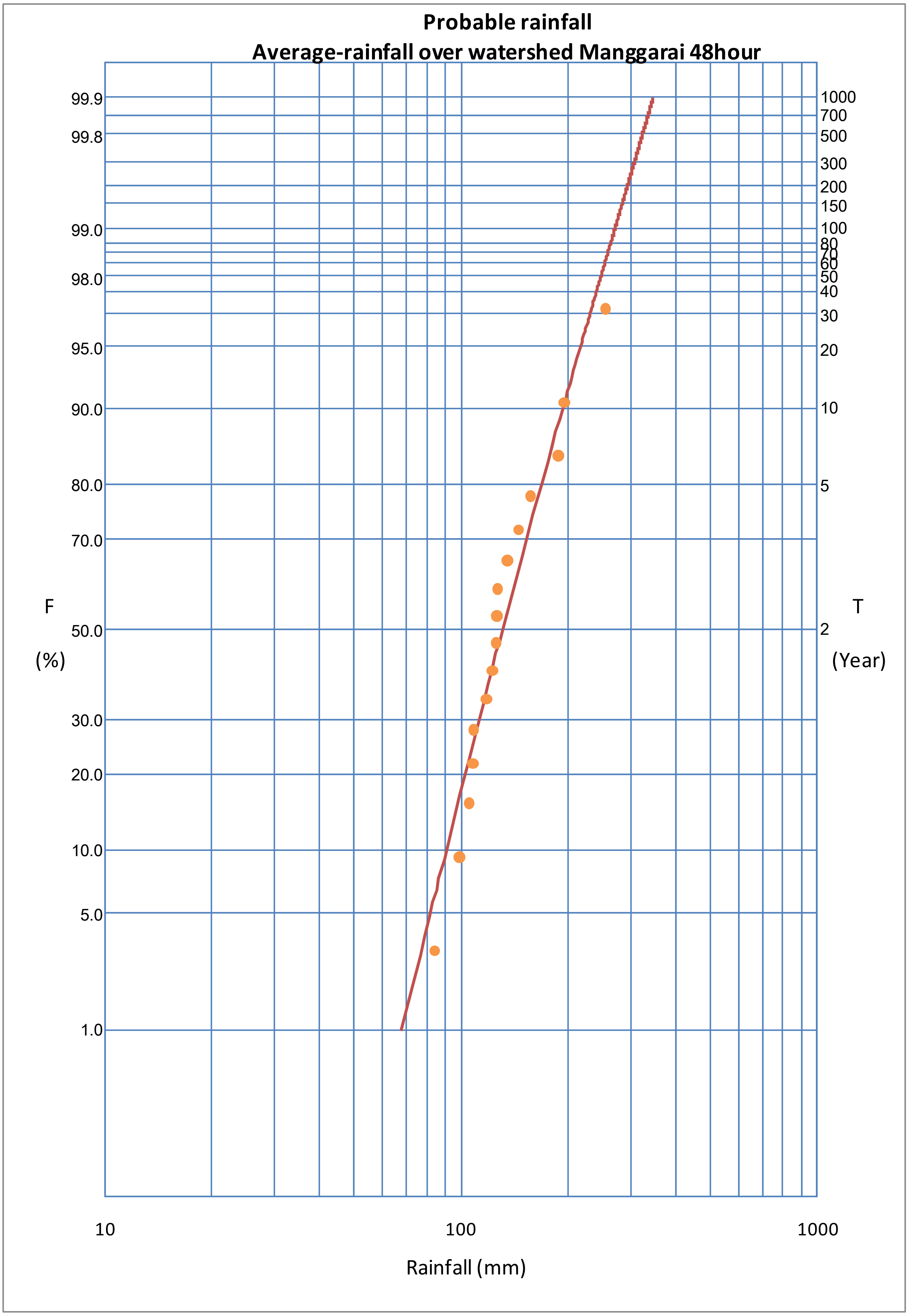


Figure 5.3-3 Result of Probability Calculation (Manggarai, 48 hours, N=9)

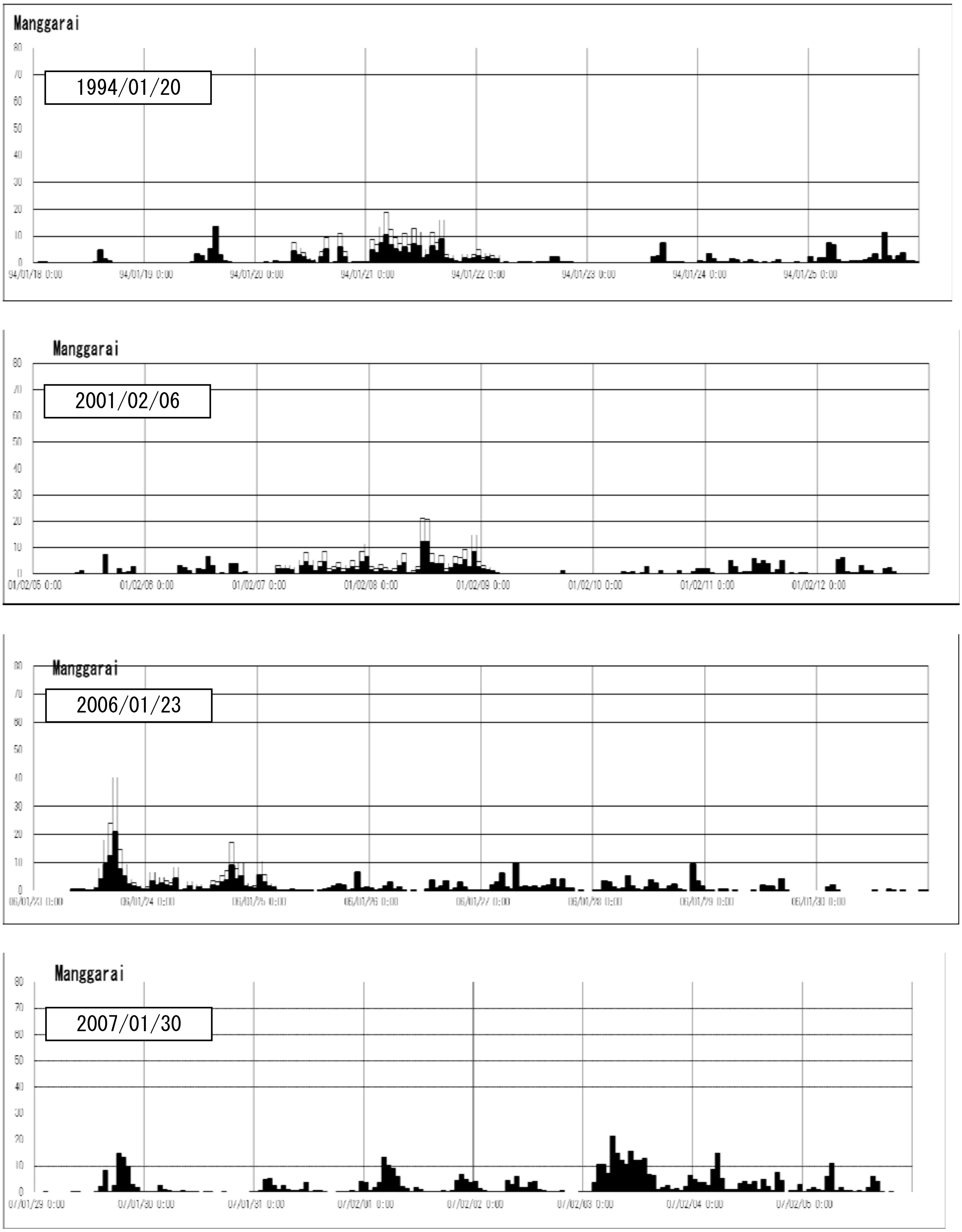


Figure 5.3-4 Design Rainfall Curve

5.3.3 Determination of Flood Inundation Analysis Model

(1) Selection and Establishment of Flood Inundation Analysis Model

1) Selection of Inundation Model Type

In this analysis, (a) mountain areas and hilly areas are regarded as discharge basin, (b) low-lying areas are regarded as inundation area, and adequate hydraulic model will be applied according to each type of flow. Brief overviews of hydraulic models for discharge basin and inundation area are shown as follows:

(a) Model for Discharge Basin

Kinematic Wave Method will be applied because it is able to present the flow of the slope regardless of water level in the downstream. Adopted form of the model is Distributed Runoff Model, which has the same mesh structure with the inundation area and is able to track flow of each mesh along with the land features and slopes, in order to provide the flow volume according to the minute meshes of inundation area.

(b) Model of Inundation Area

Dynamic Wave Method, which is able to present the change of flows affected by the land features and structures such as drains, will be applied and trace the inundation flows. And adopted form of the model is Two Dimensional Un-Steady Flow Model which is able to recreate the propagation phenomena of flooding flow in greatest detail.

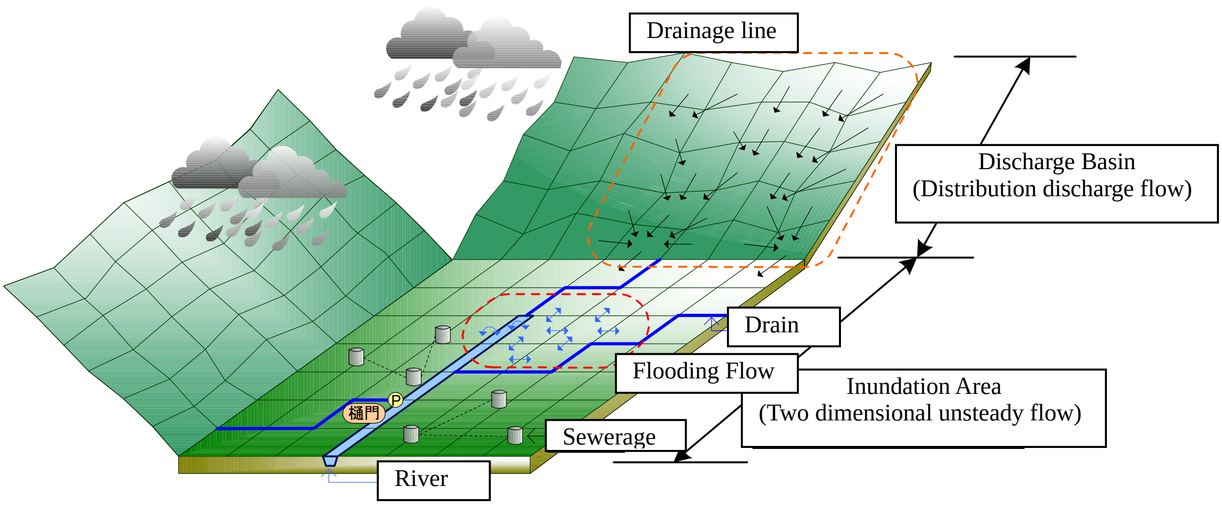


Figure 5.3-5 Image of Flood Inundation Analysis Model

2) Basic Structure of Flood Inundation Model

To settle the basis structure of flood inundation model, the features of target basin need to be reflected. The features of target basin are shown in the following.

- Target basin is divided into "discharge basin" and "inundation area".
- Target basin has been urbanized significantly, therefore surface drainage facilities such as rainwater drainage system are developed within the basin.
- High frequency of flooding

Required functions for structuring model are described as follows:

- Duplicate the combined flooding of inland flooding and external flooding.
- Analyze discharge and inundation in the basin as consistent phenomena.
- Duplicate time-series fluctuation considering the downstream water level, runoff volume from discharge basin and effect of bridges.
- As for dimensional expansion of flood steam and propagation velocity, duplicate the flow-down resistance, etc. considering the land use and density of houses.
- Be able to secure high accuracy with consideration for the effect of drainage, earth fill and subtle land features.
- Reflect the sluice way and discharge by pumping under the effect of inland and external water level.
- Settle the culvert for sewage rainwater discharge separately from surface flow, and describe the urban discharge system.
- Settle the retention facilities and reflect the initial flood adjustment functions.

(2) Flood Inundation Analysis Model

The figure of flood inundation analysis model is shown in Figure 5.3-6.

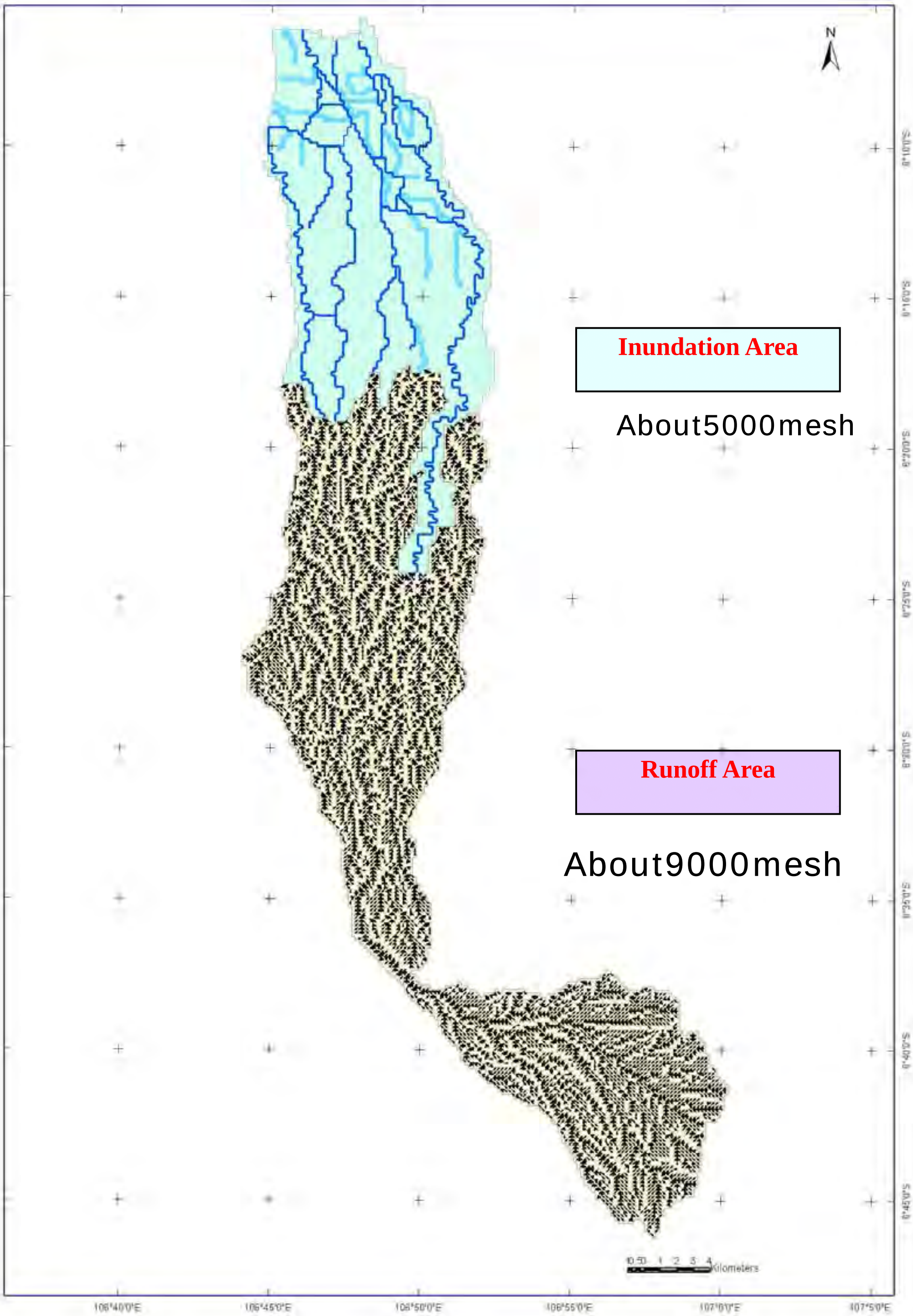


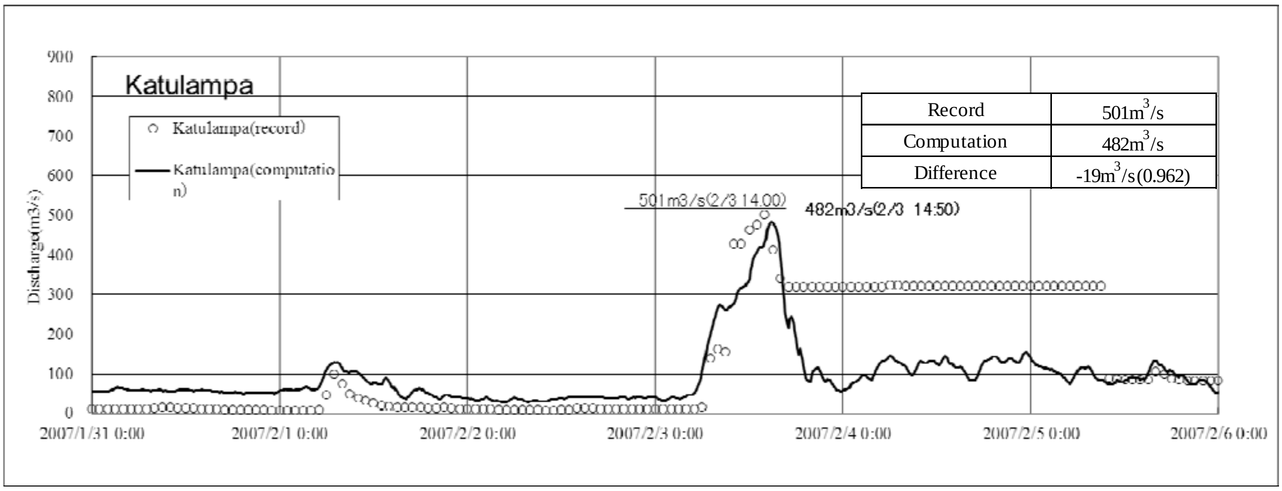
Figure 5.3-6 Figure of Flood Inundation Analysis Model

(3) Reproducibility of Flood Inundation Analysis Model

Reproduction of recent most severe flood in February 2007 will be implemented in the model. Model validity is verified by actual flow volume (HQ adjusted value) at Depok and Katulampa points for discharge basin model and by actual water level and actual inundation area at Manggarai for inundation area model.

1) River Flow Volume

Results of reproductive calculation for river flow volume at Depok and Katulampa are shown as follows.



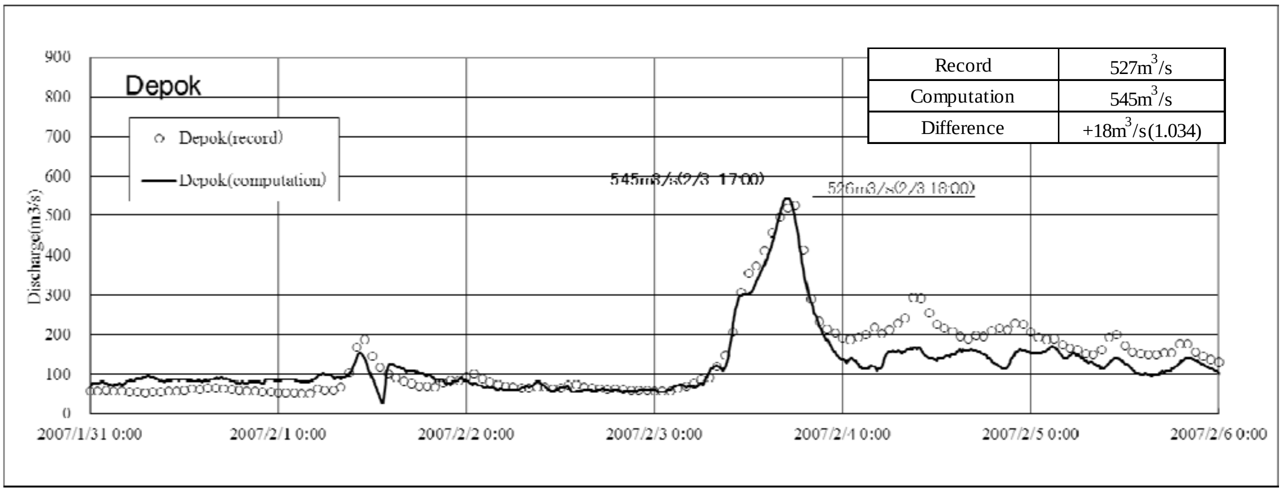


Figure 5.3-7 Discharge Hydro(Katulampa, Depok)

2) River Water Level

Results of reproductive calculation for river flow volume at Manggarai are shown as follows.

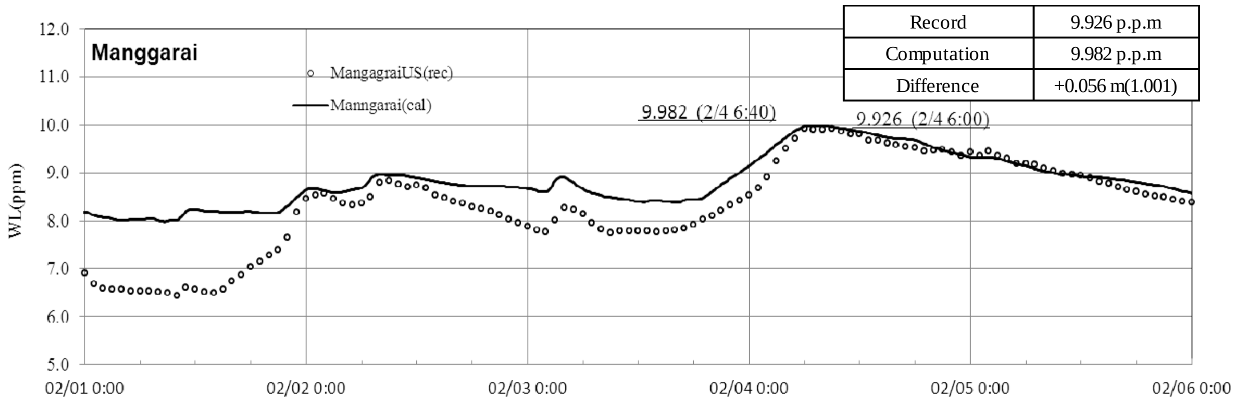


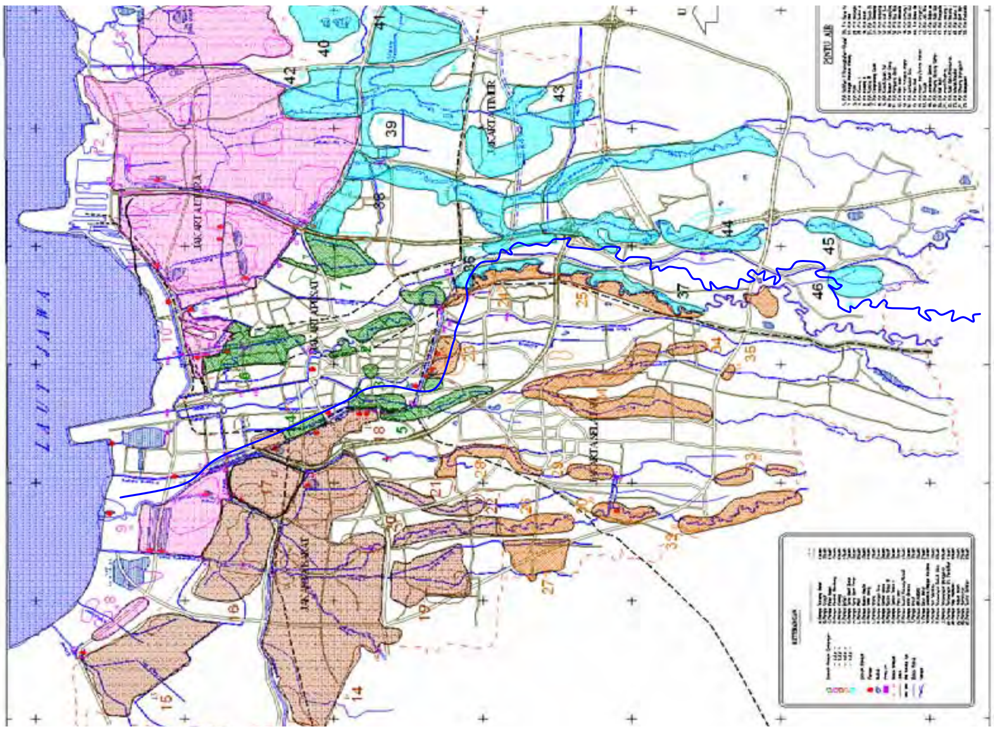
Figure 5.3-8 Water Level Hydro (Manggarai)

3) Inundation Area

Results of reproductive calculation for inundation area based on the results above are shown in the next page.

Record	Calibration

Record



Calibration

Record	87.3km^2
Computation	91.4km^2
Difference	4.1km^2 (1.047)

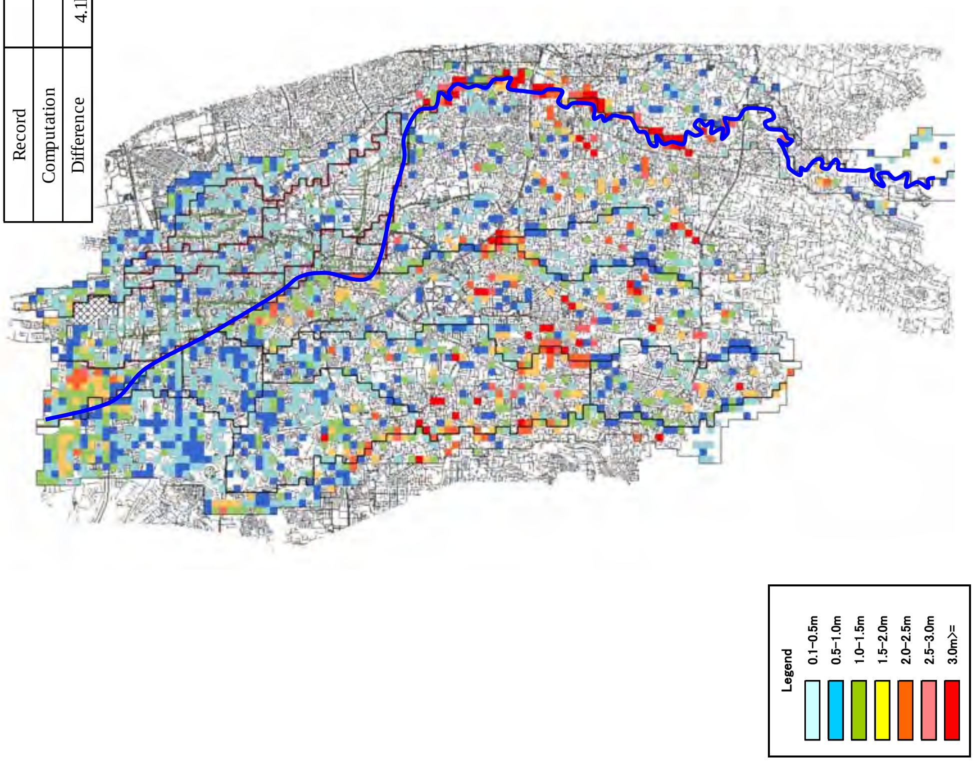


Figure 5.3-9 Maximum Inundation Depth

5.3.4 Determination of Basin Design Discharge Volume

The basin design discharge volume is estimated by converting the design rainfall curve (see in 5.3.2) to the discharge volume by flood inundation analysis model (see in 5.3.3), and determined by considering the past floods and the specifications of planned flood facilities.

Moreover, it is necessary to set up the peak discharge volume of basin design discharge volume estimated from design rainfall by comprehensively considering the peak discharge of past largest floods, past largest rainfalls and maximum possible rainfall.

Thus, the basin design discharge volume is evaluated based on design rainfall, the past largest flood in February 2007 and specific discharge in other rivers in JABODETABEK area. The work flow is shown in Figure 5.3-10.

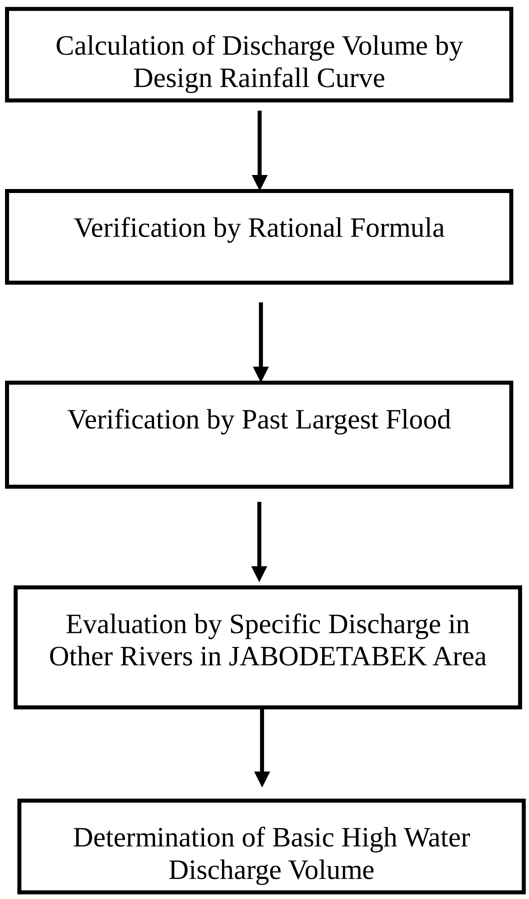


Figure 5.3-10 Determination Flow of Basin Design Discharge Volume

(1) Calculation of Discharge Volume by Design Rainfall Curve

1) Calculation Condition

The calculation conditions of peak water discharge of basin design discharge volume are shown in the following table.

Table 5.3-3 Calculation Condition

Item	Calculation Condition
Land Use (urbanization ratio)	70% (based on spatial plans (2030) of related local governments)
River Channel/ Drainage Facility	Rivers: Condition as of 2011(Without Overflow) Drainage Facility: Conditions as of 2011
Basin Constant/ Effective Rainfall	Basin Constant: Reproduced Value Effective Rainfall: Reproduced Value

Land Use classificatton	N	f1	Rsa	fsa	Application
Water area	0 (0)	1.0	0 (0)	1	Water surface
Paddy field	2 (2～3)	0	50 (50)	1	Paddy field
Uplandcrop,Open area	0.9 (0.6～1.2)	0.15	300 (300)	0.6	Hills and Forest land
Forest	0.9 (1.0～2.0)	0.25	150 (150)	0.6	Mountain
Settlment	0.3 (0.3～0.5)	0.6	55 (55)	1	Upland field, Farm, Golf course
Road&Rail,Urban area	0.1 (0.01～0.04)	0.7	55 (55)	1	Urban Area

( ) is standard value

2) Calculation Result

As a result of hydrogram simulation by flood inundation analysis model as mentioned in 5.3.2, the discharge volume at Manggarai design reference point is calculated as 720m3/s same as flood pattern on 30 January 2007.

Table 5.3-4 Peak Volume of Basin Design Discharge Volume in Design Rainfall

No.	Occurrence date	KatuLampa	Depok	Manggarai (Reference Point)	Karet
1	1994/01/20	272	413	387	431
2	2001/02/06	327	498	497	519
3	2006/01/23	341	379	380	397
4	2007/01/30	644	769	720	732

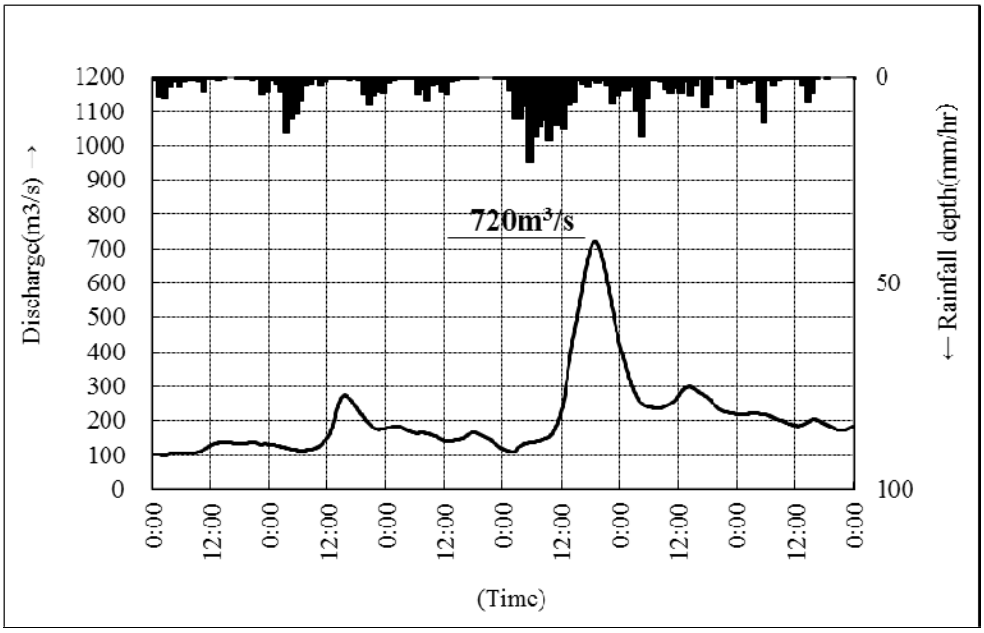


Figure 5.3-11 Hydrograph at Manggarai Point (Design Discharge: Flood Type on 30 January 2007)

(2) Calculation of Discharge Volume by Rational Formula

The peak discharge volume at the design reference point (Manggarai) is calculated by using rational formula based on 1/50 rainfall Intensity curve of Pondok Betung Cileduk and Damaga Bogor. The basin design discharge 720m m^3/s calculated by simulation using group of design rainfall patterns is small about 10 to 20 % as compared with the peak discharge calculated by the rational method shown in Table 5.3-5. It is judged as appropriate result.

Table 5.3-5 Peak Volume by Rational Formula

Rainfall Station	Runoff Coefficient f	Rainfall Intensity r (mm/hr)	Catchment Area A(km2)	Maximum Flood Discharge Q(m3/s)
Pondok Betung Cileduk	0.54	17.8	337.13	900.1
Damaga Bogor	0.54	15.9	337.13	804.1

Q=1/3.6・f・r・A

Table 5.3-6 Flood Arrival Time

Point	Elevation H1(m)	Elevation H2(m)	ΔH=H2-H1 (m)	Length (km)	Slope	V (m/s)	sectional time (min)	t1 (min)	t2 (min)	Tc=t1+t2 (min)
Katulampa	342.09	1071.47	729.38	20.424	1/ 28	3.5	97	30	97	127
Depok	62.50	342.09	279.59	41.810	1/ 150	3.0	232	30	329	359
Manggarai	6.56	62.50	55.94	47.403	1/ 847	2.1	376	30	705	735

Elevation Data From : BAKOSURTANAL

Table 5.3-7 Rainfall Intensity during Flood Arrival Time

Rainfall Station	Tc (min)	Rainfall intensity Curve Constant a	b	n	Rainfall Intensity r (mm/hr)
Pondok Betung Cileduk	735	2582.8	3.803	0.75	17.8
Damaga Bogor	735	12643.2	59.058	1.00	15.9

$r=a/(t^n+b)$

Table 5.3-8 Run-off Coefficient

Land use Classification	Area(km^2) (A)	Runoff Coefficient (f)	(A)×(f)
Settlemnet	94.800	0.50	47.400
Road	15.200	0.65	9.880
Urban	101.530	0.80	81.224
Opne Space	8.940	0.35	3.129
Forest	74.430	0.30	22.329
Upland Crops	37.190	0.30	11.157
Paddy Field	0.350	0.10	0.035
Water Area	4.690	1.00	4.690
total	337.130		179.844
mean		0.54	

(3) Verification by Past Largest Flood

By using the actual rainfall data of January 2007 flood as the largest rainfall, the adequacy of the peak discharge of basin design discharge volume is evaluated. In case of the wet conditions in the basin due to the long rainfall at the flood occurrence, the possibility of big flood occurrence will increase. Thus, the verification is conducted in the assumption of (a) design value (reproduction calculation value: wet condition is assumed as the same with 2007 flood) and (b) Saturation value (wet condition is assumed as the same with saturation).

The followings show the peak discharge at Manggarai reference point.

The basin design discharge 720m m^3/s calculated by simulation using group of design rainfall patterns is small about 5 to 6 % as compared with the maximum actual peak discharge shown in Table 5.3-9. It is judged as appropriate result.

Table 5.3-9 Peak Volume of Largest Recorded Flood (m^3/s)

Condition	Manggarai Reference Point
(a) Design (Reproduction)	767
(b) Saturation	762

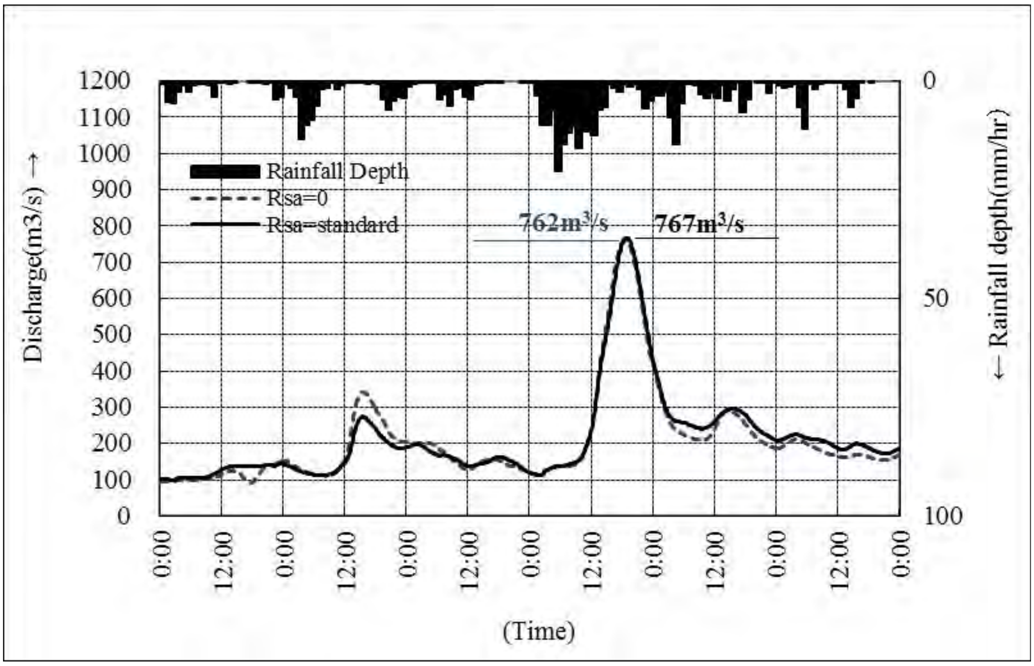


Figure 5.3-12 Hydrograph at Manggarai Point (Largest Recorded Flood: February 2007, Loss of Rainfall: Design Value, Saturation Value)

(4) Comparison of Catchment Area to Specific Discharge of Other Rivers in JABODETABEK Area

Figure 5.3-13 shows the basin design discharge volume in Cliwung river plotting in the specific discharge chart of other rivers in JABODETABEK area studied in 1997 master plan. Considering the design scale of W=1/50, it is said that the basin design discharge volume in the Project is adequate comparing with that in other rivers in JABODETABEK area.

Table 5.3-10 Comparison of Basin Design Discharge Volume and Specific Discharge ($m^3/s/km^2$) (Manggarai Point)

Point	Basin design discharge Volume (m^3/s)	Catchment Area (km^2)	Specific Discharge (m^3/s／km^2)
Manggarai (Manggarai Water Level Gauging Station)	720	337	2.14

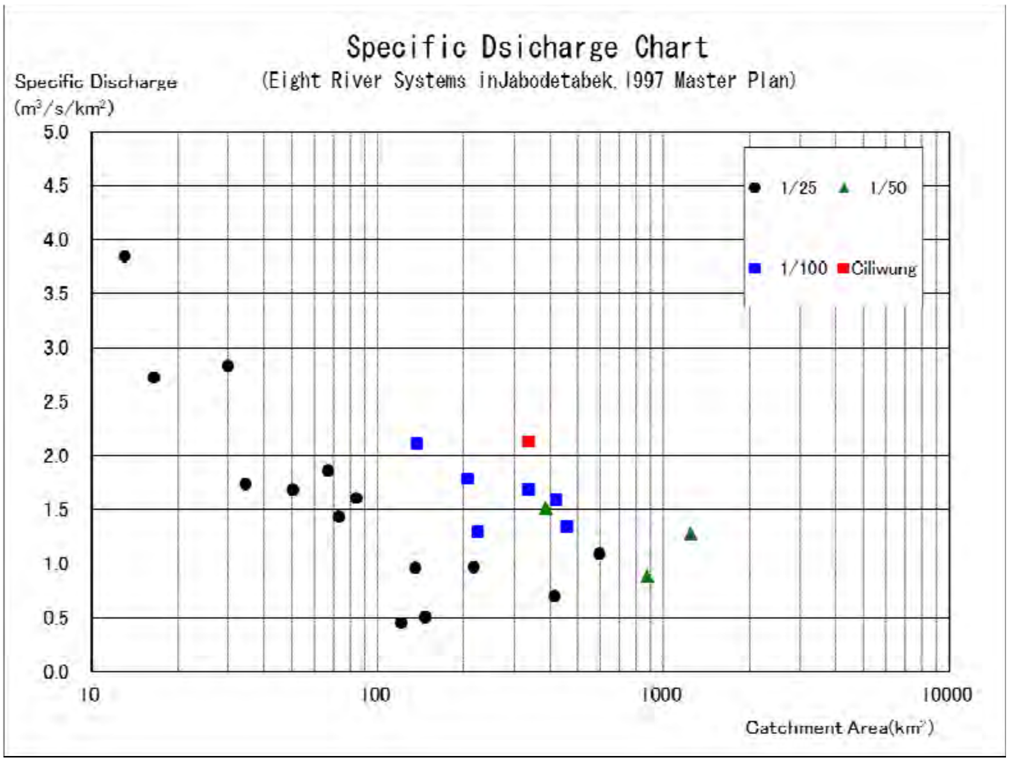


Figure 5.3-13 Comparison between Basin Design Discharge Volume of Ciliwung River and Specific Discharge of Other Rivers in JABODETABEK Area

(5) Determination of Basin Design Discharge Volume in the Basin

Examining the study results from 1) to 4) above, the peak discharge volume at design reference point (Manggarai point) is estimated as 720m^3/s by design rainfall curve.

The hydrograph of peak discharge volume at design reference point (Manggarai point) is shown in Figure 5.3-14.

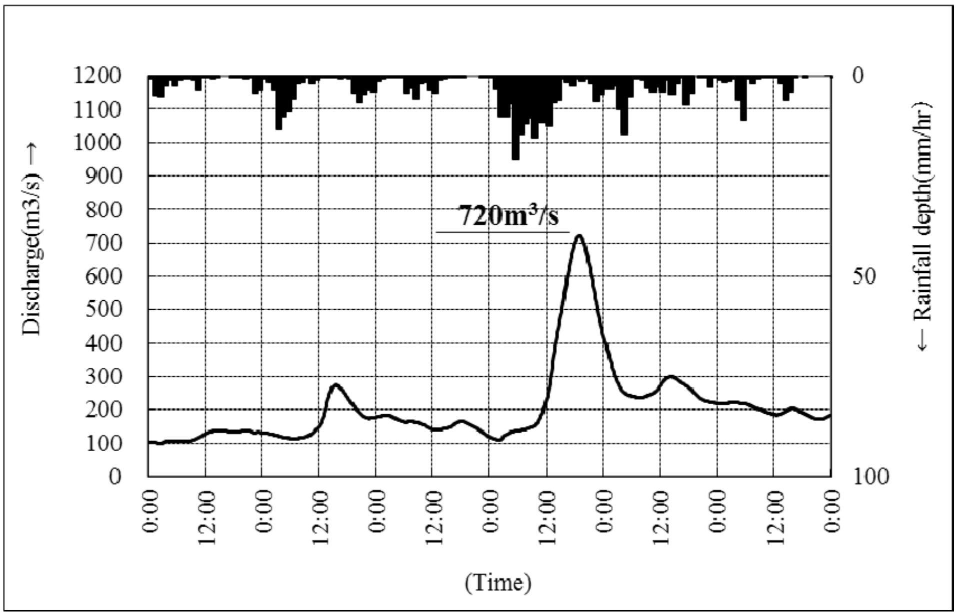


Figure 5.3-14 Hydrograph at Manggarai Point (Design Rainfall: Flood Type on 30 January 2007)

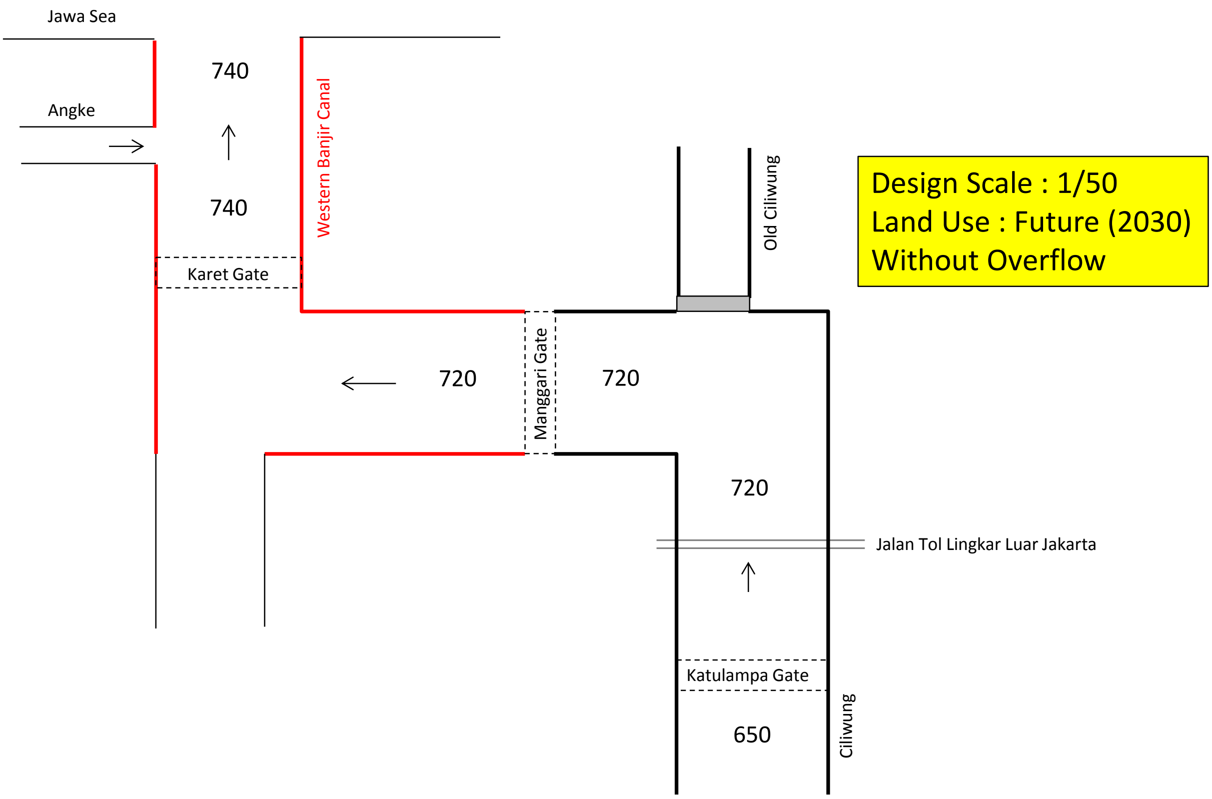


Figure 5.3-15 Allocation of Basin Design Discharge Volume

CHAPTER 6 FLOOD CONTROL MEASURES IN RIVER COURSE

6.1 Design Flood Discharge

Ciliwung River is an important river which runs through the center of JABODETABEK and many socio-economic key facilities are located in the basin. Urbanization in the basin has rapidly proceeded resulting increase of flood risk due to increase of peak discharge. Urbanization has also proceeded in the low land area of the basin downstream of Manggarai and construction of new flood control facility or improvement of WBC is difficult.

Based on the situation above and considering consistency with previous plans, project and programs, design flood discharge is determined to mitigate flood disaster due to increase of peak flood discharge under the future land use. The basic conditions for determination of design flood discharge are as follows.

- Consistency with design discharges of WBC and on-going river improvement works shall be maintained.
- Basic design discharge of 720 m^3/s at Manggarai Gate which is estimated assuming the future land use in 2030 is to be reduced to 500 m^3/s which is the design discharge of on-going river improvement works.
- Diversion of 60 m^3/s to EBC, which is included in the current flood control plan will be considered.
- Accordingly, the target control volume of structural measures including flood control measures and runoff control measures is 160 m^3/s.

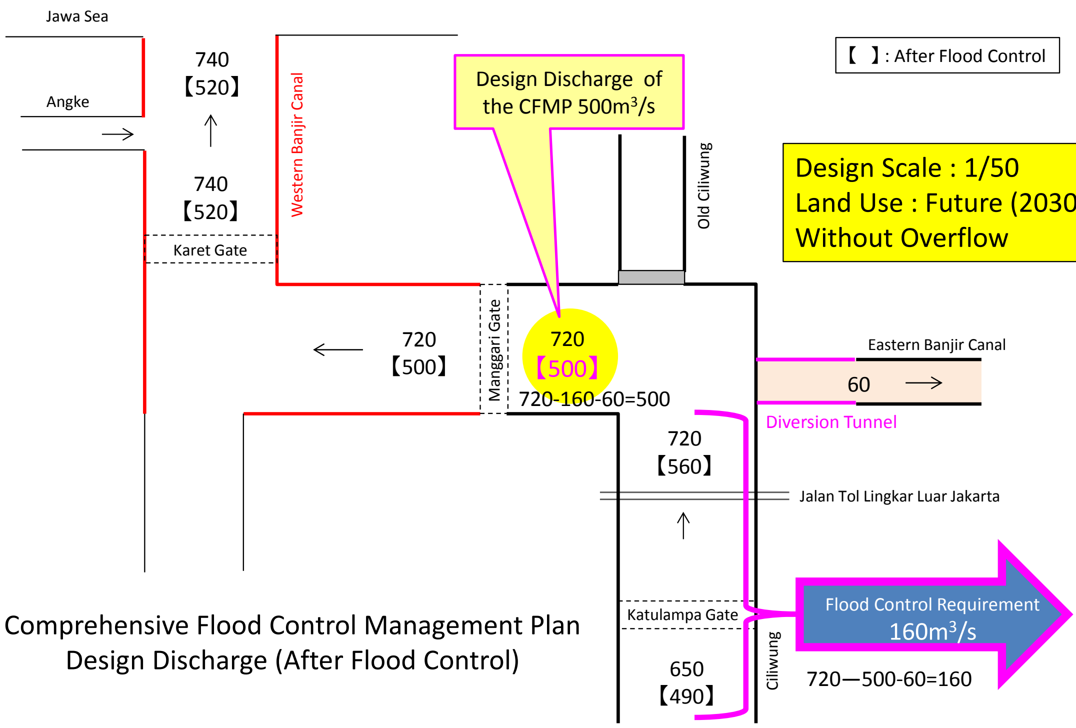


Figure 6.1-1 Design Flood Discharge Allocation

6.2 Flood Control Facilities in River Course

Candidate flood control facilities in Ciliwung River are summarized as shown in Table 6.2-1.

Table 6.2-1 Candidate Flood Control Facilities

Facility	Location/Section	Specifications
Large Dam	Upstream of Katulampa	Ciawi1 Dam: V=2,607,000m^3 (H=40m, Concrete Dam) Ciawi2 Dam: V=3,850,000m^3 (H=40m, Concrete Dam) Cisukabirus Dam: V=420,000m^3 (H=30m, Concrete Dam)
Small Dams	Upstream of Katulampa	Small Dams: 6 Locations, V=1,299,000m^3 （H=20m）
Gate Dams	Depok～Katulampa	Gate Dam: 2 Locations, V=479,000m^3 ・Pesona Kayangan V=173,000m^3 ・Bella Cassa V=306,000m^3
Tunnel Storage	Route 1 (MT.Haryono～Jawa Sea) Route 2 (Outer ring road～Krukut river)	Inside Diameter: φ＝12m, Length: L=20km, V=1,809,000m^3 Inside Diameter: φ＝12m, Length: L=6.1km, V=550,000m^3

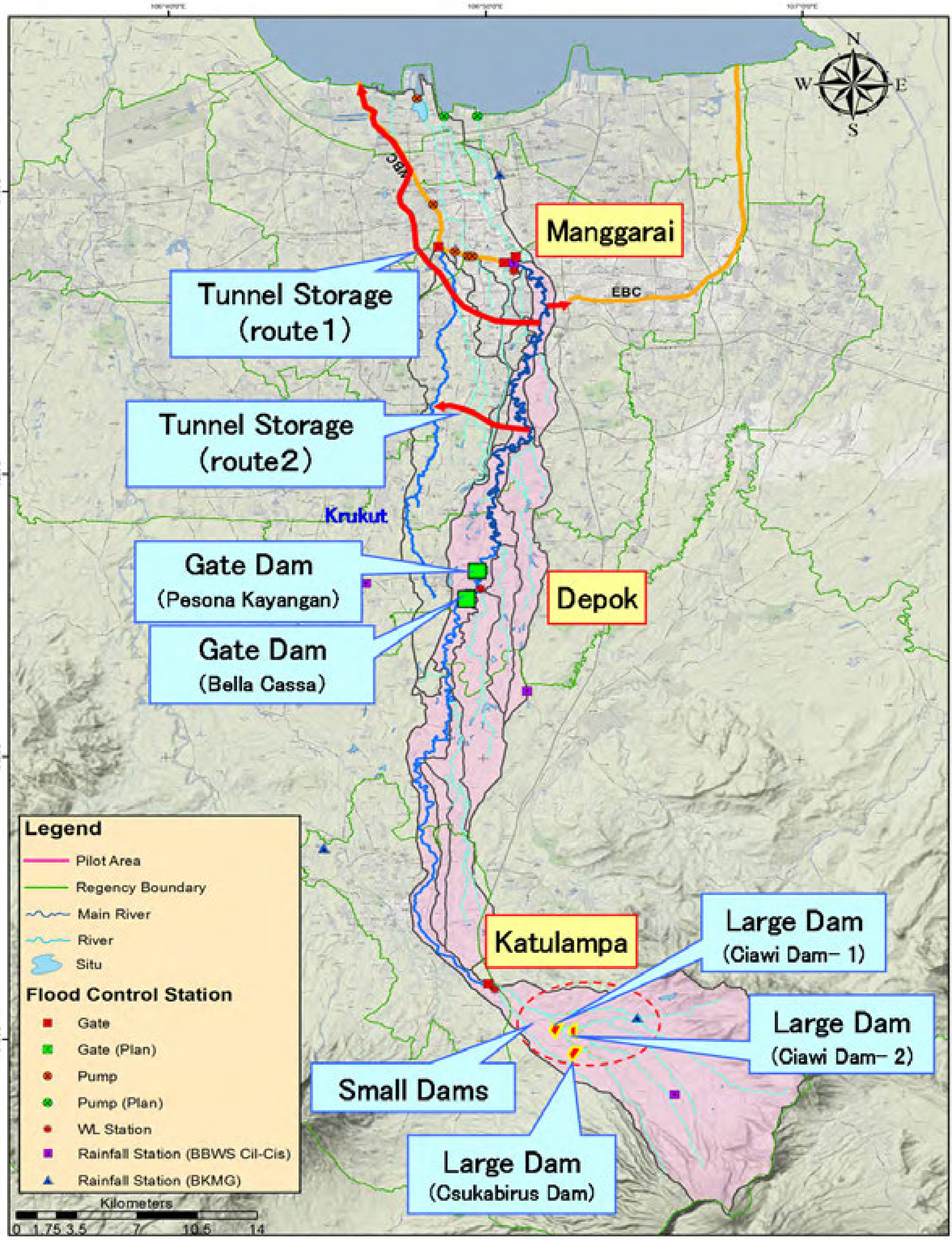


Figure 6.2-1 Locations of Candidate Flood Control Facilities in Ciliwung River

6.3 Large Dam

Optimal dam plan is examined in the as aspect of flood control effect at Manggarai, by comparing several alternatives of flood control dams.

6.3.1 Previous Dam Plan

(1) Previous Studies

Dam plans were formulated in the following studies.

- JICA Master Plan in 1997 (hereinafter referred to as the "1997 MP")
  "BRIEF NOTE ON CIAWI DAM DEVELOPMENT FOR FLOOD CONTROL PURPOSE, JULY 1996"
- Plan by PU in 2006 (hereinafter referred to as the "2006 BBWS")
  "LAPORAN AKHIB SEMENTARA PENYUSUNAN DETAIL DESAIN WADUK CIAWI TAHAP III, NOPEMBER 2006"

(2) Dam Sites and Specifications

Dam sites and their specifications in the previous studies are shown in Figure 6.3-1 and Table 6.3-1 respectively.

The 1997 MP proposed a flood control dam at upstream of Ciliwung River of which type is rockfill dam with height of 61m. The 2006 BBWS proposed a multipurpose dam at confluence of Ciliwung and Cisukabirus rivers of which type is rockfill dam with height of 90m.

In the 1997 MP, 5m difference between crest elevation of 567.5m and surcharge water level of EL562.5m was designed considering overflow depth of design discharge over spillway and free board. In the 2006 BBWS, 5.5m difference was designed. It is noted that dam heights shown in Table 6.3-1 means dam height from current riverbed elevation and the elevations of dam foundation are uncertain.

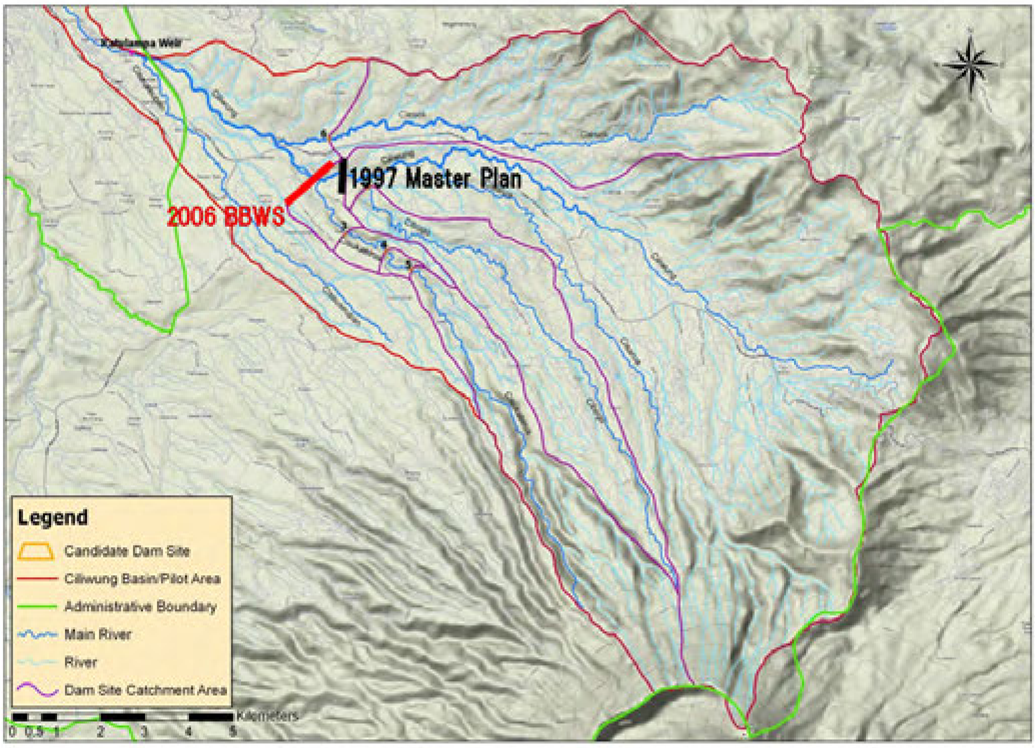


Figure 6.3-1 Locations of Proposed Dam Sites in Previous Studies

Table 6.3-1 Specifications of Proposed Dams in Previous Studies

Item	Unit	1997 MP	2006 BBWS
Dam type		Rock fill Dam	Rock fill Dam
Dam height	m	61.0	90.0
Dam volume	m^3	-	13,897,227
Catchment area	km^2	88.0	105.1
Water surface area	km^2	0.5250	1.4688
Gross storage volume	m^3	8,719,000	41,440,000
Effective storage volume	m^3	2,119,000	35,670,000
Flood control volume	m^3	2,119,000	5,770,000
Water use capacity	m^3	-	33,290,000
Sediment deposit	m^3	6,600,000	2,380,000
Non-overflow section elevation	EL.m	567.5	570.5
Design flood level	EL.m	566.5	569.5
Surcharge water level	EL.m	562.5	565.0
Riverbed elevation	EL.m	506.5	480.5
Sediment deposit level	EL.m	556.0	514.0
Flood control effect			
Dam point	m^3/s	270(400-130)	
Manggarai Weir	m^3/s	70(570-500)	123(472-349)

(3) Boring Survey

1) Locations and Numbers of Boring Survey

Locations and numbers of boring survey conducted in the previous studies are summarized in Figure 6.3-2 and Table 6.3-2 - Table 6.3-3.

Table 6.3-2 Boring Survey Conducted in 1997 MP

Dam site	Drilling site	Drilling depth (m)	Remarks
Ciawi Dam	CD-1	60	Right abutment
	CD-2	40	River bed
	CD-3	60	Left abutment
Total	3holes	160	

Table 6.3-3 Boring Survey Conducted in 2006 BBWS

Site	Drilling site	Drilling depth (m)	Remarks
Dam	BA-2	70	
	BA-6	70	
	DD-15	85	
	DD-8	80	
	BA-11	70	
	DD-3	60	DD-3=AW-3
	DD-2	60	DD-2=AW-2
	DD-1	40	DD-1=AW-1
Spillway	AW-1	20	N Value
	AW-2	20	N Value
	AW-3	80	N Value
	AW-4	60	N Value, Lu Value
	AW-5	60	N Value
	AW-6	40	N Value
Total	11(14)holes	815	

Although different names are given, DD-1, DD-2 and DD-3 bores might be same as AW-3, AW-2 and AW-1 bores, respectively.

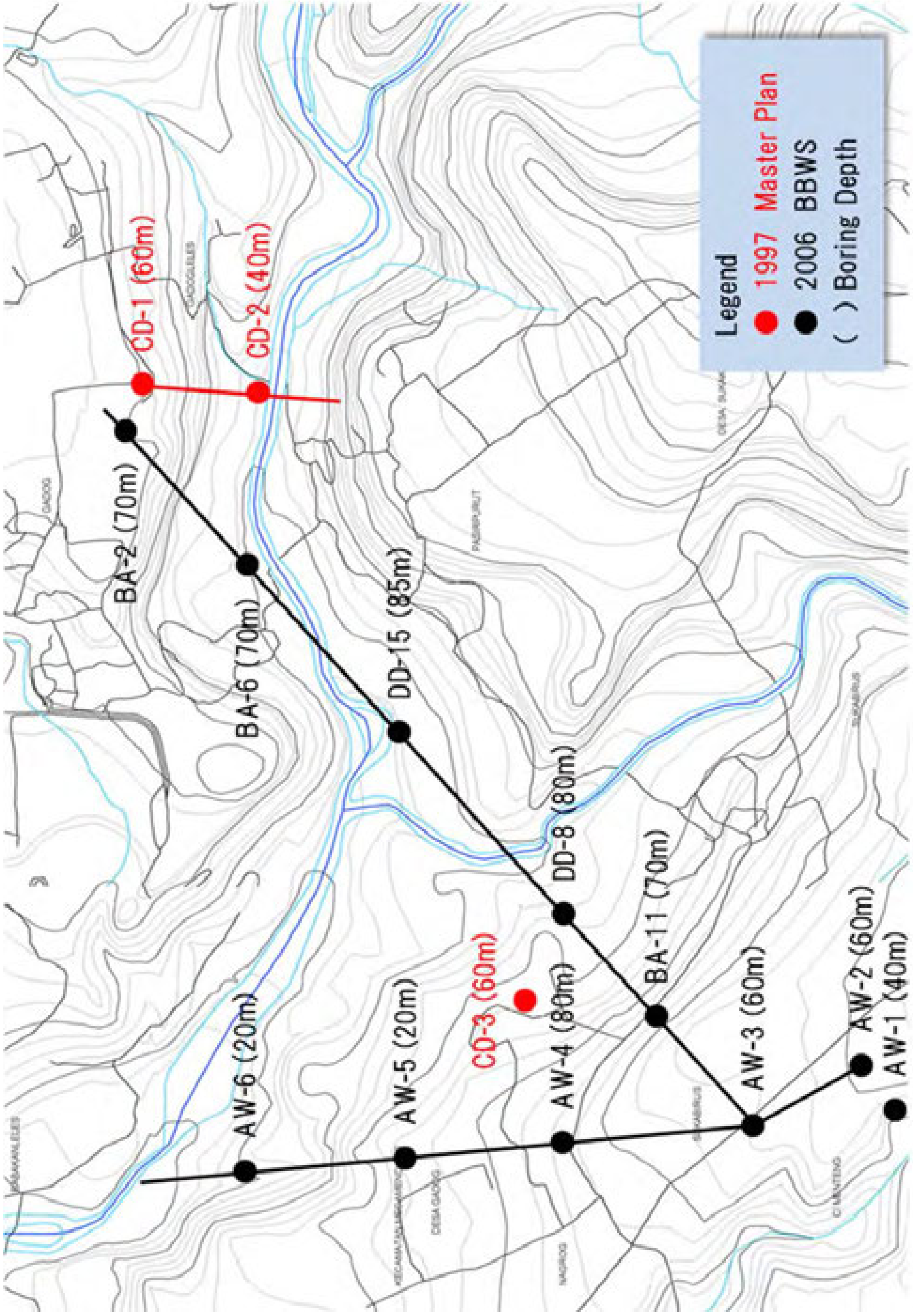


Figure 6.3-2 Locations of Boring Survey

2) Results of Boring Survey

The results of boring surveys in previous studies are summarized in Table 6.3-4, while the evaluations of geology are summarized as follows.

Table 6.3-4 Summary of Boring Survey Records in Previous Studies

			1997 MP	2006 BBWS	Remarks
Boring Survey	Locations of Bore		○	○	Location maps are available.
	No. of Bore	Dam	3	5	
		Spillway	×	6	
	Borehole Log	Dam	×	×	
		Spillway	×	○	N Value and Lu Value are available.
	Geological Profile	Dam	×	○	Geological profile of dam axis is available.
		Spillway	×	×	
	Evaluation		○	×	Evaluation is available only in 1997 MP.

- Ciawi Dam site is composed of Younger Volcanic Rocks of G. Pangrango named in the Geological Pap of Bogor Quadrangle (1986), which consists of old deposits, lahar and lava, andesitic basalt with oligoclase-andesite, labradorite, olivine etc., mostly strongly weathered.
- The drilling test results reveals existence of an intensively weathered layer with approximately 20m thickness, consisting of brownish clay to dark brown sandy silt with N values in the range of 2 to 14.
- The lower layer consisting of Breccia and Lava unit from Mt. Kencana, is an intercalation of andesite lava, gravely sand, fine sand, silty clay, andesite and breccia, with N values more than 50 in general.
- The river deposits, with a thickness of approximately 13m, consisting of loose sand and gravel with boulders, are found along the riverbed at the dam site. The lower part of layer consisting of gravelly sand or sand layer and breccia shows high permeability of which Lugeon unit ranges from 10 to 50.
- To determine quarry site of core rocks for rockfill dam, further geotechnical survey is required in the areas which are composed of the Old Volcanic Rocks.
- It is concluded that the rockfill dam with a vertical clay core can be recommended of which maximum height is 60m. Further, it is important to keep in mind the existence of a rather thick layer of river deposits in the riverbed.

(4) Design Sediment Storage Capacity

Design sediment storage capacity was planned in the 1997 MP as follows.

A) Sediment storage capacity of Ciawi Dam is estimated as 1.5 – 2.0mm/year/km^2 based on past experiences in West Java.

B) Relation between sediment storage capacity and effective capacity of Ciawi Dam is summarized as shown in table below.

Relation between Sediment Storage Capacity and Effective Capacity of Ciawi Dam

Case	Catchment Area (km2)	Sediment Yield (mm/year)	Duration (years)	Sediment Volume (×10^6m3)	Sediment Level (EL,m)	Effective StorageVol (×10^6m3)	Remarks
(i)	88.0	2.0	100	17.6	overHWL	–	
(ii)	88.0	2.0	50	8.8	overHWL	–	
(iii)	88.0	1.5	100	13.2	overHWL	–	
(iv)	88.0	1.5	50	6.6	556.0	2.119	8.719–6.6

C) As shown in above table, sediment will overflow within 50 years assuming annual sediment yield of 2.0mm/year/km^2. The effective capacity of 2.12 million m^3 can be secured assuming annual sediment yield of 1.5mm/year/km^2 and lifetime of 50 years.

D) Thus, Ciawi Dam plan with 60m height is feasible assuming minimum sedimentation conditions which are annual sediment yield of 1.5mm/year/km^2 and lifetime of 50 years.

There is no description about estimation of design storage capacity in the 2006 BBWS Report. Annual sediment yields are estimated as follows by calculating back from the design sediment storage capacity of 2.38 million m^3 and the catchment area of 105.1km^2.

- 50 Years Life Time: 0.45mm/year/km^2
- 100 Years Life Time: 0.23mm/year/km^2

These values range in 1/3 to 1/6 of the assumption in the 1997 MP.

6.3.2 Selection of Candidate Dam Sites in This Study

(1) Selection Criteria

As selection criteria, the following topographic or land use features are considered.

- Downstream site is preferable to maximize reservoir capacity as much as possible.
- Difficulty of land acquisition/compensation shall be avoided.

Upstream basin of Ciliwung River Basin consists of volcanic deposits and secondary deposits.
Since riverbed width and valley become wide at downstream of confluence of Ciliwung and Cisukabirus rivers, the candidate dam site shall be at upstream the confluence.

In the upstream basin, valley is relatively deep and riverbed slope is relatively steep as shown in Figure 6.3-7. Cisukabirus River is steeper than Ciliwung River but the both rivers have same characteristics that upper reaches are steeper.

Downstream site is preferable as much as possible to maximize reservoir capacity since downstream site can secure reservoir capacity with larger catchment area and gentle river slope. In the upstream area, there are dense housing areas as shown in Figure 6.3-8 and there is a power plant at the downstream of confluence. It is important to avoid difficulty of land acquisition or compensation for early implementation.

Figure 6.3-3 Residential Area at Banks of Ciliwung River (Upstream of Confluence of Cisukabirus River)

Figure 6.3-4 Villa at Upstream Area of Ciliwung River

Figure 6.3-5 Exposure of Base Rock at Upstream of Ciliwung River

Figure 6.3-6 Exposure of Base Rock at Upstream of Cisukabirus River

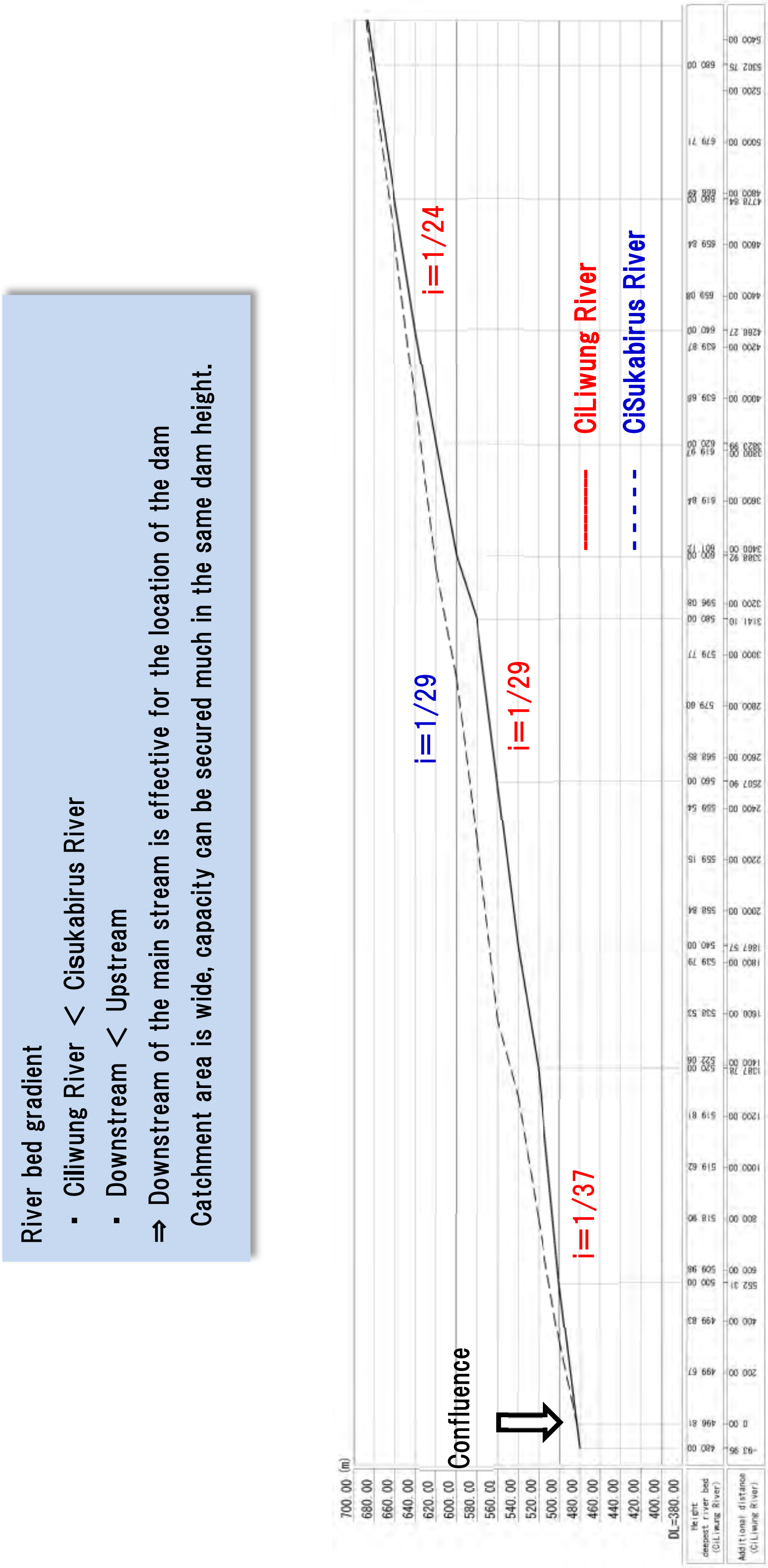


Figure 6.3-7 Riverbed Slopes of Ciliwung and Cisukabirus Rivers Upstream of Confluence

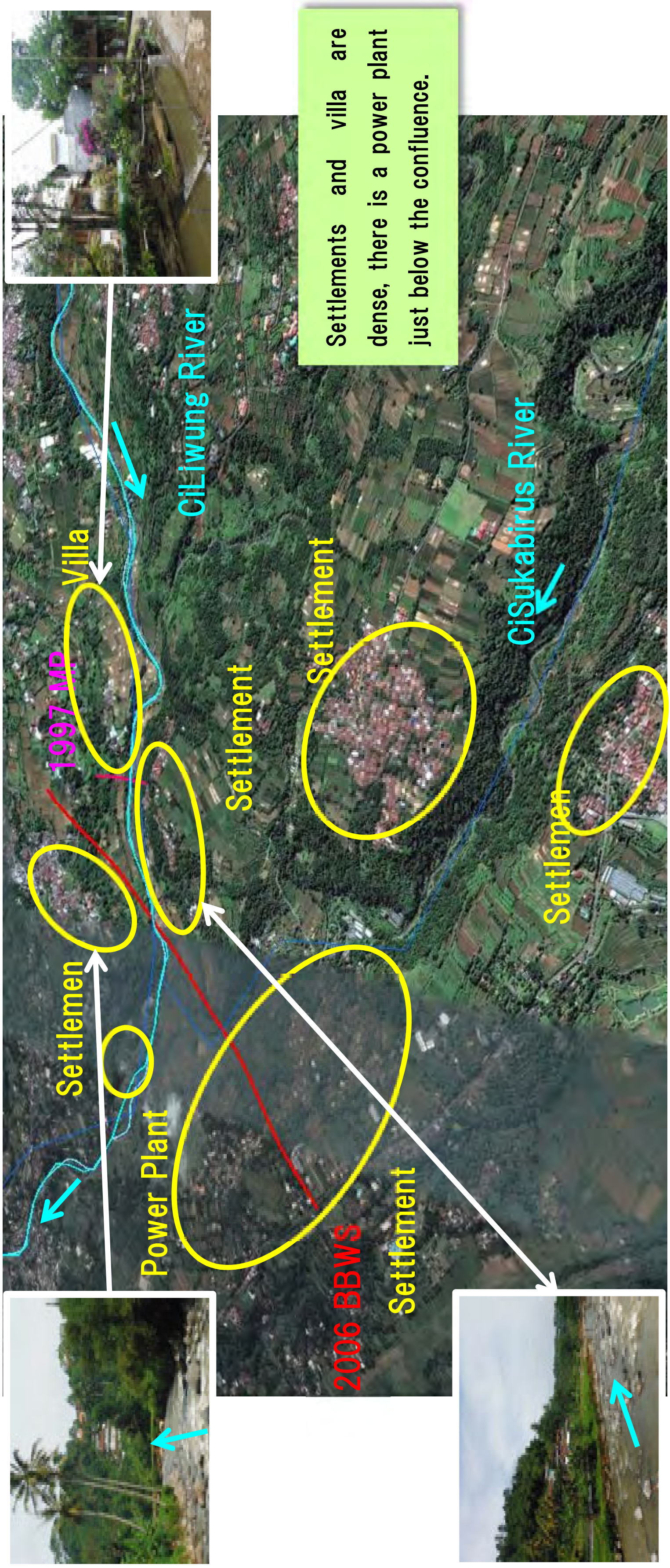


Figure 6.3-8 Land Use of Ciliwung and Cisukabirus River Basins Upstream of Confluence

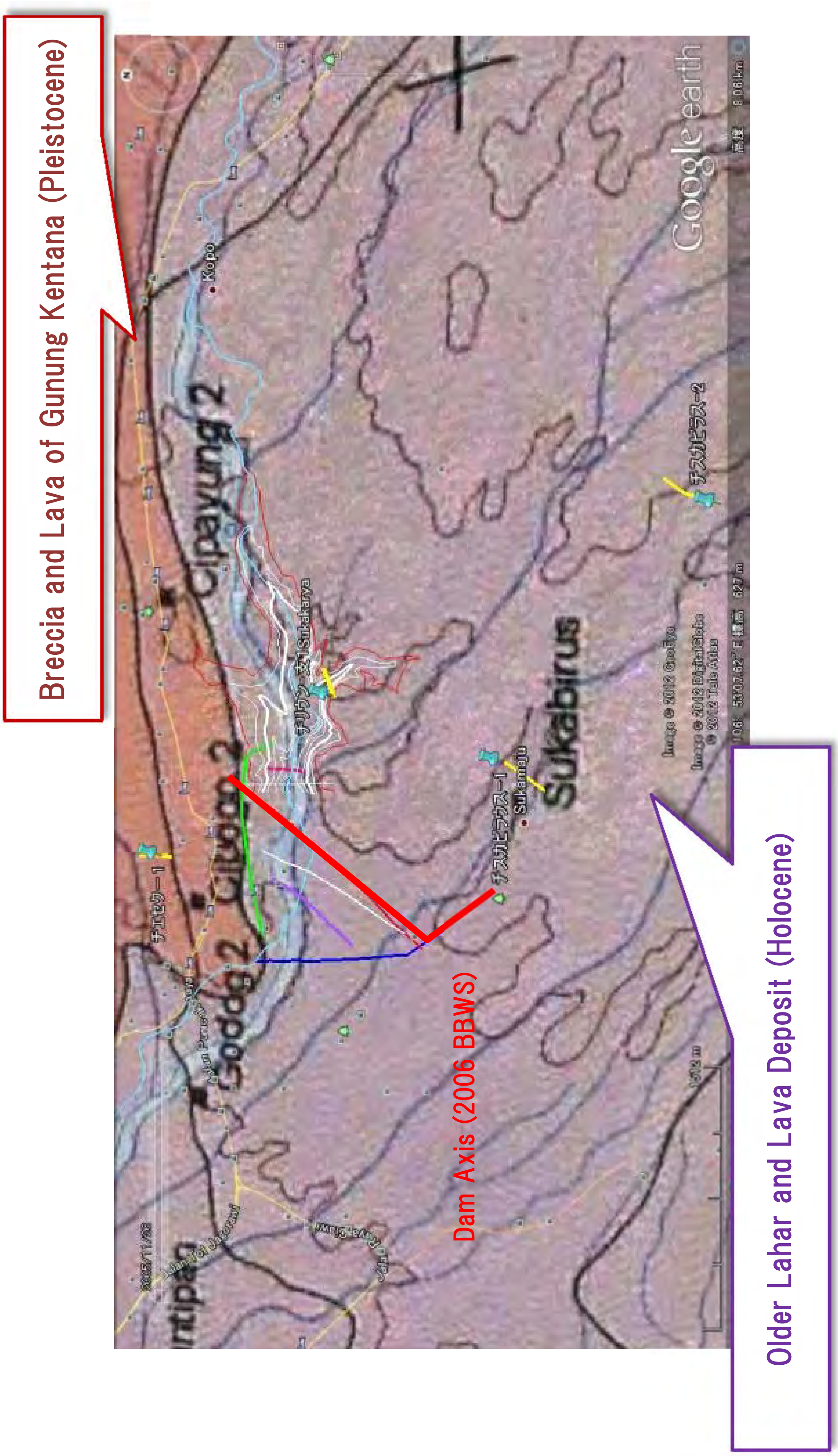


Figure 6.3-9 Geology of Dam Candidate Area

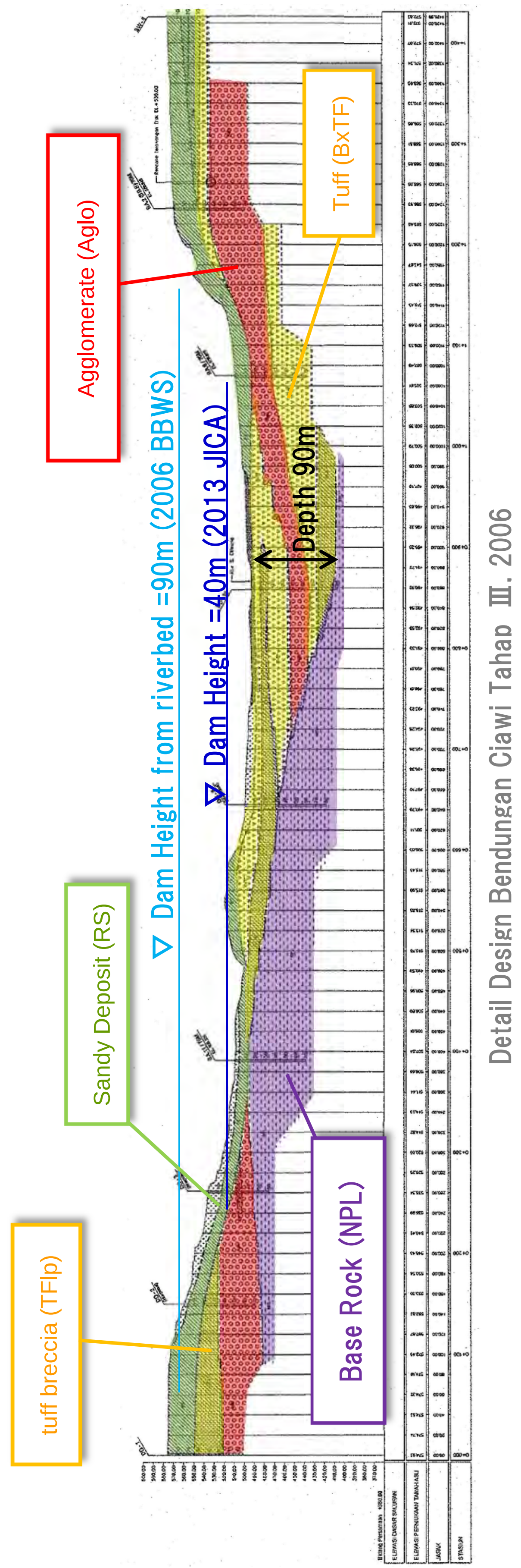


Figure 6.3-10 Geological Profile of Proposed Dam Axis in 2006 BBWS Study

(2) Selection of Candidate Dam Sites

Considering topography and land use as well as geology, the following 3 dam sites as shown in Figure 6.3-11 are selected as candidates.

Ciawi Dam-1: Downstream of confluence which can secure largest catchment area. Ciawi Dam-2: Just upstream of confluence. Cisukabirus Dam: Narrow Section

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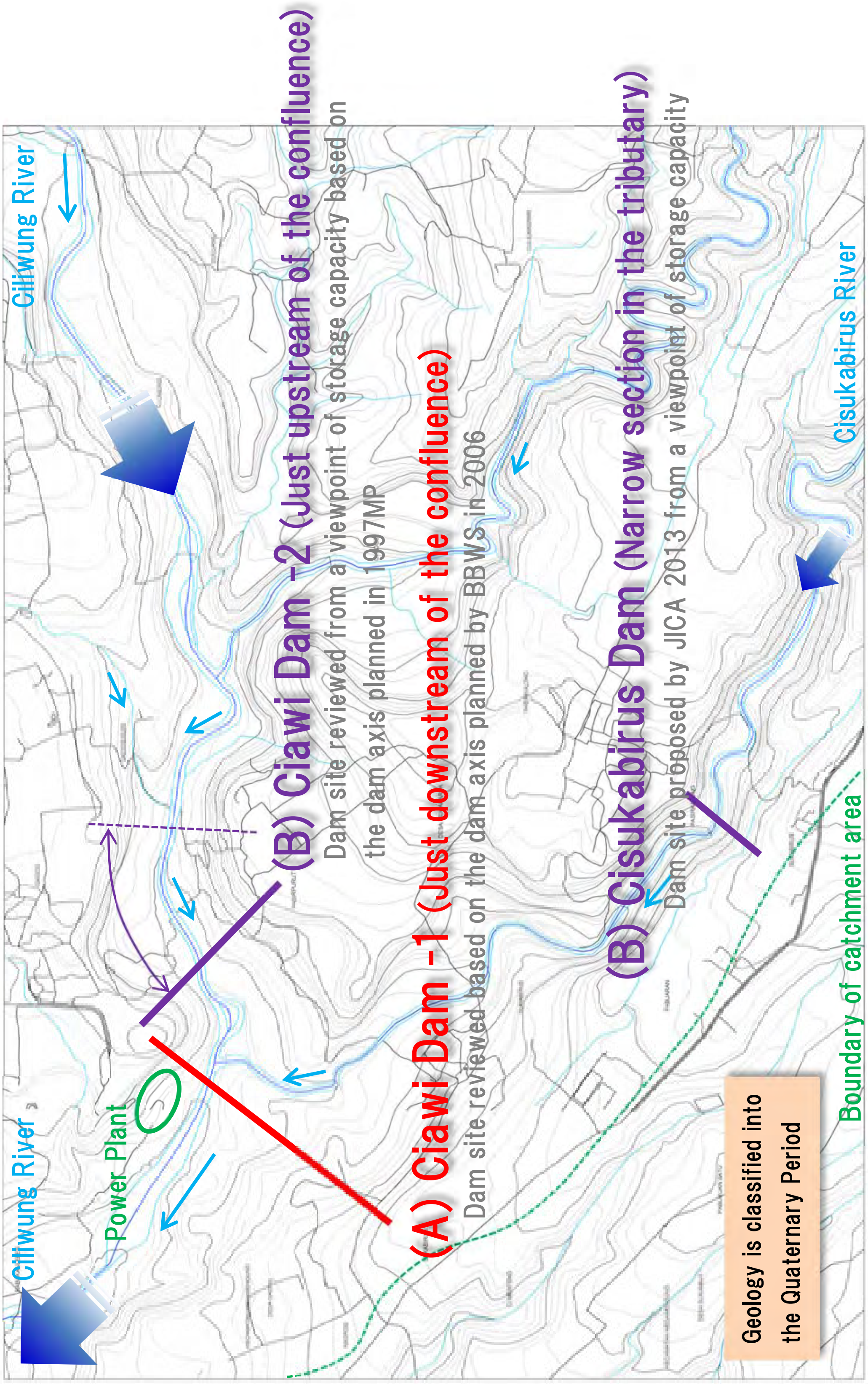


Figure 6.3-11 Candidate Dam Sites

6.3.3 Dam Height and Type

(1) Issues on Dam Planning

Based on topographic and land use features of the site, issues on examination of available dam height and type are as follows.

A) Dam sites which can secure reservoir capacity are limited due to steep riverbed gradient and heights of ridges.
B) Base rocks are belongs to Quaternary Layer which has low strength. Seepage failure due to head of reservoir shall be considered.
C) Sediment inflow volume is large from surface erosion and tributaries including boulders.
D) Social constraint such as land acquisition and compensation shall be considered.

(2) Available Dam Height and Type

Regardless of dam type, available dam height depends on conditions of seepage failure of foundation. Figure 6.3-10 shows geological profile of the proposed dam axis in BBWS 2006. As shown in Figure 6.3-10, dam site consists of relatively new tuff or hard weathered tuff breccia, and agglomerate underneath. For such foundations, the following conditions shall be considered.

<New Tuff or Hard Weathered Tuff Breccia>
A) Strength of foundation is low.
B) Permeability is relatively low but seepage failure can be occurred due to high hydraulic gradient if dam height is too high.
<Agglomerate>
C) Permeability is high and improvement is difficult.

Available dam height is governed by the above conditions A) and B).

Regarding the condition A), available maximum dam heights are estimated as 60m for rockfill dam and 40m for concrete dam based on the experiences in Japan.

Regarding the condition B), the limit pressure shall be estimated by P-Q curve through permeability test. Based on the experiences in Japan of construction of dam with similar foundation conditions, the limit pressure is estimated 0.4MPa and available dam height of 40m.

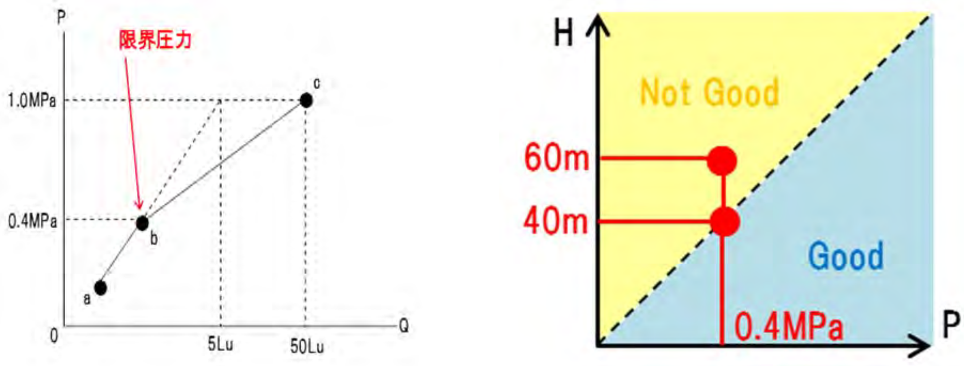


Figure 6.3-12 Limit Pressure by P-Q Curve

As similar experiences in Japan, Fujinami Dam which is rockfill storage dam and Nishinoya Dam which is concrete dry dam are listed.

Fujinami Dam has height of 52m with storage capacity of 2.95 million m^3. Right side of dam site consists of lutaceous sandy gravel layer in Quaternary Period with permeability of 5Lu and limit pressure of 0.5MPa.

Cisukabirus River consists of Holocene volcanic deposits which is newer than Fujinami Dam Site and concreteness of foundation is expected as lower than Ciliwung River Basin. Thus, available maximum dam height is estimated as 30m since resistance to seepage failure is small.

Table 6.3-5 Available Maximum Dam Height and Type

Factor	Ciliwung Dam -1, 2	Cisukabirus Dam
1) Strength of foundation ground	Concrete Gravity H<40m Rock-fill H<60m	Concrete Gravity H<30m Rock-fill H<60m
2) Resistance to seepage failure (piping) of foundation ground	H<40m	H<30m

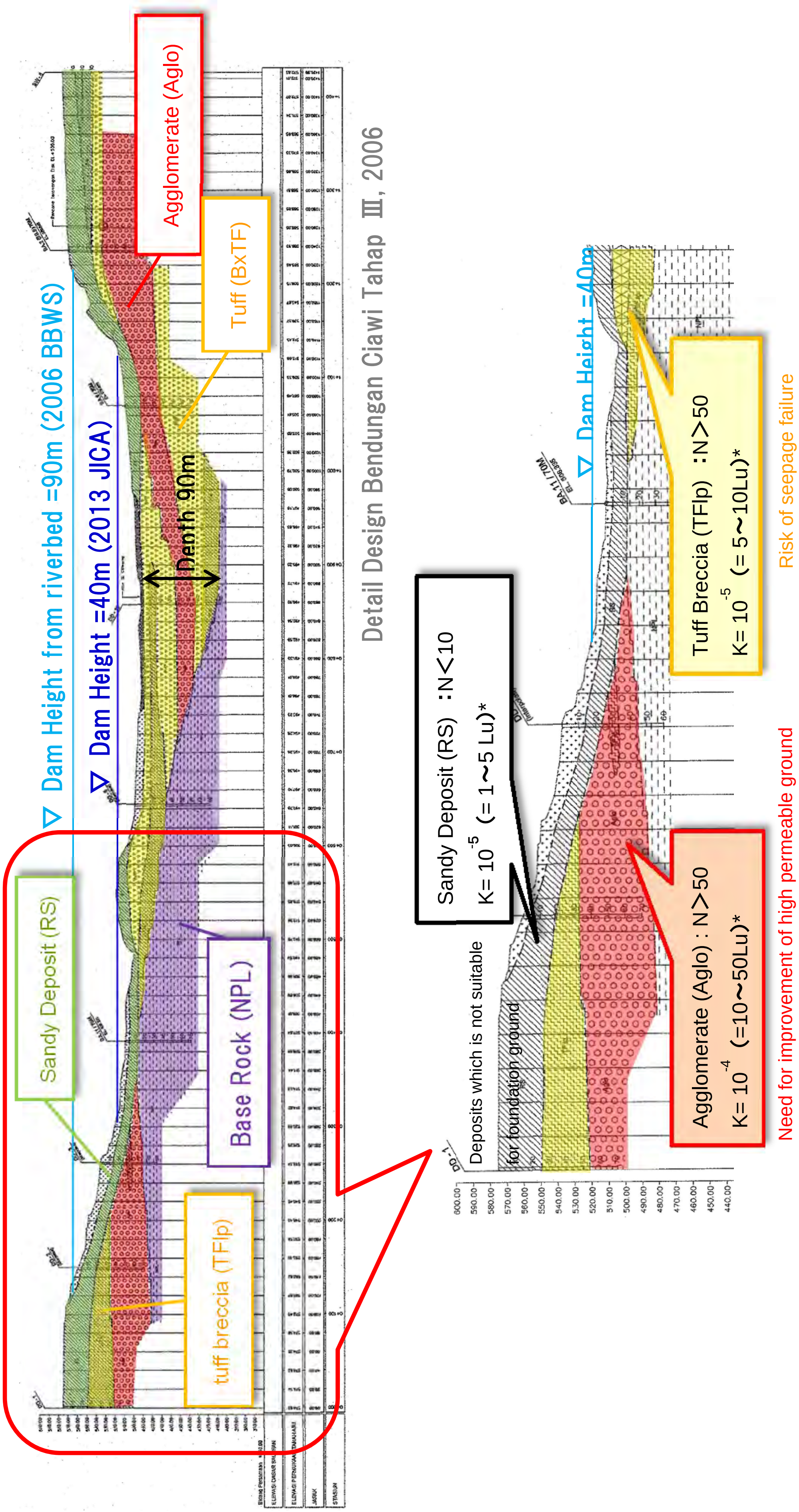
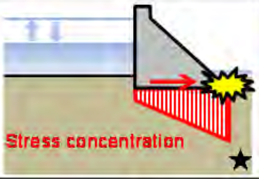
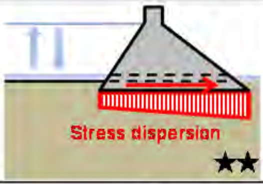
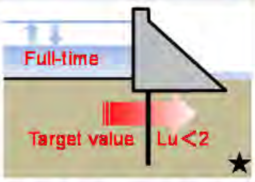


Figure 6.3-13 N Values and Permeability

(3) Comparison of Storage Dam and Dry Dam

Storage dam and dry dam have different functions; storage dam can be a multipurpose dam while dry dam is only for flood control. To determine dam type, Agglomerate with high permeability of 10 to 50Lu shall be considered. Agglomerate is a cracky rock and improvement of permeability is relatively easy comparing tuff or tuff breccia. However, maximum improvement by grouting is to reduce Lugeon Value of one digit. By grouting to foundation of 50Lu, permeability can be improved to 5Lu as maximum. In general, storage type dam requires 2Lu after improvement while dry dam type is applicable with 5Lu since dry dam stores flood water temporary and high permeability is affordable. Therefore, dry dam is appropriate to the proposed dam sites as shown in Table 6.3-6.

Table 6.3-6 Comparison of Storage and Dry Dam Types

	Storage Dam		Dry Dam
1. Dam Type	Concrete gravity dam	Rock-fill dam	Concrete gravity dam
2. Function	Multi-Purpose ★★★		Only Flood Control ★
3. Water Quality	Affected ★		Not Affected ★★★
4. Sedimentation	Enough capacity for sedimentation needs to be taken into account. ★		Sedimentation can be flushed to downstream ★★★
5. Bearing Capacity and Shear Strength of Foundation	 ★	 ★★★	 ★★
6. Target for Permeability Improvement (to minimize water leakage)	 ★	 ★★	 ★★★
Evaluation	"Dry Dam" is suitable for these dam sites, because a) target for permeability improvement can be reduced because of temporary water rising and b) storage capacity can be used efficiently since most sediment can be flushed to downstream through openings.		
	★	★★	★★★

6.3.4 Selection of Dam Sites and Specifications

(1) Candidate Dam Sites

As the dam planning, the following two alternatives are examined applying dam heights of 40m for Ciawi Dam and 30m for Cisukabirus Dam.

Plan A: Ciawi Dam-1 at Confluence of Ciliwung and Cisukabirus Rivers Plan B: Combination of Ciawi Dam-2 at Upstream of Confluence and Cisukabirus Dam

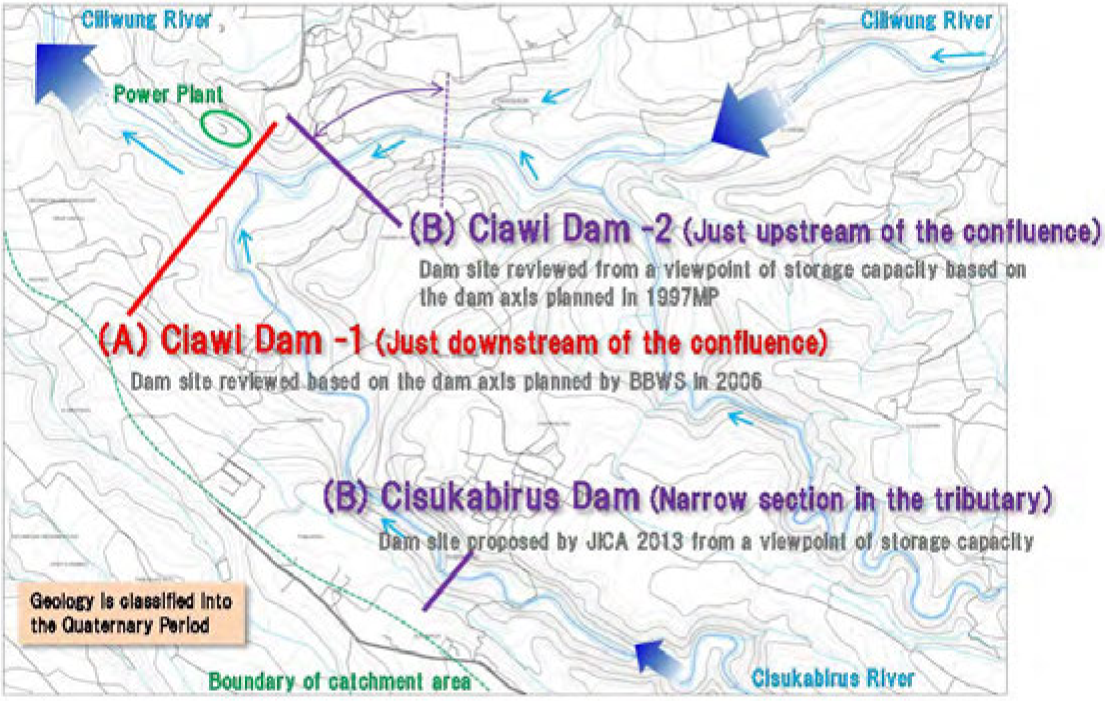


Figure 6.3-14 Candidate Dam Sites

The areas where compensation is required are shown in Figure 6.3-15.

Ciawi Dam-1 affects the settlements located downstream of Cisukabirus River while Ciawi Dam-2 affects a villa located upstream basin of Ciliwung River.

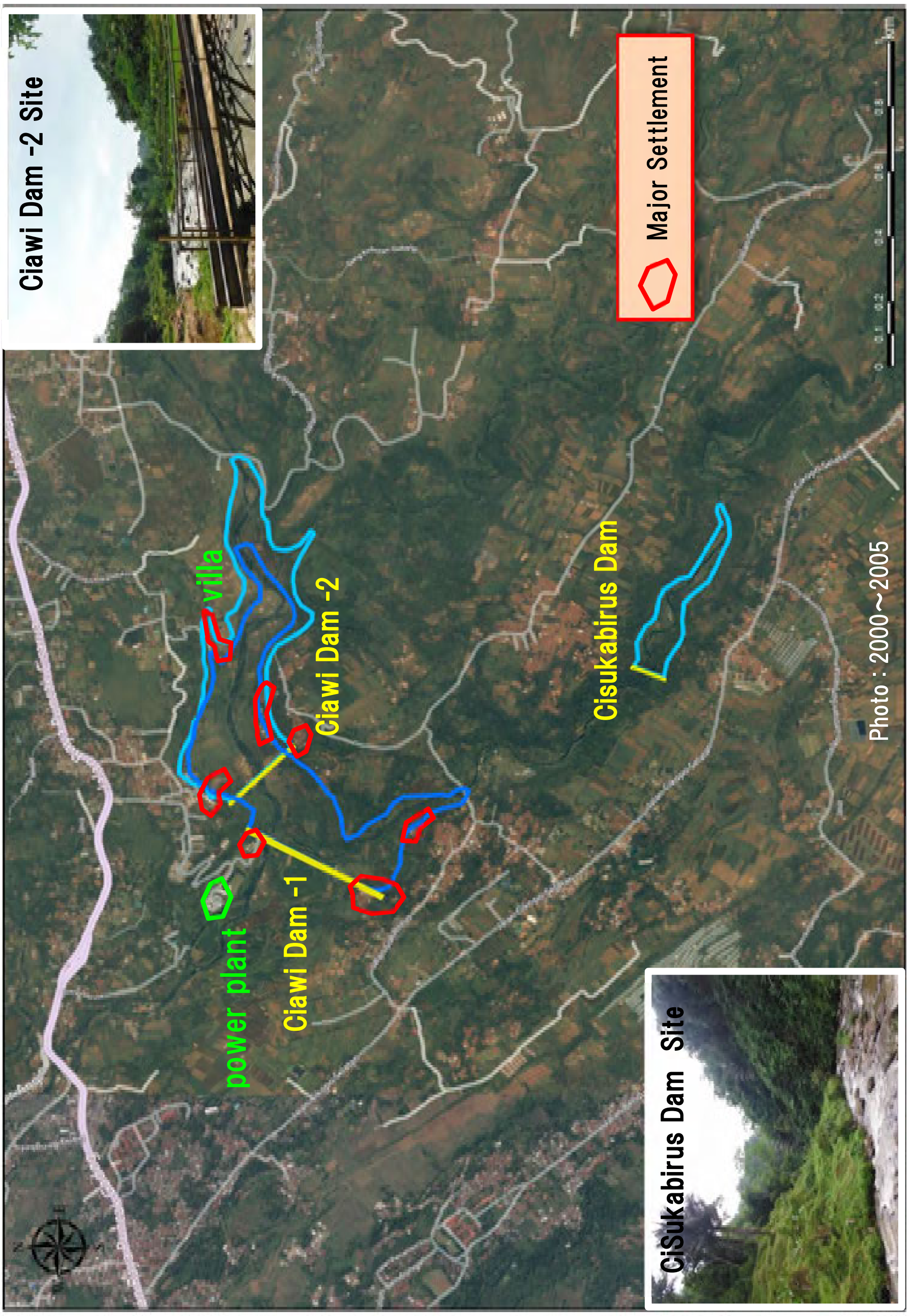


Figure 6.3-15 Dam Sites and Affected Areas

(2) Specifications of Proposed Dams

Assuming the concrete dry dam type with the aforementioned available dam heights, specifications and upstream views of proposed dams are shown in Table 6.3-8 and Figure 6.3-16, respectively. Approximate dam volumes are summarized in Table 6.3-7.

Ciawi Dam-1: Height 40m, Crest Length 600m, Dam Volume 438,000m^3
Ciawi Dam-2: Height 40m, Crest Length 375m, Dam Volume 320,000m^3
Cisukabirus Dam: Height 30m, Crest Length 150m, Dam Volume 8,000m^3

Table 6.3-7 Estimated Dam Volumes

Ciawi Dam -1					Ciawi Dam -2					Cisukabirus Dam				
Station No.	Distance (m)	Area (m^2)	Average area (m^2)	Volume (m^3)	Station No.	Distance (m)	Area (m^2)	Average area (m^2)	Volume (m^3)	Station No.	Distance (m)	Area (m^2)	Average area (m^2)	Volume (m^3)
Left bank		0			Left bank		0			Left bank		0		
	60		82	4,890		60		348	20,850		15		85	1,268
		163					695					169		
	360		713	256,500		60		979	58,710		30		449	13,470
		1,262					1,262					729		
	105		1,262	132,510		135		1,262	170,370		75		729	54,675
		1,262					1,262					729		
	60		713	42,750		45		979	44,033		30		365	10,935
		163					695					0		
	15		82	1,223		75		348	26,063					
Right bank		0			Right bank		0			Right bank				
Length of Dam	600	m	Total	437,873	Length of Dam	375	m	Total	320,025	Length of Dam	150	m	Total	80,348

Table 6.3-8 Specifications of Proposed Dams

Item	Unit	2013 JCFM		
		Ciawi Dam -1	Ciawi Dam -2	Cisukabirus Dam
Dam Type		Dry Dam (Gravity Concrete Dam)		
Dam Site Geology		Pleistocene Quaternary Volcanic Sediments	Pleistocene Quaternary Volcanic Sediments	Holocene Quaternary Volcanic Sediments
Maximum Dam Height	m	40.0	40.0	30.0
Dam Crest Length	m	600	375	150
Dam Volume	m^2	438,000	320,000	80,000
Catchment area	km^2	105.5	88.5	15.8

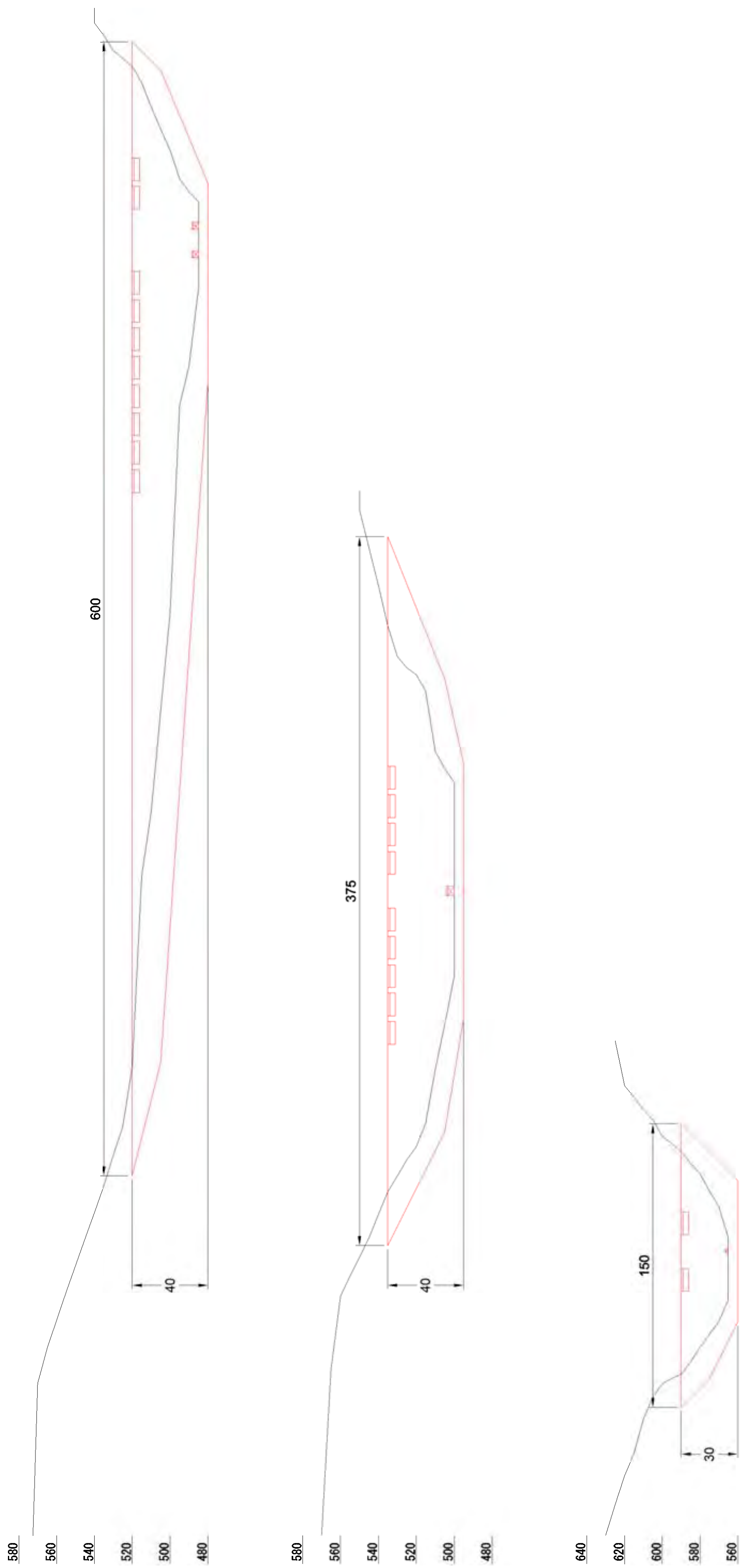


Figure 6.3-16 Schematic Upstream Views

6.3.5 Flood Control Effect by Large Dams

Flood control effects of the proposed alternatives at flood control points including the dam suites and Manggarai are estimated examining the relation of flood control volume and design of service outlet of dam.

(1) Study Case

The following two cases are examined.

Plan A: Ciawi Dam-1 at Confluence of Ciliwung and Cisukabirus Rivers
Plan B: Combination of Ciawi Dam-2 at Upstream of Confluence and Cisukabirus Dam

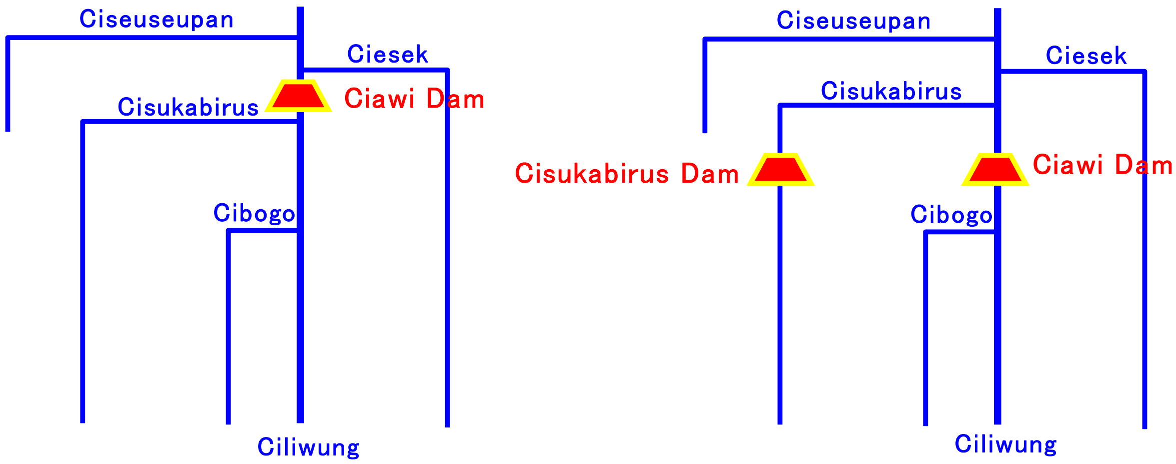


Figure 6.3-17 Schematic Figure of Dam Locations

(2) Specifications of Proposed Dams

1) Basic Specifications

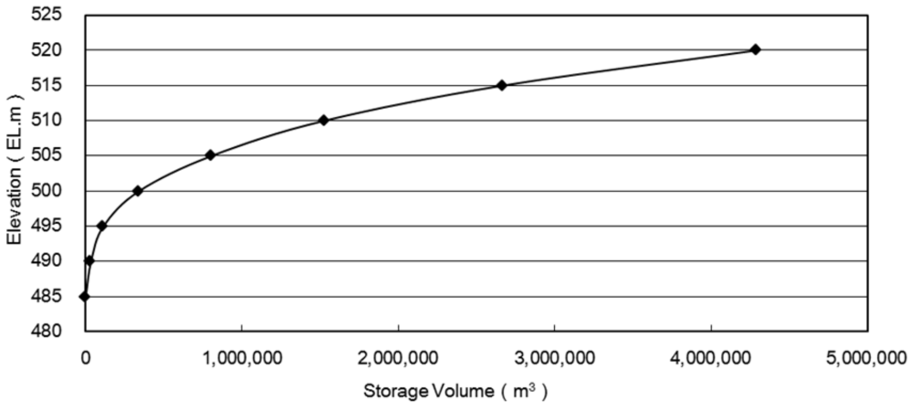
Basic specifications of proposed dams are as follows.

Table 6.3-9 Basic Specifications of Proposed Dams

Item	Unit	2013 JCFM		
		Ciawi Dam -1	Ciawi Dam -2	Cisukabirus Dam
Dam Type		Dry Dam (Gravity Concrete Dam)		
Dam Site Geology		Pleistocene Quaternary Volcanic Sediments	Pleistocene Quaternary Volcanic Sediments	Holocene Quaternary Volcanic Sediments
Maximum Dam Height	m	40.0	40.0	30.0
Dam Crest Length	m	600	375	150
Dam Volume	m^2	438,000	320,000	80,000
Catchment area	km^2	105.5	88.5	15.8
Gross Storage Volume	m^3	2.607×10^6	3.850×10^6	0.420×10^6
Flood Control Volume	m^3	2.607×10^6	3.850×10^6	0.420×10^6
Dam Crest Elevation	EL.m	520.0	535.0	590.0
Surcharge water level	EL.m	515.0	530.0	585.0
Riverbed Elevation	EL.m	485.0	500.0	565.0
Sediment Deposit level	EL.m	485.0	500.0	565.0
Foundation Elevation	EL.m	480.0	495.0	560.0
Gate Elevation	EL.m	485.0	500.0	565.0
Gate Dimension (B×H)	m	3.6×3.0×2gate	4.9×3.6	2.1×1.7

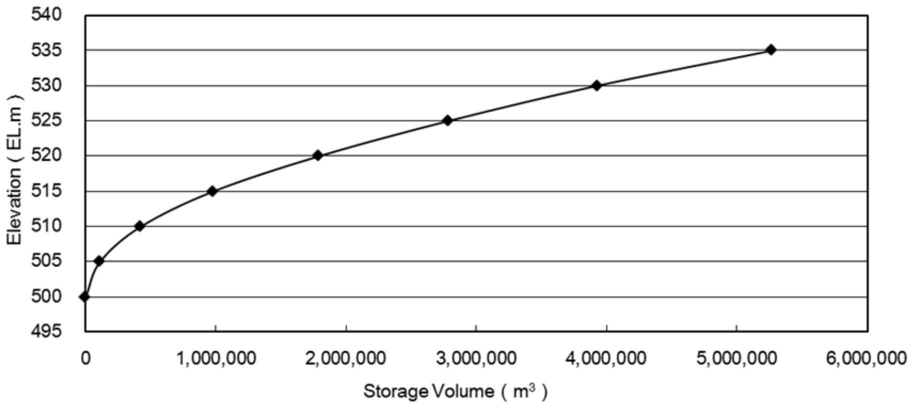
Ciawi Dam 1

Elevation (EL.m)	Storage Volume (m^3)
485	0
490	33,500
495	113,000
500	341,625
505	806,500
510	1,528,500
515	2,673,125
520	4,296,750



Ciawi Dam 2

Elevation (EL.m)	Storage Volume (m^3)
500	0
505	110,600
510	425,050
515	983,250
520	1,794,900
525	2,788,750
530	3,931,275
535	5,274,255



Cisukabirus Dam

Elevation (EL.m)	Storage Volume (m^3)
565	0
570	30,270
575	104,053
580	238,299
585	443,702
590	705,226
595	1,022,559
600	1,434,091

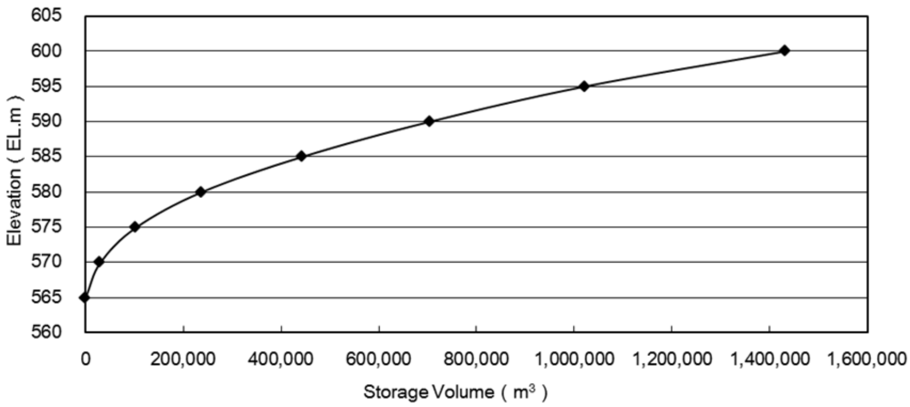


Figure 6.3-18 H-V Curves of Proposed Dams

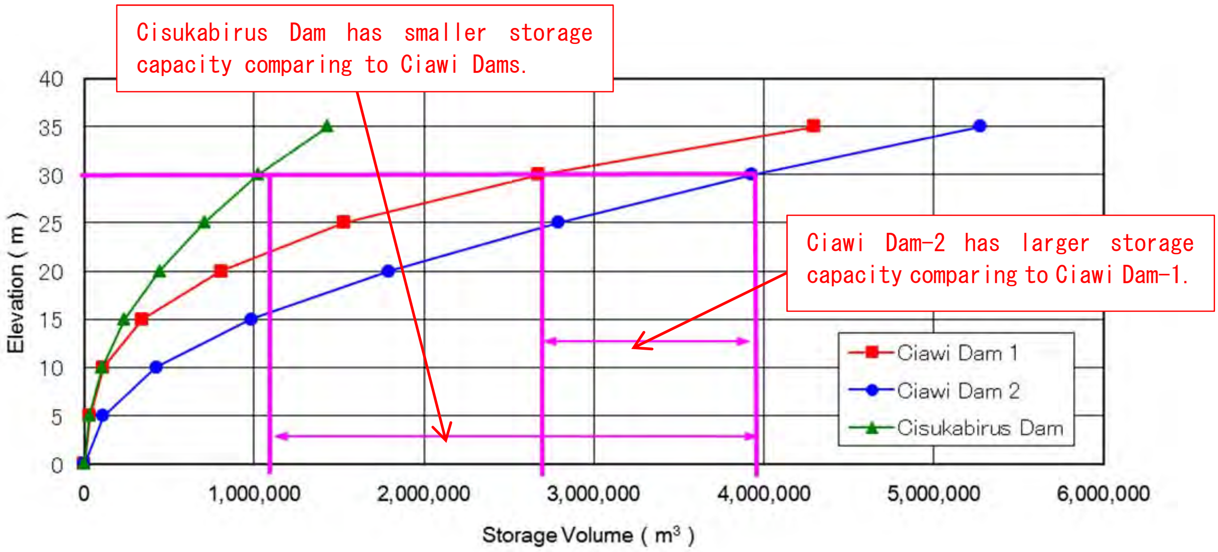


Figure 6.3-19 Comparison of H-V Curves

- Until storage water level of 30m, Ciawi Dam-2 has larger storage capacity comparing to Ciawi Dam-1.
- Cisukabirus has smaller storage capacity comparing to Ciawi Dams.

2) Design of Service Outlet

The services outlets of proposed dams are designed by trial with considering flood control volume of surcharge water level (S.W.L) of the proposed dams.

For design of service outlet, design scale shall be examined carefully since step wise improvement of dam is difficult and it shall be designed considering future safety degree of downstream river course.

The Comprehensive Flood Management Plan targets 1/50 years floods of which smaller than ordinary dam planning of which ranges 1/100 to 1/200 years. On the other hand, as shown in Table 6.3-11, 1/100 year improvement is required in the future. Since the scale of dam is limited by geological conditions of site and step wise improvement such as heightening is impossible, the service outlets of proposed dam is designed based on the following concepts.

- Service outlet is designed with 1/100 years flood.
- Since the dam improvement works to increase storage capacity is impossible, flow capacity of the service outlet shall be designed to discharge 1/100 flood safely. Thus, larger flow capacity against 1/50 years flood shall be required for the service outlet is required. As tentative measures to discharge 1/50 flood, the service outlet is partially closed using stop log. By this mean, same S.W.L. of 1/50 years and 1/100 years can be maintained. Design of the stop log shall be simple one without control and is to be demolished in future when 1/100 years flood control of river course is achieved.
- Considering the safety of dam body, the service outlet shall be less than 5m x 5m which is 1/3 of one block of dam body. If more flow capacity is required, several outlets shall be installed.

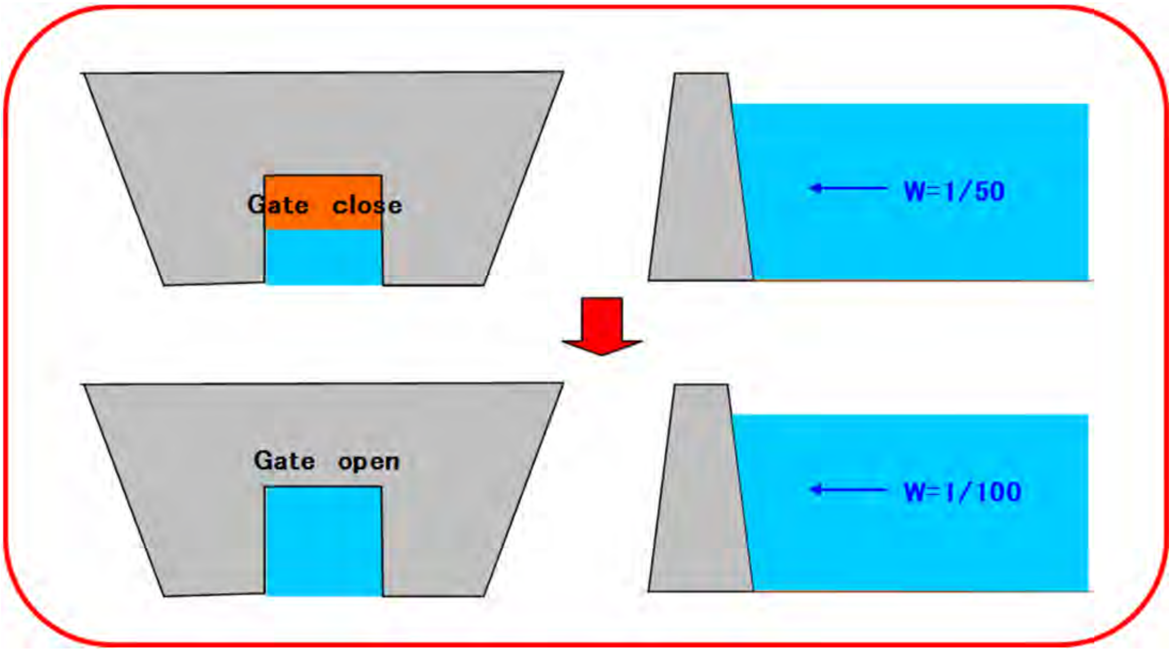


Figure 6.3-20 Image of Service Outlet Design

Table 6.3-10 Design of Service Outlet of Each Proposed Dam

Ciawi Dam 1

Return period	Height of Dam (m)	Orifice			Flood control Volume (m^3)	SWL (EL.m)	Max Outflow (m^3/s)
		Width (m)	Height (m)	Number of Gate			
50	40	3.6	3.0	2	2,606,904	514.7	308.6
100	40	3.6	3.6	2	2,503,322	514.3	364.2

Ciawi Dam 2

Return period	Height of Dam (m)	Orifice			Flood control Volume (m^3)	SWL (EL.m)	Max Outflow (m^3/s)
		Width (m)	Height (m)	Number of Gate			
50	40	4.4	3.6	1	3,849,672	529.6	222.0
100	40	4.4	4.4	1	3,799,347	529.4	267.1

Cisukabirus Dam

Return period	Height of Dam (m)	Orifice			Flood control Volume (m^3)	SWL (EL.m)	Max Outflow (m^3/s)
		Width (m)	Height (m)	Number of Gate			
50	30	2.1	1.7	1	420,046	584.4	40.9
100	30	2.1	2.1	1	402,471	584.0	49.5

Table 6.3-11 Design Flood Recurrence Intervals Used in Indonesia

Project Name	Design Recurrence Interval (Years)					
	Agricultural Development		Rural Development		Urban/Industrial Development	
	Short Term	Long Term	Short Term	Long Term	Short Term	Long Term
Cimanuk & Cisanggarung River Basin Development Project (West Java)	Varies 10 to 15	Varies 25 to 50	Varies 10 to 15	Varies 25 to 50	Varies 10 to 15	Varies 25 to 50
Citanduy River Basin Development Project (West Java)	25	50	25	50	25	50
South Kedu Multipurpose Project (Central Java)	Varies 5 to 15	Varies 15 to 20	15	20	15	20
Solo River Basin Development Project (Central Java)	Varies 5 to 10	50	10	50	Varies 10 to 50	50
Brantas River Basin Development Project (East Java)	Varies 10 to 25	50	25	50	25	50
Pemali Flood Control Project (Central Java)	5	25	5	25	5	25
Jakarta Metropolitan Flood Control Project (West Java)	25	100	-	100	-	100
Krueng Aceh Flood Control Project (Aceh)	5	-	5	-	5	-
Lower Asahan River Flood Control Project (North Sumatra)	25	-	25	-	25	-
Padang Urban Flood Control Project (West Sumatra)	25	50	25	50	Varies 10 to 25	50
Jeneherang River Basin Development Project (South Sulawesi)	Varies 10 to 25	50	25	50	25	50

3) Width of Emergency Spillway

Width of emergency spillway shall be less than the crest length of dam. Necessary width of emergency spillway is estimated with the following conditions.

- Flow depth shall be less than 2m.
- Design discharge is the peak discharge of 1/200 years flood at the dam location.
- The following equation is applied.
  Q = C・B・H1.5
  Where, Q: Discharge (m^3/s), B: Overflow width (m), C: Discharge coefficient ($m^{0.5}/s$) =1.7, H: Overflow depth (m) = 2.0m

As shown in below, necessary overflow width is less than the crest length of proposed dams.

Table 6.3-12 Overflow Width of Emergency Spillway

Dam	Inflow (W=1/200) (m^3/s)	Overflow depth (m)	Overflow width (m)	Crest Length (m)
Ciawi Dam 1	561	2	117	600
Ciawi Dam 2	481	2	101	375
Cisukabirus Dam	72	2	15	150

(3) Calculation of Flood Control Volume

Flood control volume is calculated by differentiating the following formula.

$dV/dt = Q_{in} – Q_{out}$

Where, V: Storage volume (m^3), Q_{in}: Inflow discharge (m^3/s), Q_{out}: Outflow discharge (m^3/s), dt: Calculation interval (s) = 10 min

The above formula can be differentiated as follows.

（Vt – Vt-1）/Δt = (Qint+Qint-1)/2 - (Qoutt+Qoutt-1)/2

Where, t: Current time, t-1: Δt Previous time

Since Vt-1, Qint, Qint-1 and Qoutt-1 are known amount while Vt and Qout is unknown amount, above equation can be expanded as follows.

Vt/Δt + Qoutt/2 = Vt-1/Δt + (Qint+Qint-1)/2 – Qoutt-1/2

Since relation of storage volume and water level is given by H-V curve, the function Vt = V(Ht) can comprise. On the other hand, outlet discharge is governed by water level with outflow characteristics, the function Qoutt = Qout(Ht) can comprise. Thus, the water Ht which satisfies the above formulas is calculated by trial calculations.

Outflow characteristics of the service outlet can be divided into the following three types according to water level H which is converted to water depth to the bottom of outlet and height of outlet D.

1) Free Over Flow (H/D≦1.3)
2) Transition Flow (1.3＜H/D＜1.8)
3) Closed Conduit Flow (Orifice Flow) (H/D≧1.8)

The range of H/D varies depending on type of outlet. Referring to conduit flow condition of one side bell mouth H/D＞1.3～1.8, closed conduit condition of H/D≧1.8 and transition flow condition of 1.3＜H/D＜1.8 are applied.

1) Free Over Flow

The following equation is applied.

Q = C・B・H1.5

Where, Q: Discharge (m^3/s), B: Overflow width (m), C: Discharge coefficient ($m^{0.5}$/s) =1.6, H: Overflow depth (m) = 2.0m

It is noted that the discharge coefficient of 1.6 is applied considering loss by contracted flow since width of outlet is very small comparing crest length.

2) Transition Flow

Discharge of transition flow is estimated by linear interpolation of free flow and closed conduit flow. Since the range of transition flow is 1.3＜H/D＜1.8, water level H ranges 1.3D＜H＜1.8D.

Upper Limit of Free Flow: Q1 = C・B・H1.5 = C・B・（1.3D）1.5

Where, C: Discharge coefficient ($m^{0.5}$/s) = 1.6

Lower Limit of Conduit Flow: Q2 = C・B・D・（2g・H）1.5 = C・B・D・（2g・（1.8D））1.5

Where, C: Discharge coefficient ($m^{0.5}$/s) = (0.408 – 0.311・(D/(1.8D))0.5

Thus, Q = A・H + B

Where, A = (Q2 – Q1)/(0.5D), B = Q1 – A・(1.3D)

3) Closed Conduit Flow (Orifice Flow)

Low pressure orifice is defined as the conduit outlet with active water head less than 25m. Since the proposed dam is dry dam of which water depth is less than 30m and water head loads only during flood, flow condition is considered as low pressure orifice. However, work pressure and flow conditions shall be confirmed by hydraulic model test.

Popular type of low pressure orifice is one side bell mouth type and knife edge type. Knife edge type requires larger flow area than one side bell mouth type so that it is generally applied when clogging by drift woods or garbage is concerned. Since there is much sediment, drift woods and garbage discharge to the dam site, knife edge type shall be applied to prevent clogging.

Discharge by closed conduit flow (orifice flow) is calculated by the following formula.

Q = C・B・D・（2g・H）0.5

Where, Q: Discharge (m^3/s), B: Width of conduit (m), D: Height of conduit (m), C: Discharge coefficient ($m^{0.5}$/s), H: Water head (m) (=Water Level – Elevation of Conduit Bottom)

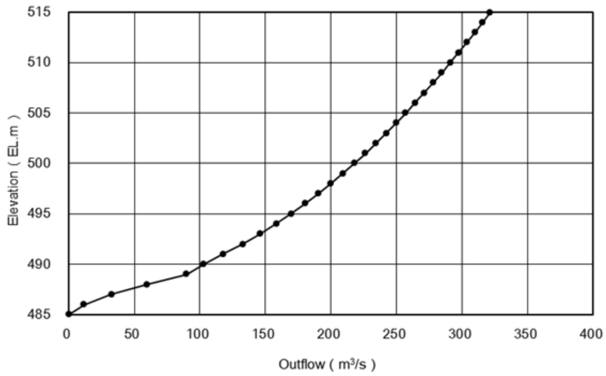
Discharge coefficient C ($m^{0.5}$/s) is estimated by the following formula.

C = (a – b・D／H)0.5

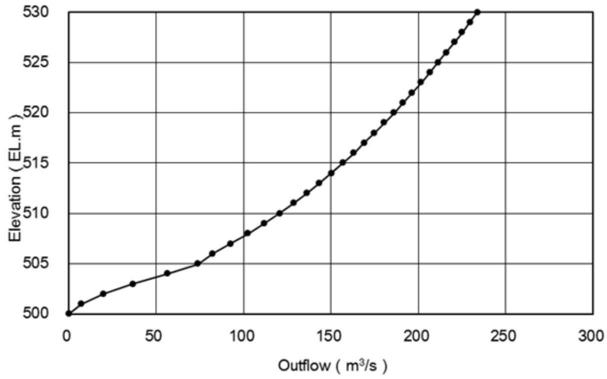
Where, a = 0.408, b = 0.311

Outflow characteristics of each proposed dam is shown below. It is needed to confirm outflow characteristics by hydraulic model test.

Ciawi Dam 1



Ciawi Dam 2



Cisukabirus Dam

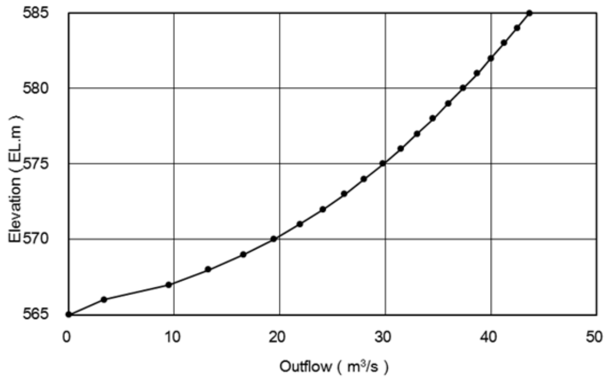


Figure 6.3-21 Outflow Characteristics of Proposed Dams

(4) Flood Control Effect by Proposed Dams

1) Flood Control Effect at Flood Control Points

Flood control effects at dam locations are estimated by difference of peak inflow discharge and maximum outflow discharge. For the Plan B which is the combination of Ciawi Dam-2 at upstream of confluence and Cisukabirus Dam, flood control effect is estimated by difference of inflow discharge and maximum outflow discharge which is estimated based on combined hydrographs. Besides, flood control effects at Katulampa and Manggarai are also estimated by the following procedure.

- Flood control effect is estimated by flood analysis with three different hydrographs applied.
- Relation of flood control effect at dam site and control points are plotted on the graph.
- Flood control effect at control points are estimated read by the graph.

The results are summarized in the table below.

Table 6.3-13 Flood Control Effect of Proposed Plans

Dam site	Height	Dam Flood control effect	Katulampa Flood control effect	Manggarai Flood control effect
Ciawi Dam 1 (downstream site)	H=40m	115m^3/s (425m^3/s→310m^3/s)	135m^3/s (645m^3/s→510m^3/s)	95m^3/s (720m^3/s→625m^3/s)
Ciawi Dam 2 (upstream site) + Cisukabirus Dam	H=40m H=30m	150m^3/s (415m^3/s→265m^3/s) 140m^3/s (365m^3/s→225m^3/s) 20m^3/s (60m^3/s→40m^3/s)	170m^3/s (645m^3/s→475m^3/s)	130m^3/s (720m^3/s→590m^3/s)

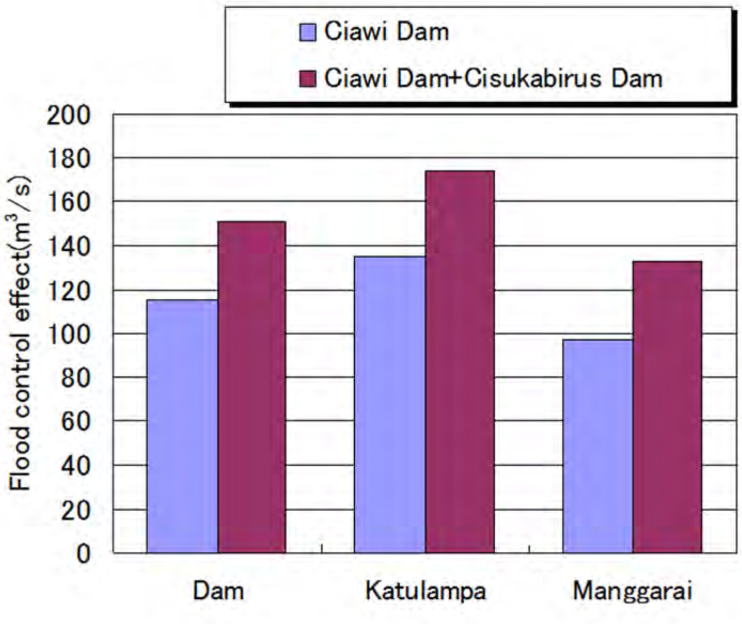


Figure 6.3-22 Comparison of Flood Control Effects

Table 6.3-14 Construction Costs and Flood Control Effects of Proposed Plans

<table>
<tr><th rowspan="2">Item</th><th rowspan="2">Unit</th><th colspan="3">2013 JCFM</th></tr>
<tr><th>(A) Ciawi Dam -1</th><th colspan="2">(B) Ciawi Dam -2 & Cisukabirus Dam</th></tr>
<tr><td>Maximum Dam Height</td><td>m</td><td>40.0</td><td>40.0</td><td>30.0</td></tr>
<tr><td>Dam Volume</td><td>m^3</td><td>438,000</td><td>320,000</td><td>80,000</td></tr>
<tr><td>Gross Storage Volume</td><td>m^3</td><td>2.607×10^6</td><td>3.850×10^6</td><td>0.420×10^6</td></tr>
<tr><td>Flood Control Effect of Manggarai Point</td><td>m^3/s</td><td>95</td><td colspan="2">130</td></tr>
<tr><td>Flood Control Effect of Katulampa Point</td><td>m^3/s</td><td>135</td><td colspan="2">170</td></tr>
<tr><td rowspan="2">Flood Control Effect of Dam Point</td><td rowspan="2">m^3/s</td><td rowspan="2">115</td><td colspan="2">150</td></tr>
<tr><td>140</td><td>20</td></tr>
<tr><td>Project Cost</td><td rowspan="3">Million Rp</td><td>2,453,000</td><td colspan="2">2,291,000</td></tr>
<tr><td>Construction cost (Dam)</td><td>1,533,000</td><td>1,120,000</td><td>281,000</td></tr>
<tr><td>Land acquisition</td><td>920,000 (36.8 ha)</td><td>737,500 (29.5 ha)</td><td>152,500 (6.1 ha)</td></tr>
<tr><td>Evaluation</td><td></td><td>★ ★</td><td colspan="2">★ ★ ★</td></tr>
</table>

[Land Acquisition Cost : 25,000 mil.Rp/ha]

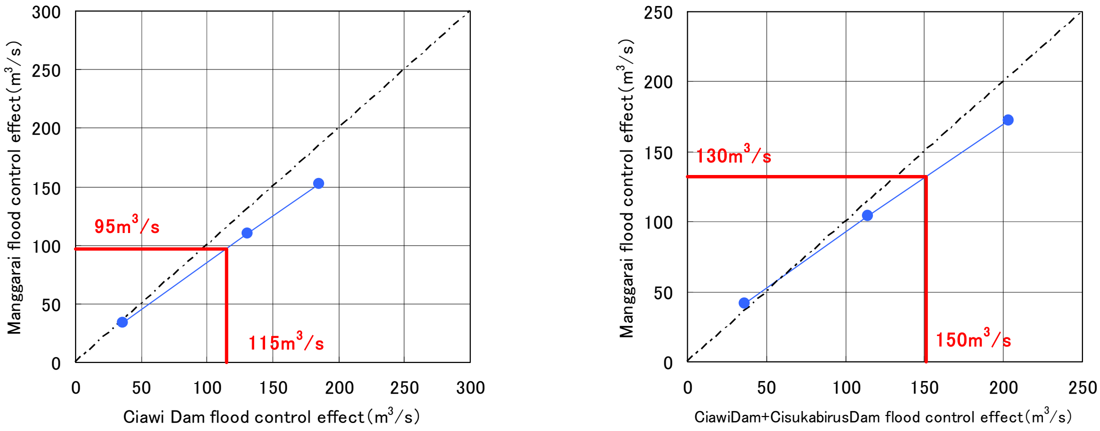


Figure 6.3-23 Relation of Flood Control Effects at Dam Site and Manggarai

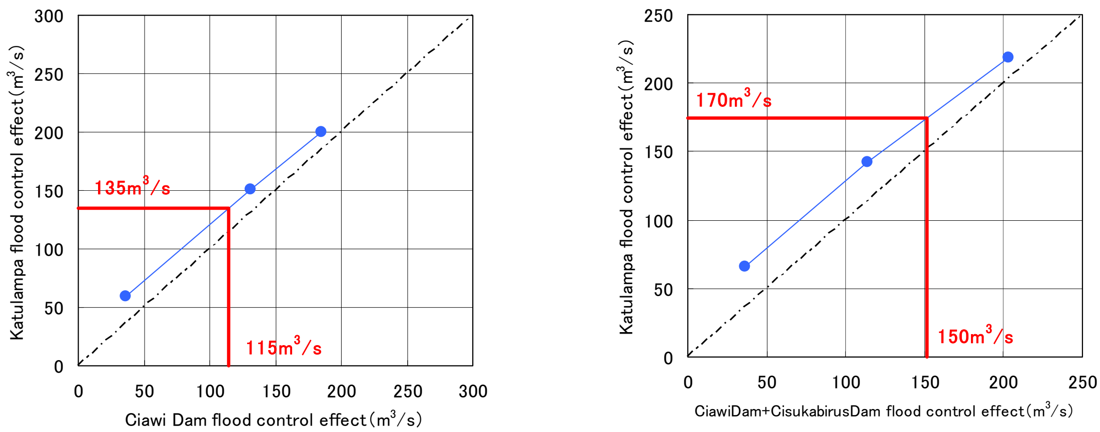


Figure 6.3-24 Relation of Flood Control Effects at Dam Site and Katulampa

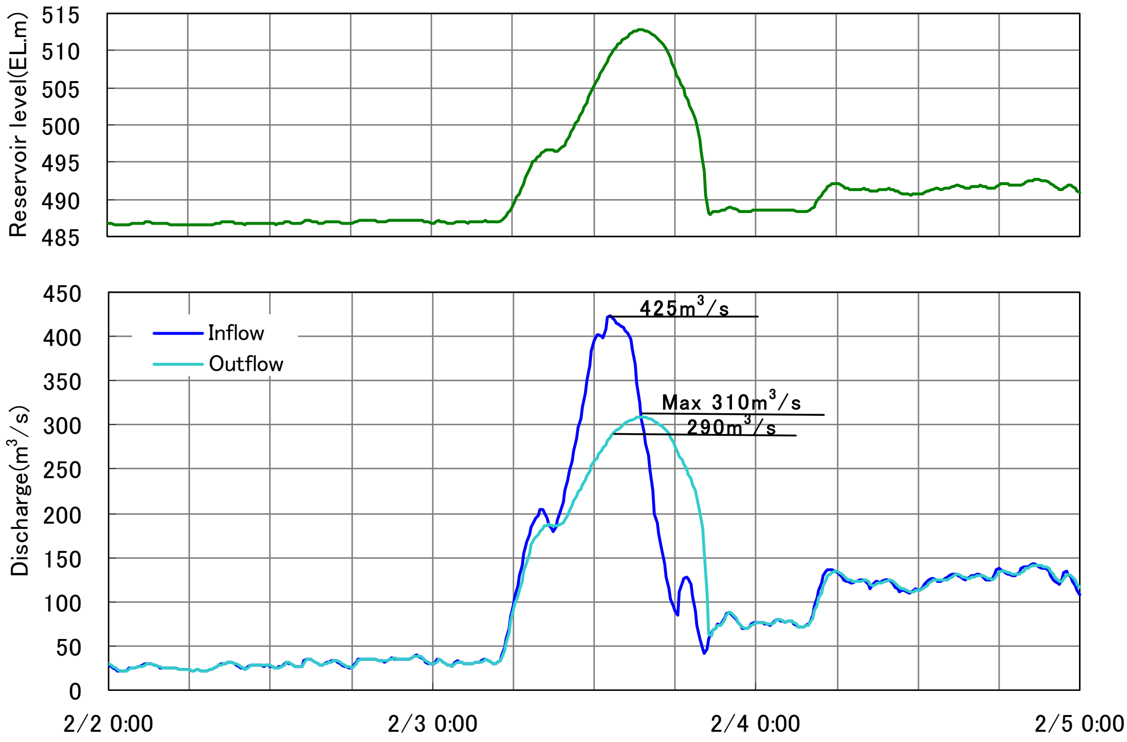


Figure 6.3-25 Hydrograph of Ciawi Dam-1

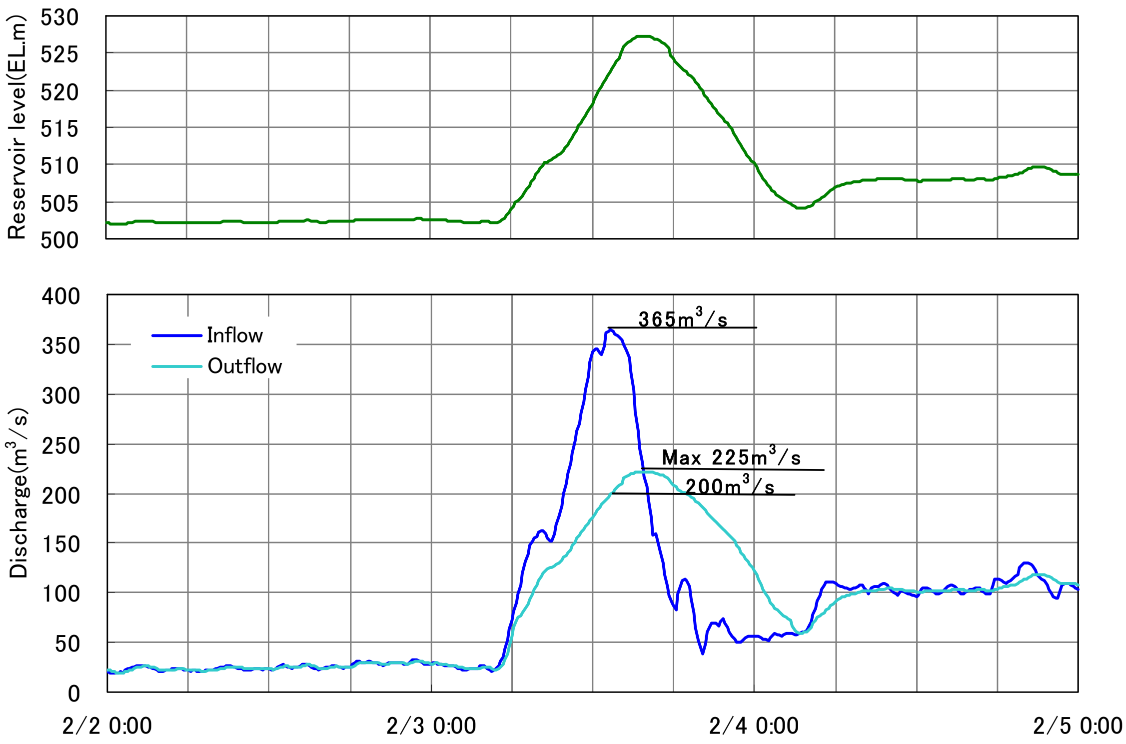


Figure 6.3-26 Hydrograph of Ciawi Dam-2

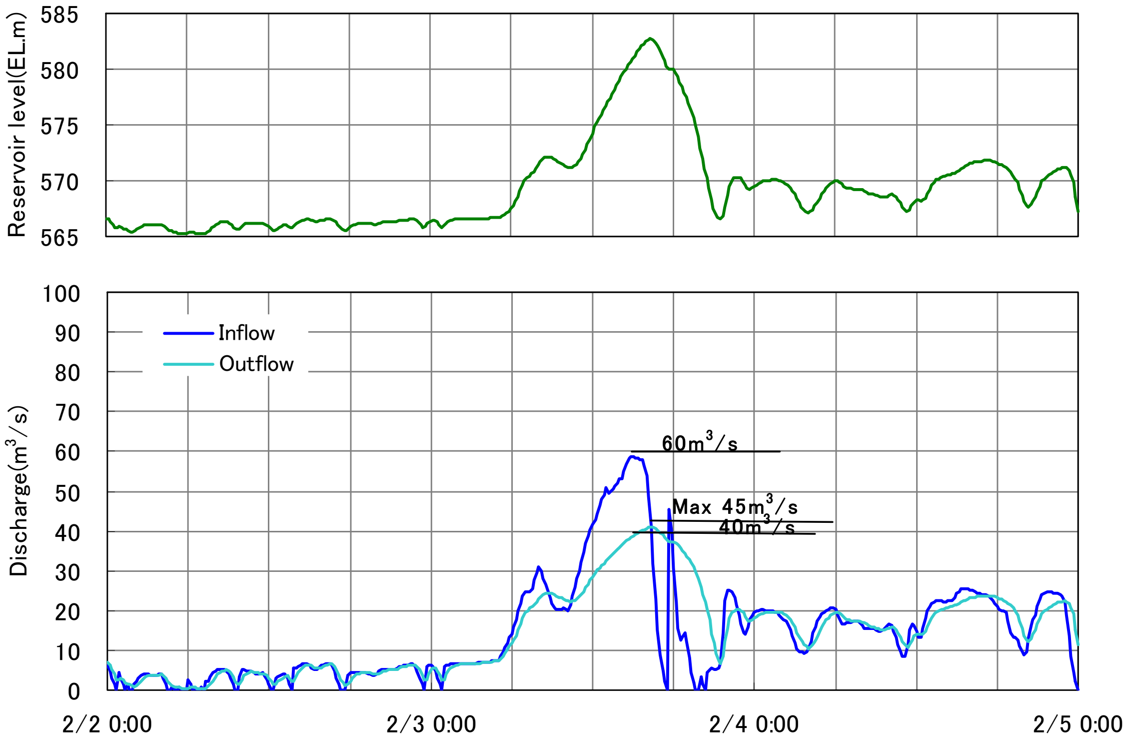


Figure 6.3-27 Hydrograph of Cisukabirus Dam

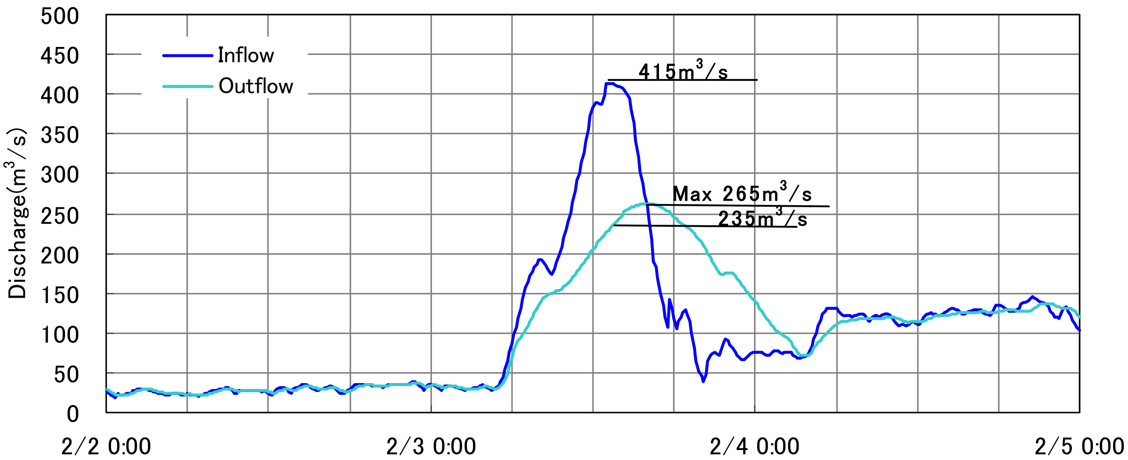


Figure 6.3-28 Hydrograph of Ciawi Dam-2 + Cisukabirus Dam

2) Difference of Peak Times between Ciawi Dam-2 and Cisukabirus Dam

Flood control effects of Ciawi Dam-2 and Cisukabirus Dam are 140m^3/s and 20m^3/s, respectively. However, combined flood control effect is estimated at 150m^3/s which is smaller than the numerical sum of each flood control effect.

Peak times of inflows of Ciawi Dam-2 and Cisukabirus Dam are different by 100 minutes while peak times of outflows are different by 30 minutes. Difference of peak time makes smaller peak discharge at the confluence than numerical sum of peak discharges of tributaries, and larger difference of peak time occurrence in tributaries, smaller peak discharge at the confluence. Thus, combined flood control effect is smaller than the numerical sum of each flood control effect because peak time occurrence at the outlet of both dams becomes closer.

Table 6.3-15 Flood Control Effect of Plan B

Dam site	Height	Dam Flood control effect	Katulampa Flood control effect	Manggarai Flood control effect
Ciawi Dam （upstream site） + Cisukabirus Dam	H=40m H=30m	150m^3/s (415m^3/s→265m^3/s) 140m^3/s (365m^3/s→225m^3/s) 20m^3/s (60m^3/s→40m^3/s)	170m^3/s (645m^3/s→475m^3/s)	130m^3/s (720m^3/s→590m^3/s)

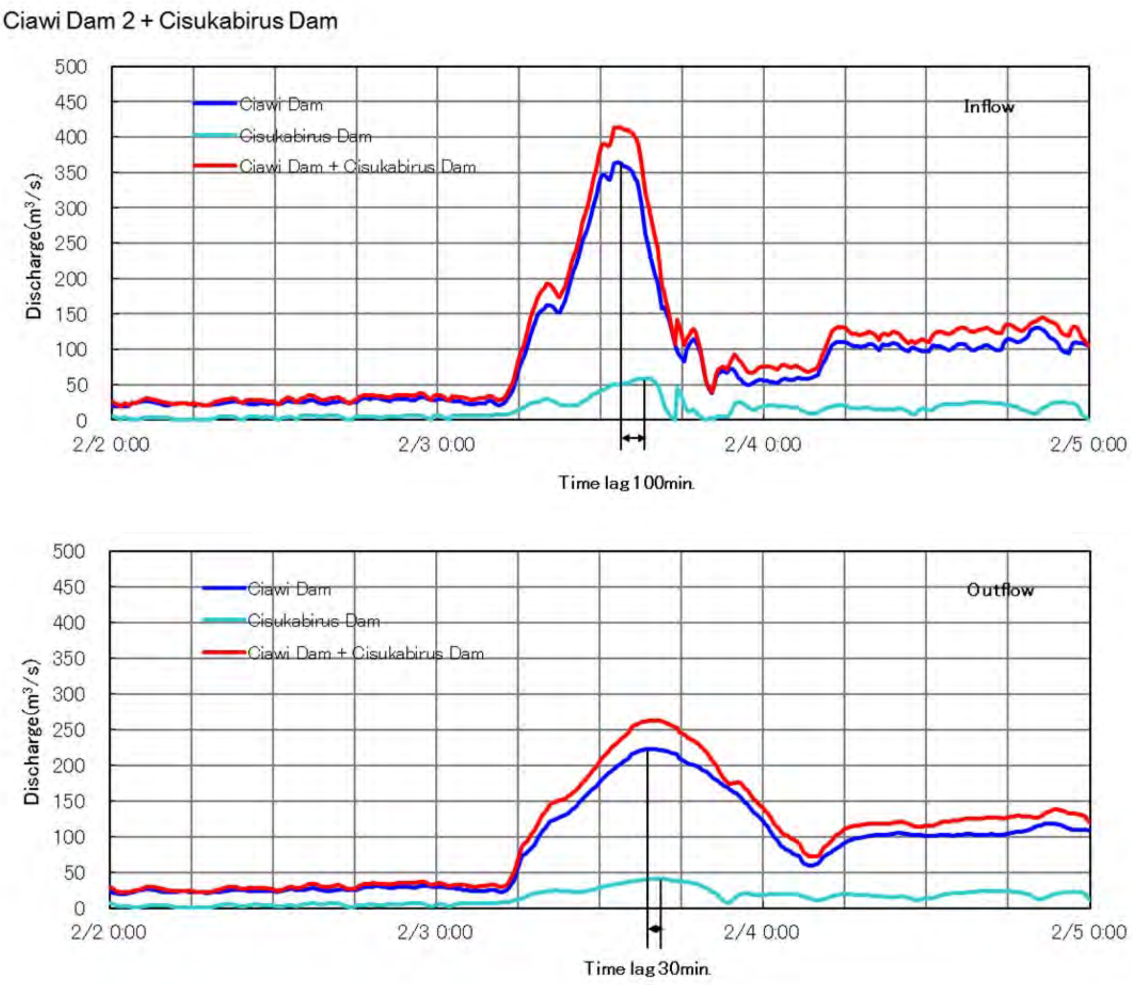


Figure 6.3-29 Peak Time of Inflow and Outflow Discharges

3) Dam Height and Flood Control Effect

Relation of dam height and flood control effect for each dam is examined. The design of service outlet shown in Table 6.3-10 are determined assuming dam height of 40m for Ciawi Dam-1 and 2 and 30m for Cisukabirus Dam. By changing dam height, flow section of outlet shall be changed to discharge necessary volume according to dam storage capacity. Smaller storage capacity of dam, larger outlet is required. For the calculation of flood control effect of dam other than the proposed height, square shape of conduit with 5m maximum length of edge is applied. If more area than 5m x 5m is needed, several conduits are applied. The results are summarized in Figure 6.3-30 to 6.3-34. For each dam, flood control effect becomes half if dam height becomes lower by 10m. Since the proposed dam heights are estimated ones based on past experiences. Dam heights shall be carefully examined based on detailed geotechnical investigations.

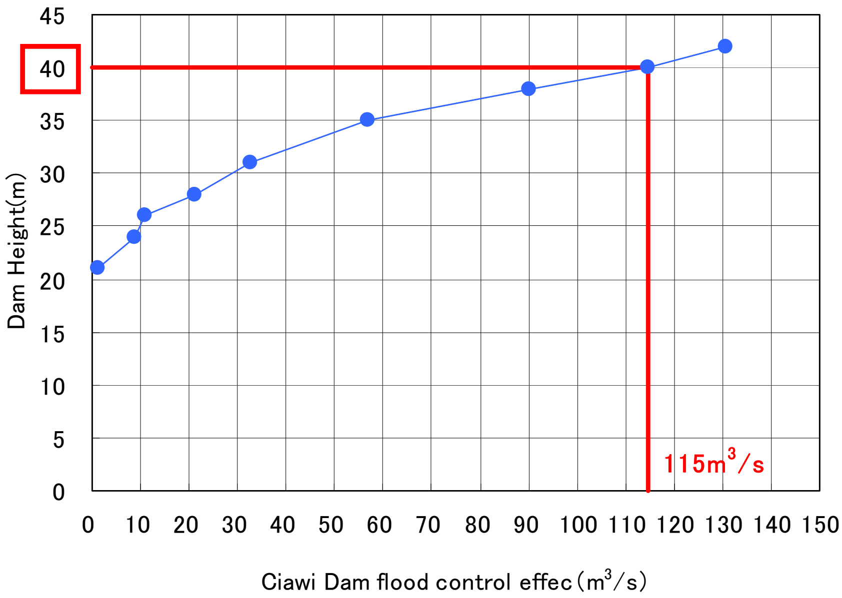


Figure 6.3-30 Dam Height and Flood Control Effect of Ciawi Dam-1 at Dam Site

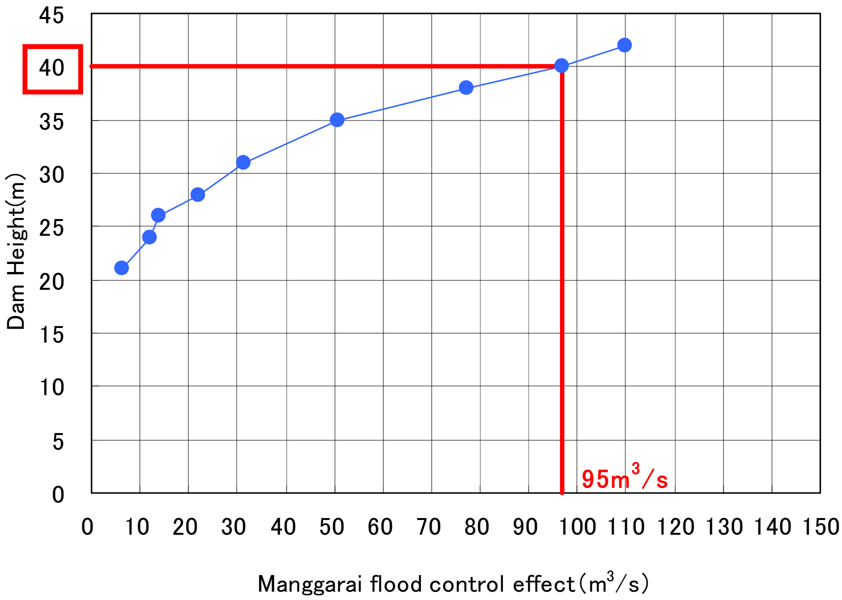


Figure 6.3-31 Dam Height and Flood Control Effect of Ciawi Dam-1 at Manggarai

Height of Dam (m)	Orifice Width (m)	Orifice Height (m)	Flood control Volume (m^3)	SWL (EL.m)	Max Outflow (m^3/s)	Effective discharge Dam (m^3/s)	Effective discharge Manggarai (m^3/s)
21	12.0	5.0	135,502	495.5	421.92	1.36	6.29
24	10.0	5.0	267,833	498.4	414.61	8.66	12.13
26	9.0	5.0	372,402	500.3	412.44	10.84	13.87
28	8.0	5.0	572,365	502.5	402.22	21.05	22.04
31	7.0	5.0	899,055	505.6	390.58	32.70	31.36
35	6.0	5.0	1,397,085	509.1	366.53	56.75	50.60
38	5.0	5.0	2,119,240	512.6	333.30	89.98	77.18
40	7.2	3.0	2,606,904	514.7	308.61	114.67	96.94
42	4.5	4.5	3,078,010	516.2	292.54	130.74	109.79

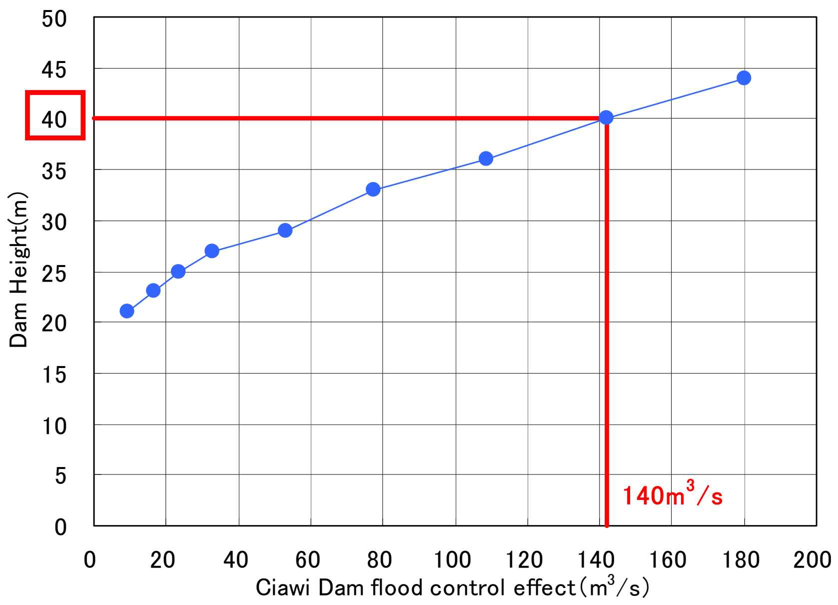


Figure 6.3-32 Dam Height and Flood Control Effect of Ciawi Dam-2 at Dam Site

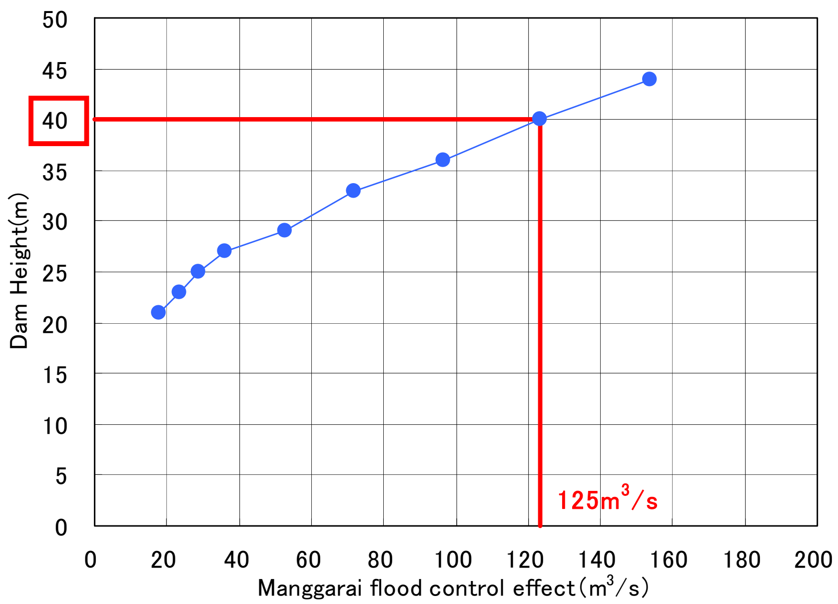


Figure 6.3-33 Dam Height and Flood Control Effect of Ciawi Dam-2 at Manggarai

Height of Dam (m)	Orifice		Flood control Volume (m^3)	SWL (EL.m)	Max Outflow (m^3/s)	Effective discharge	
	Width (m)	Height (m)				Dam (m^3/s)	Manggarai (m^3/s)
21	10.0	5.0	523,341	510.9	354.55	9.42	17.72
23	9.0	5.0	679,748	512.3	347.46	16.51	23.36
25	8.0	5.0	895,877	514.2	340.64	23.34	28.79
27	7.0	5.0	1,182,660	516.2	331.28	32.69	36.23
29	6.0	5.0	1,612,187	518.9	310.64	53.33	52.65
33	5.0	5.0	2,211,426	522.1	286.56	77.41	71.80
36	4.5	4.5	2,947,426	525.7	255.43	108.54	96.57
40	4.4	3.6	3,849,672	529.6	222.00	141.97	123.16
44	3.5	3.5	4,972,448	533.9	183.71	180.26	153.62

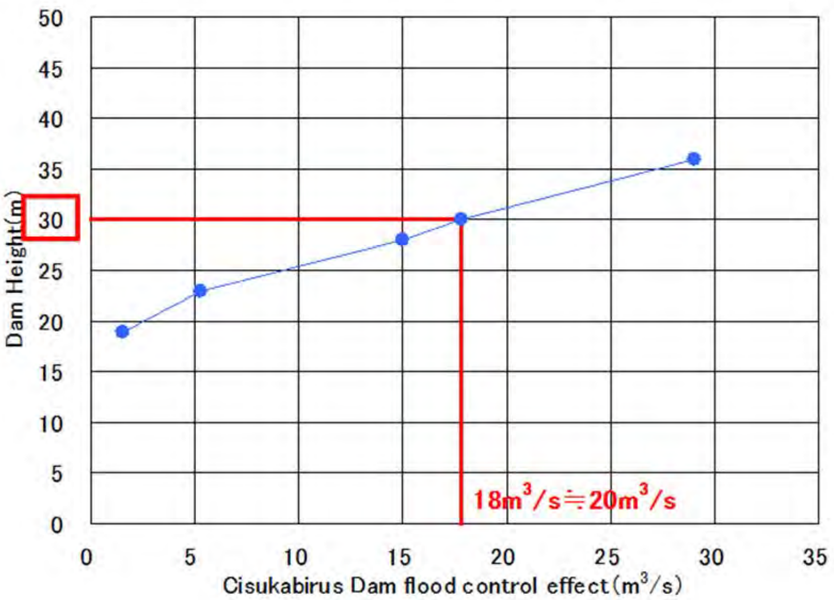


Figure 6.3-34 Dam Height and Flood Control Effect of Cisukabirus Dam at Manggarai

Height of Dam (m)	Orifice		Flood control Volume (m^3)	SWL (EL.m)	Max Outflow (m^3/s)	Effective discharge	
	Width (m)	Height (m)				Dam (m^3/s)	Manggarai (m^3/s)
15	5.0	5.0	27,432	569.5	58.70	0.04	0.02
15	4.5	4.5	29,426	569.9	58.69	0.04	0.03
16	4.0	4.0	31,826	570.1	58.68	0.05	0.03
16	3.5	3.5	38,142	570.5	58.04	0.70	0.43
19	3.0	3.0	77,584	573.2	57.16	1.58	0.97
23	2.5	2.5	158,684	577.0	53.45	5.29	3.26
28	2.0	2.0	358,988	582.9	43.75	14.99	9.23
30	2.1	1.7	420,046	584.4	40.91	17.82	10.97
36	1.5	1.5	719,034	590.2	29.73	29.01	17.85

4) Flood Analysis Results for Estimation of Flood Control Effects at Flood Control Points (Manggarai, KAtulampa)

Flood control effects at Katulampa and Manggarai are also estimated by the following procedure.

- Flood control effect is estimated by flood analysis with three different hydrographs applied.
- Relation of flood control effect at dam site and control points are plotted on the graph.
- Flood control effect at control points are estimated read by the graph.

The results of flood analysis are shown in Figure 6.3-35 to Figure 6.3-41.

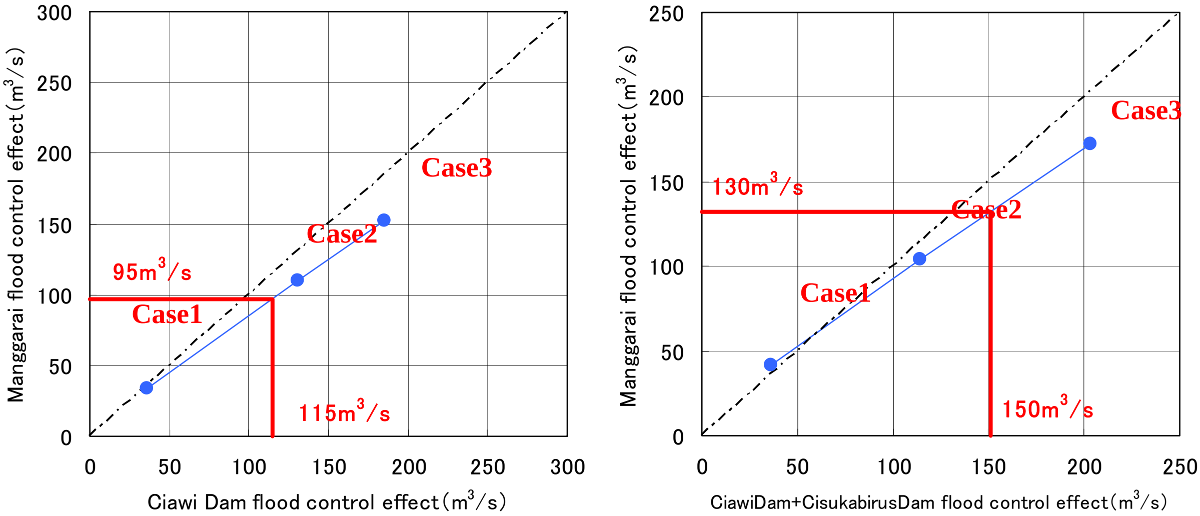


Figure 6.3-35 Relation of Flood Control Effects between Dam Sites and Flood Control Points

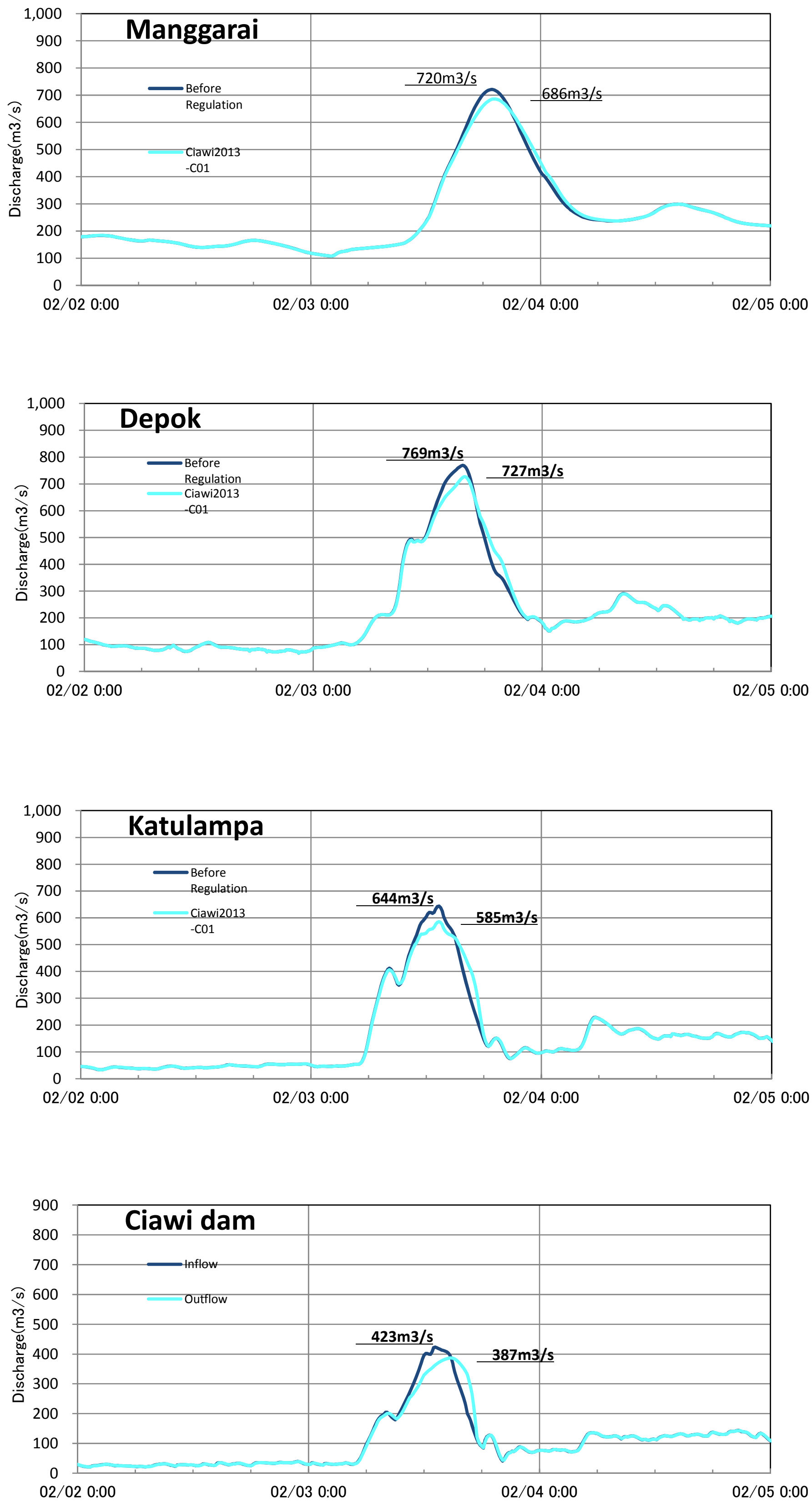


Figure 6.3-36 Ciawi Dam-1: Hydrographs of Flood Control Points in Case 1 Flood

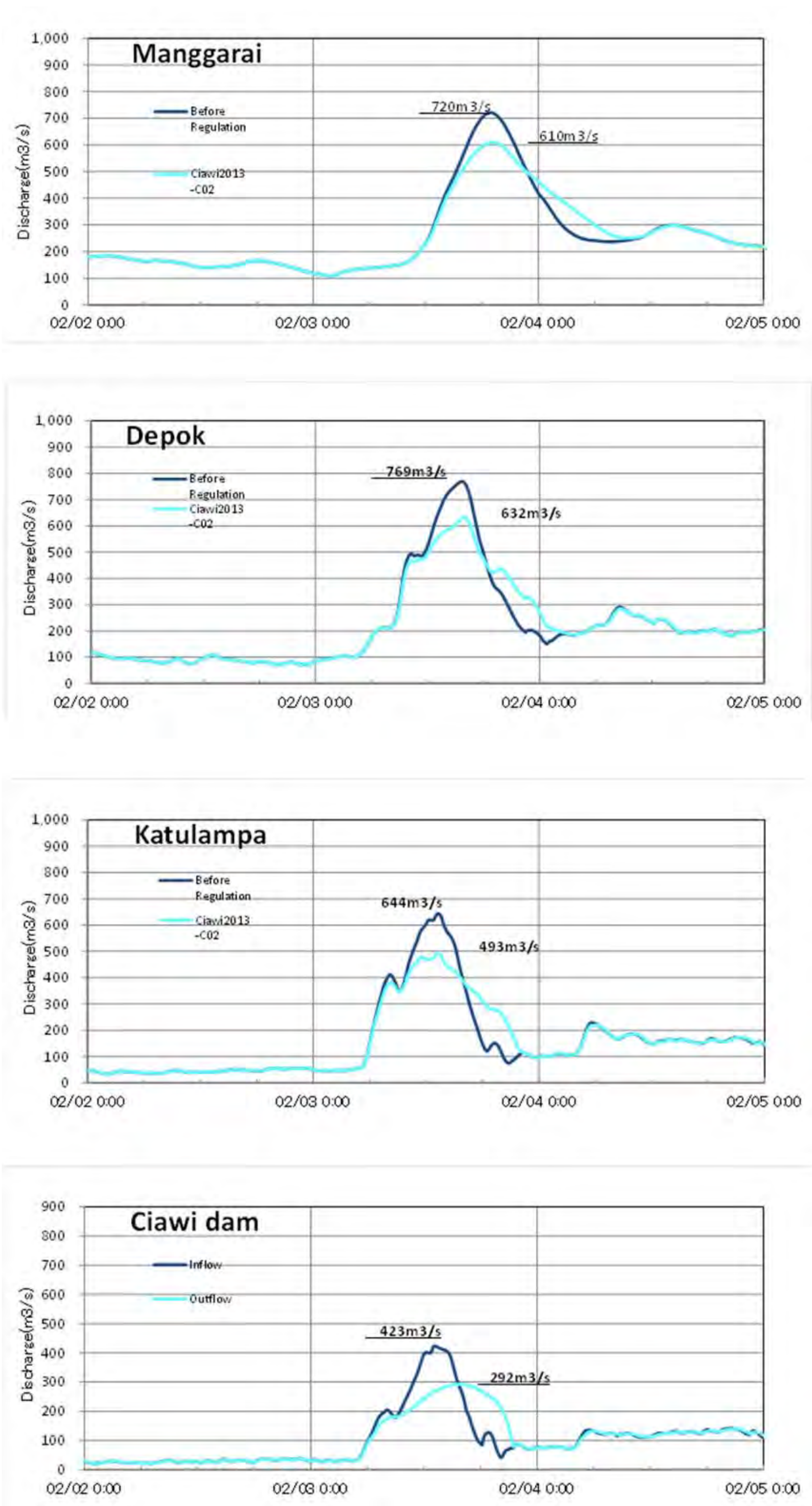


Figure 6.3-37 Ciawi Dam-1: Hydrographs of Flood Control Points in Case 2 Flood

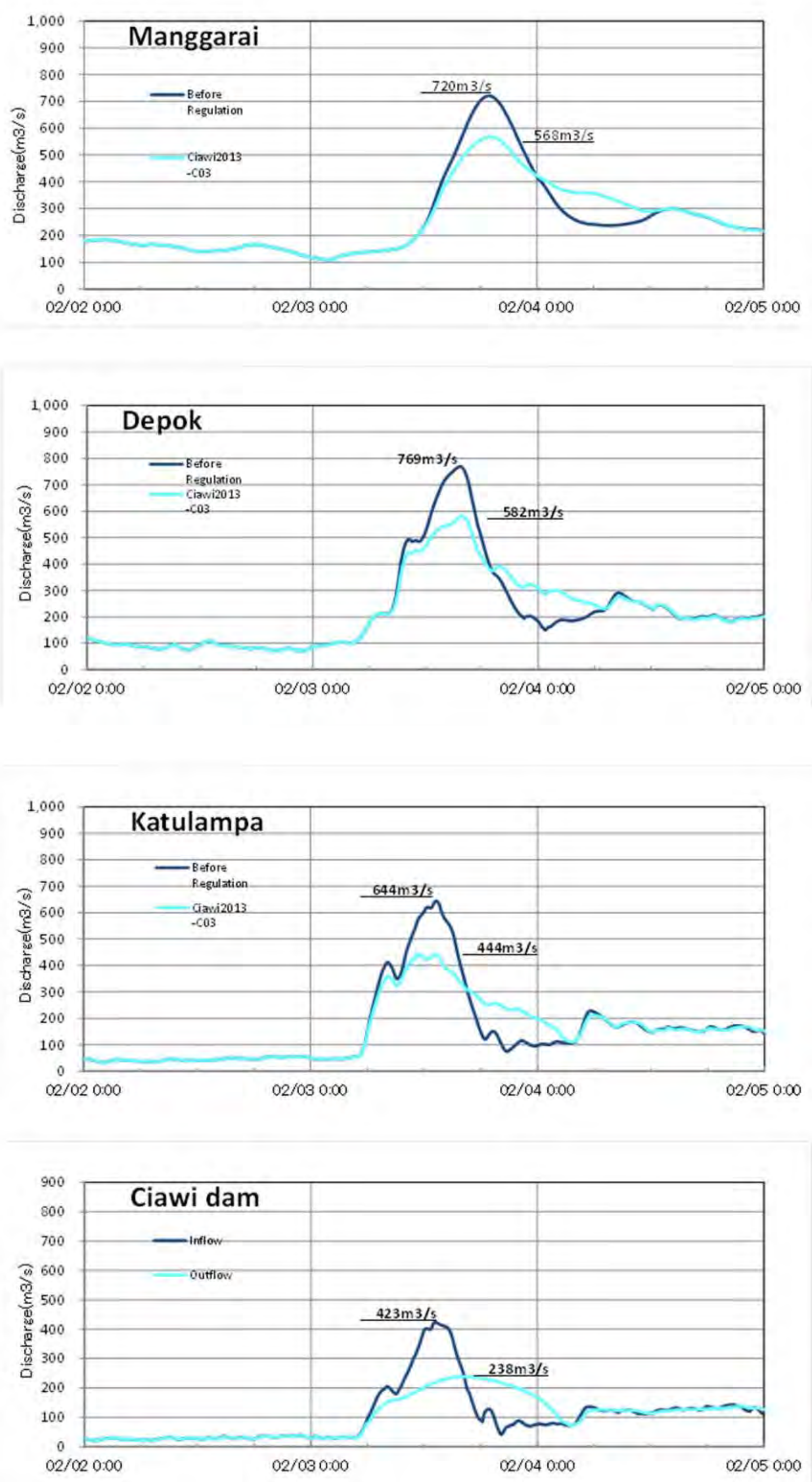
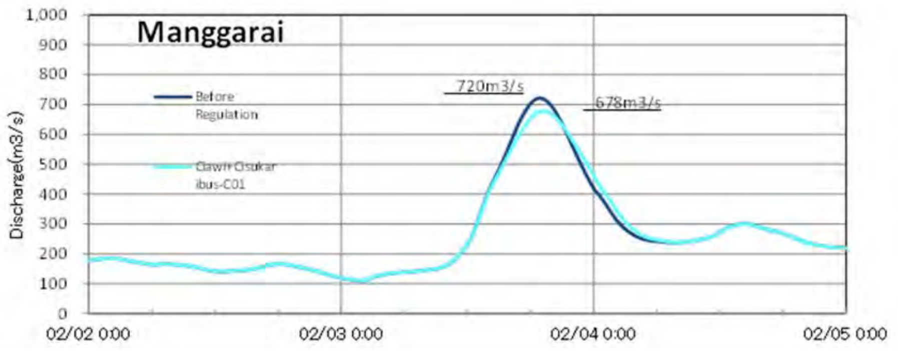
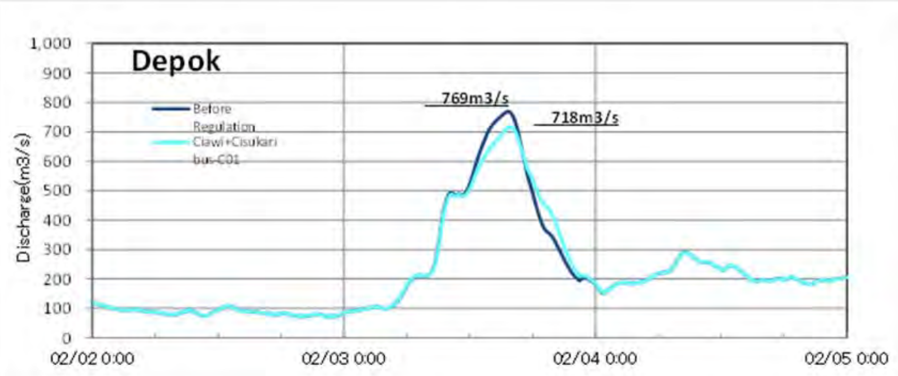
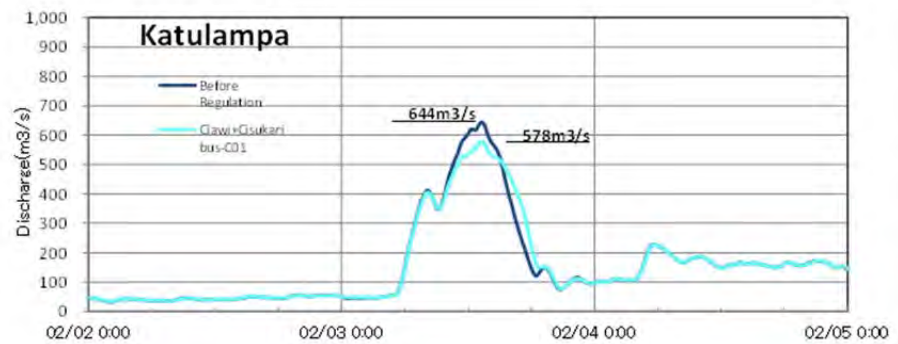
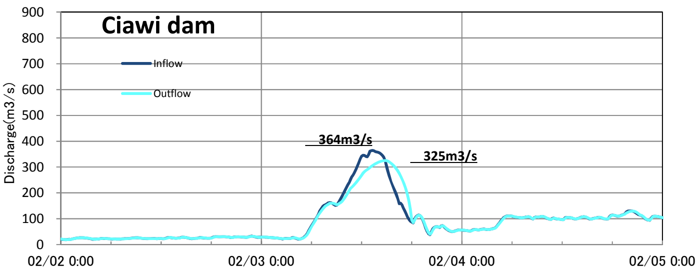


Figure 6.3-38 Ciawi Dam-1: Hydrographs of Flood Control Points in Case 3 Flood









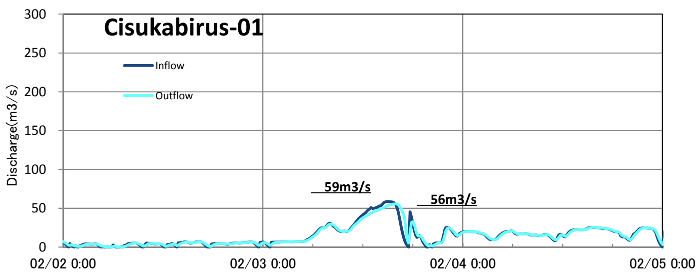


Figure 6.3-39 Ciawi Dam 2 + Cisukabirus Dam: Hydrographs of Flood Control Points in Case 1 Flood

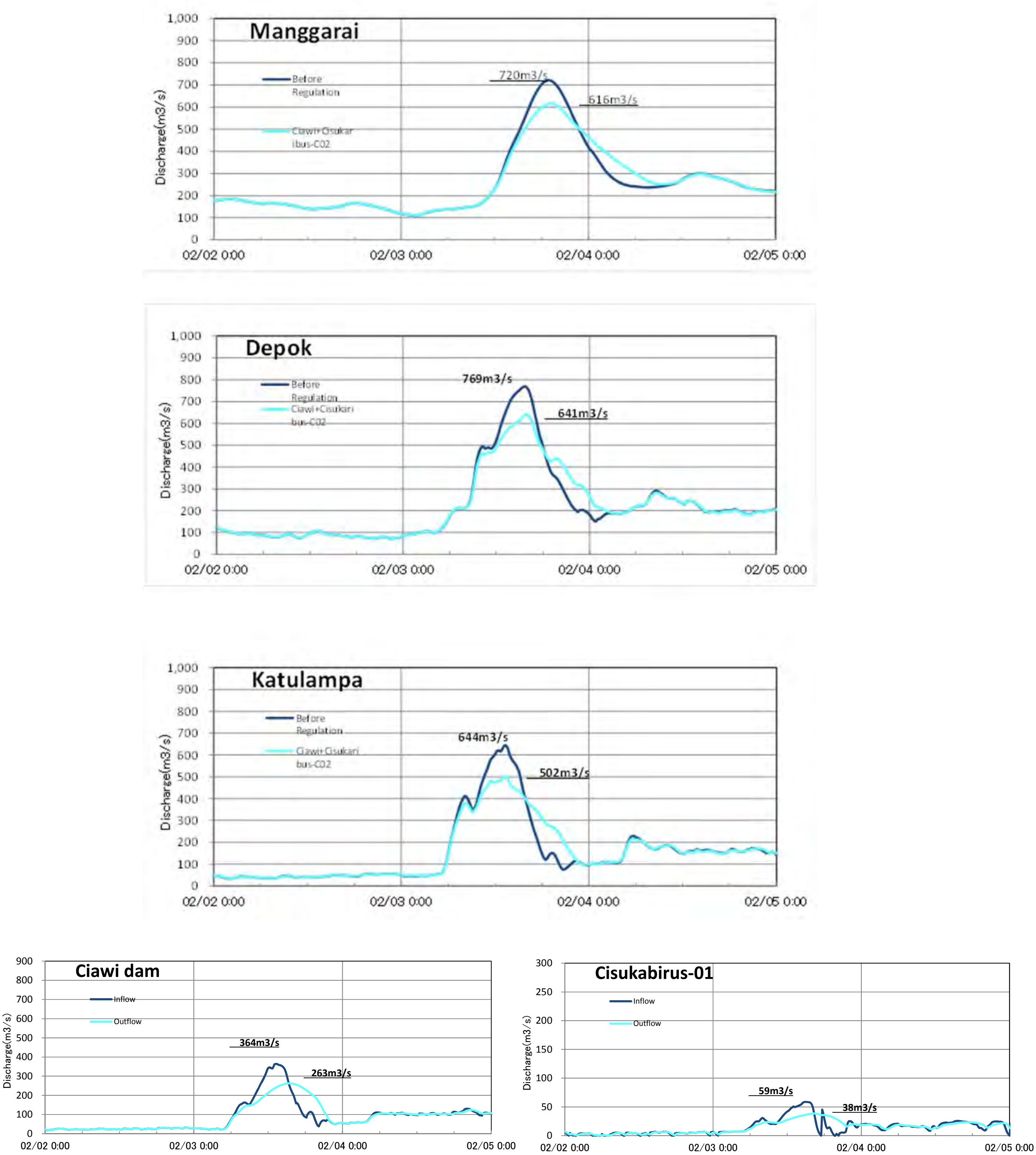
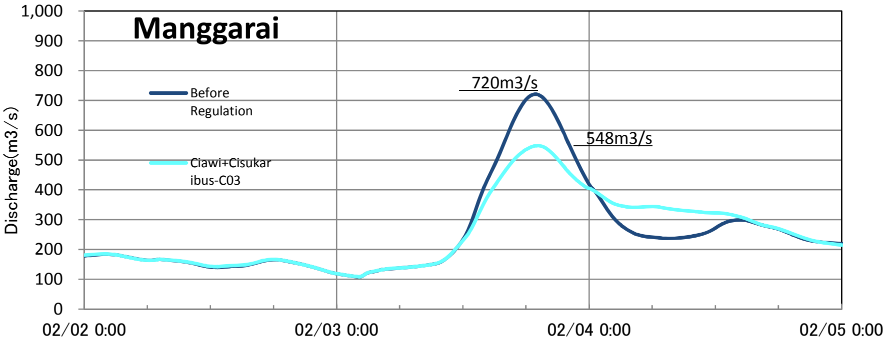
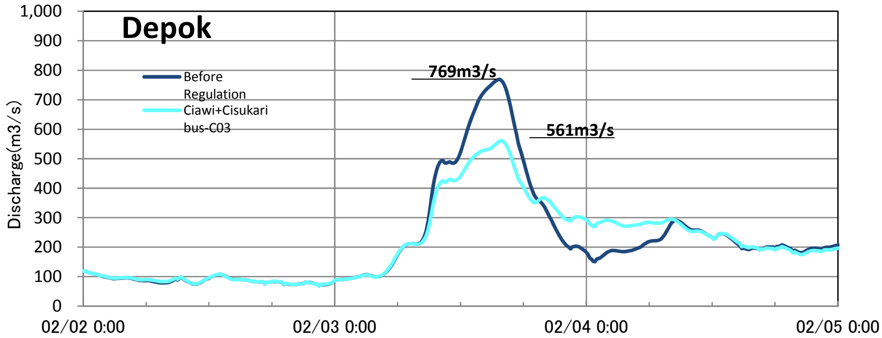
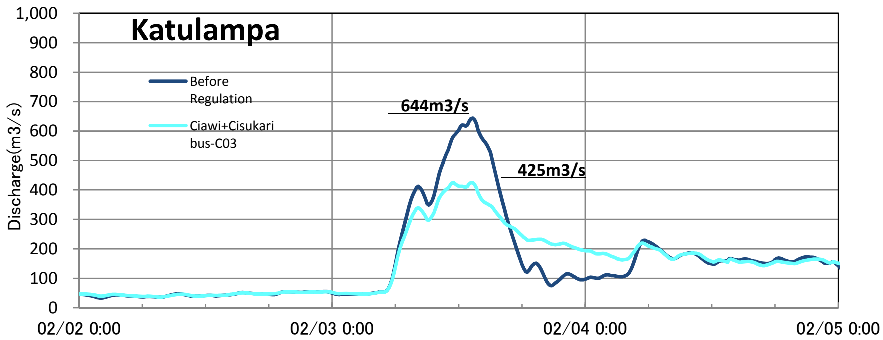
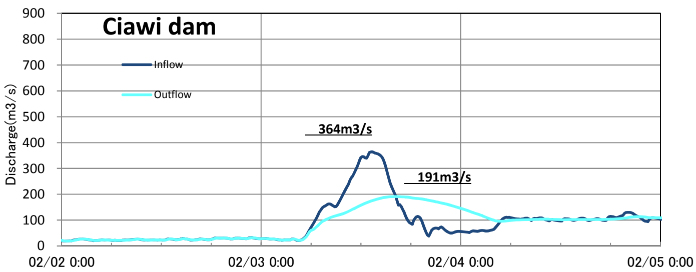


Figure 6.3-40 Ciawi Dam 2 + Cisukabirus Dam: Hydrographs of Flood Control Points in Case 2 Flood









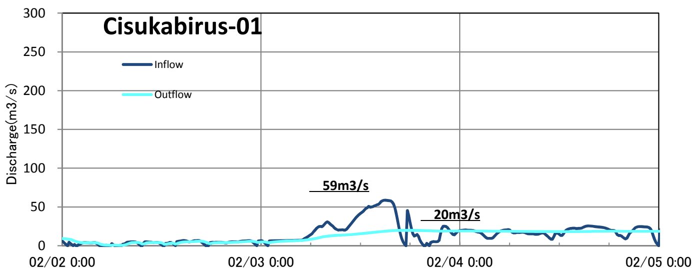


Figure 6.3-41 Ciawi Dam 2 + Cisukabirus Dam: Hydrographs of Flood Control Points in Case 3 Flood

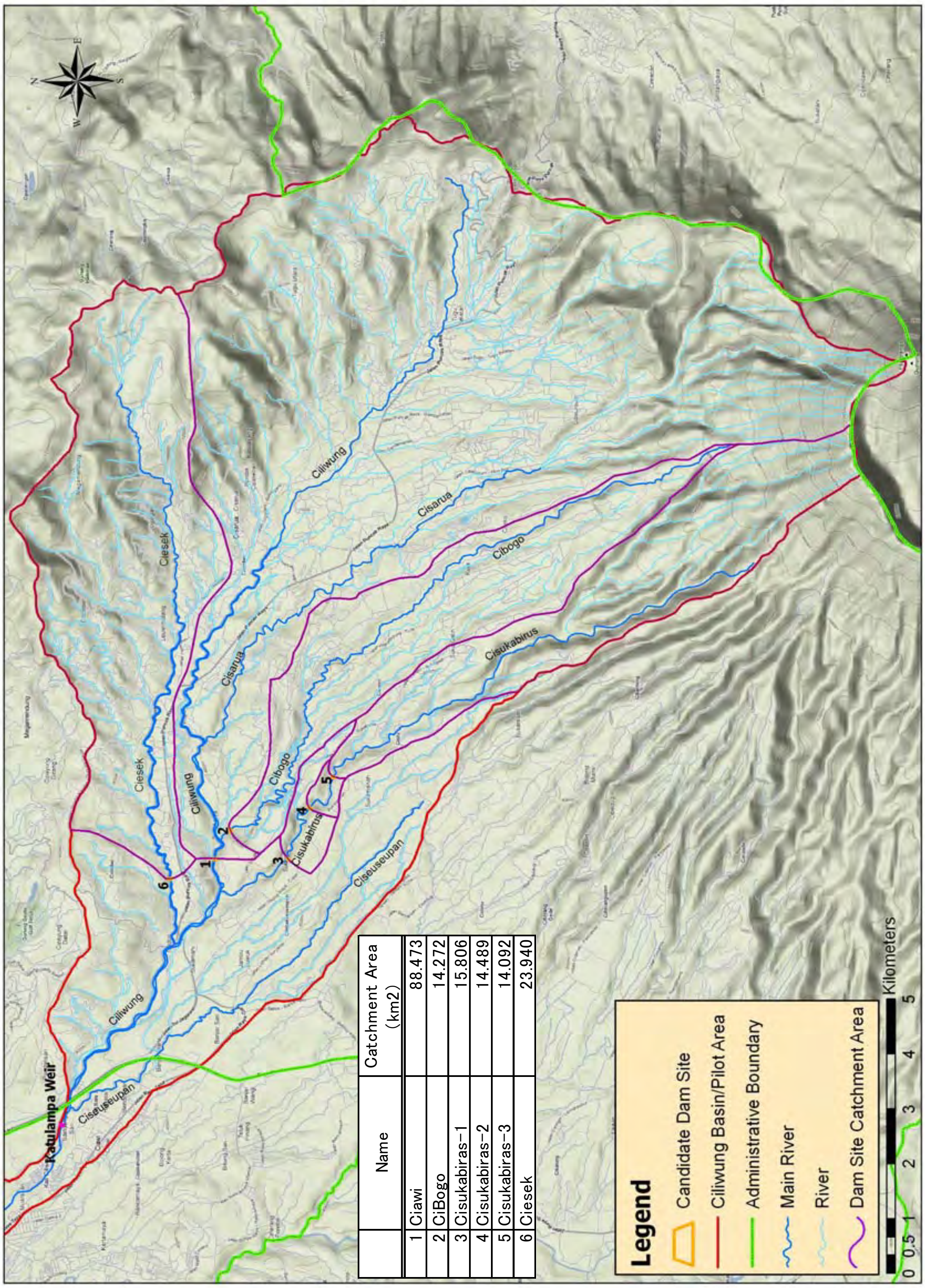
6.4 Small Dams

6.4.1 Candidate Dam Sites

Candidates six dam sites for small dams are studied as summarized in Table 6.4-1. The catchment areas and plans of reservoirs of six dams are shown in Figure 6.4-1 and Figure 6.4-2, respectively. H-V curves of six dams are estimated based on 1/10,000 scale topographic map as shown in Figure 6.4-3.

Table 6.4-1 Summary of Candidate Small Dams

Name of Site	Topography	Geology	Condition of Basement Rock	Estimated Dam Basement (EL.m)	Possible Height in terms of Topography Height/Elevation	High Water Level	Capacity (m^3)	Height/Elevation In case of Consider the Impact on Existing Houses and Description	Capacity In case of left column (m^3)
Ciliwung (Existing Site)	River terrace at both side. Relatively steep/	Pleistocene Quaternary Volcanic Sediments	Well consolidated	500	60m EL.560m	EL.555m	8,200,000	In case the dam is 60m in height, Around 50 houses etc. are influenced. 30m (25m) EL.525m(High Water Level)	1,000,000
Ciliwung-tr-1	Steep Slope Relatively gentle in the upper reach of reservoir	Holocene Quaternary Volcanic Sediments	Relatively well consolidated	530	40m EL.570m	EL.565m	600,000	Non	Non
Cisukabiras-1	Relatively Steep	Ditto	Ditto	560	60m EL.620m	EL.615m	3,300,000	There is one influenced house at the upper most part of reservoir 55m(50m) EL.610m (High Water Level)	2,600,000
Cisulabiras-2	Ditto Wide terrace (old river channel) area in the upper part of dam site.	Ditto	Ditto	605	55m EL.660m	EL.655m	2,600,000	There is 3 influenced houses. It is unavoidable influence for the house which is located immediate upper reach of dam site. 2 houses can be avoidable. 40m(35m) EL.640m (High Water Level)	1,000,000
Cisukabirasu-3	Relatively Steep. There is wide terrace in the upper part of reservoir	Ditto	Ditto	635	45m EL.680m	EL.675m	1,600,000	There is a private villa in the upper part of reservoir 30m(25m) EL.660m (High Water Level)	350,000
Ciesek	Steel The depth of valley is shallow.	Pleistocene Quaternary Volcanic Sediments	Well consolidated	490	35m EL.525m	EL.520m	500,000	In the middle area of reservoir, there are around 30 houses both river side/. 20m(15m) EL.505m (High Water Level)	50,000



	Name	Catchment Area (km2)
1	Ciawi	88.473
2	CiBogo	14.272
3	Cisukabiras-1	15.806
4	Cisukabiras-2	14.489
5	Cisukabiras-3	14.092
6	Ciesek	23.940

Figure 6.4-1 Locations and Catchment Areas of Small Dams

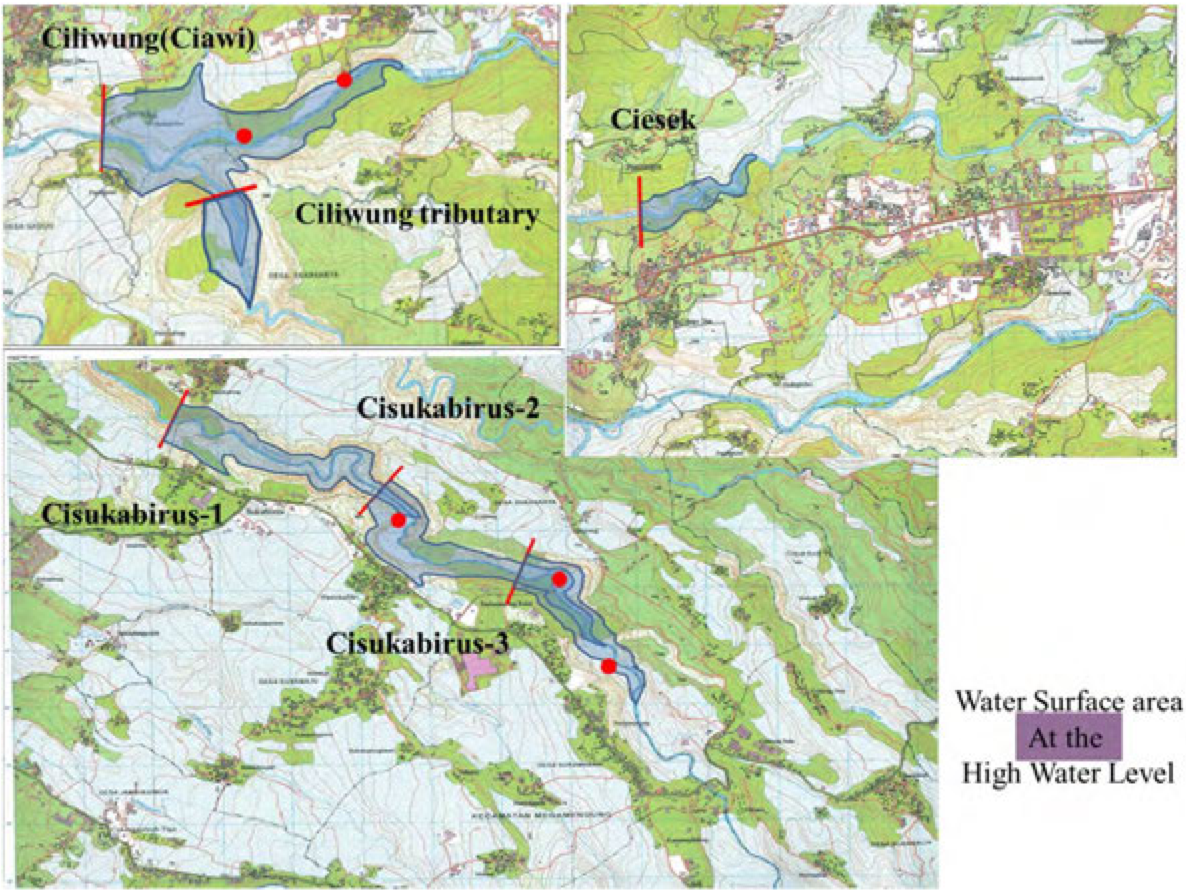
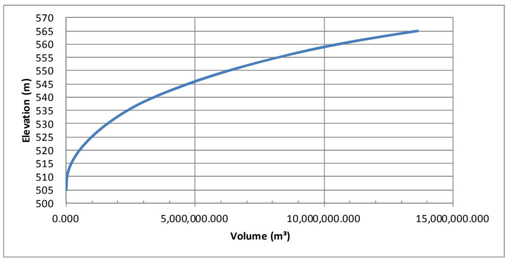


Figure 6.4-2 Plan of Reservoirs of Small Dams

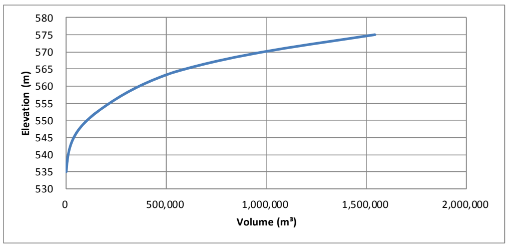
CILIWUNG 01
Dam Base: EL.500m

Elevation (EL.m)	Resorvior Area (m2)	Storage Volume (m3)
505	0	0
510	13,030	32,574
515	49,659	189,295
520	81,925	518,253
525	108,738	994,909
530	132,378	1,597,698
535	177,559	2,372,540
540	239,425	3,414,998
545	281,704	4,717,819
550	351,466	6,300,742
555	420,357	8,230,298
560	500,324	**10,531,999**
565	733,405	13,616,322



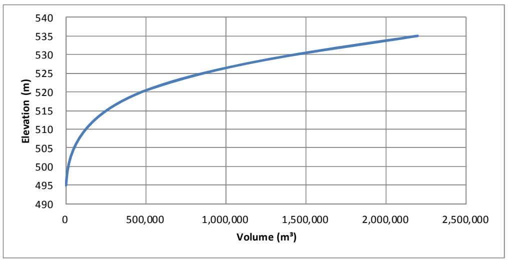
CILIWUNG 02/CiBogo
Dam Base: EL.530m

Elevation (EL.m)	Resorvior Area (m2)	Storage Volume (m3)
535	0	0
540	4,081	10,203
545	6,870	37,583
550	19,629	103,832
555	25,632	216,986
560	33,945	365,929
565	56,936	593,131
570	100,368	**986,391**
575	121,720	1,541,611



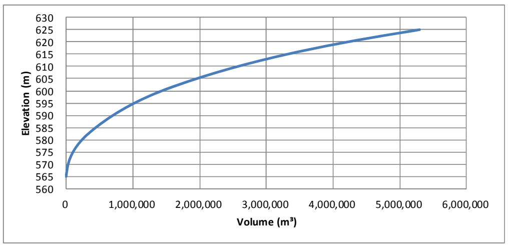
CIESEK
Dam Base: EL.490m

Elevation (EL.m)	Resorvior Area (m2)	Storage Volume (m3)
495	0	0
500	5,674	14,184
505	9,251	51,495
510	19,507	123,389
515	32,507	253,424
520	55,837	474,283
525	97,132	**856,705**
530	132,409	1,430,559
535	173,308	2,194,853



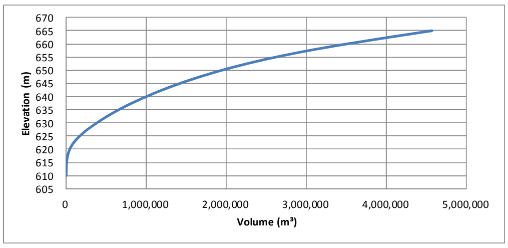
CISUKABIRUS 01
Dam Base: EL.560m

Elevation (EL.m)	Resorvior Area (m2)	Storage Volume (m3)
565	0	0
570	12,108	30,270
575	17,405	104,053
580	36,293	238,299
585	45,868	443,702
590	58,742	705,226
595	68,191	1,022,559
600	96,421	1,434,091
605	111,668	1,954,314
610	137,509	2,577,258
615	162,319	3,326,829
620	191,525	**4,211,441**
625	242,441	5,296,357



CISUKABIRUS 02
Dam Base: EL.600m

Elevation (EL.m)	Resorvior Area (m2)	Storage Volume (m3)
610	0	0
615	2,612	6,530
620	12,082	43,265
625	37,315	166,758
630	48,811	382,073
635	60,284	654,810
640	77,944	1,000,380
645	88,554	1,416,624
650	120,541	1,939,361
655	155,593	2,629,697
660	**192,163**	**3,499,089**
665	234,602	4,566,002



CISUKABIRUS 03
Dam Base: EL.635m

Elevation (EL.m)	Resorvior Area (m2)	Storage Volume (m3)
640	0	0
645	863	2,156
650	13,788	38,782
655	30,366	149,165
660	48,300	345,829
665	70,295	642,316
670	95,480	1,056,752
675	120,197	1,595,944
680	156,899	**2,288,685**
685	193,557	3,164,827
690	235,993	4,238,703
695	290,458	5,554,831

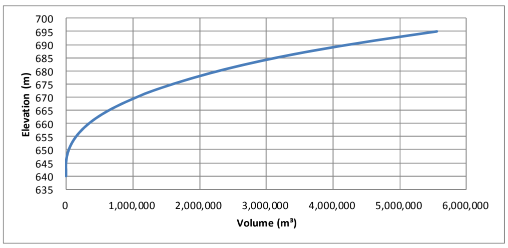


Figure 6.4-3 H-V Curves of Small Dams

6.4.2 Flood Control Effects of Small Dams

As mentioned above, six dam sites are selected as candidate small dam sites. Assuming the dam height of 20 from dam foundation, flood control effect is analyzed. As shown in Table 6.4-2, total storage capacity is estimated at $1{,}299 \times 10^3 m^3$.

Table 6.4-2 Proposed Small Dams with Height of 20m

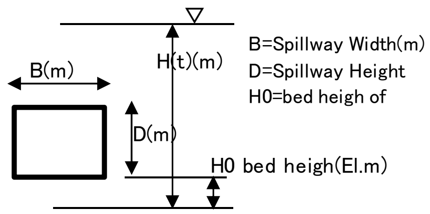
No	Major Reservoirs	Water Source	Height Reservoir (m)	Dike Lenght (m)	Reservoir Area (m2)	Storage Volume (m3)
1	Ciiwung (Existing Site)	S Ciliwung	20	185.0	81,925	518,253
2	Ciliwung-tr-1	S CiBogo	20	106.0	19,629	103,832
3	Cisukabiras-1	S Sukabirus	20	108.5	36,293	238,299
4	Cisulabiras-2	S Sukabirus	20	84.0	37,315	166,758
5	Cisukabirasu-3	S Sukabirus	20	105.0	30,366	149,165
6	Ciesek	S Ciesek	20	91.0	19,507	123,389
Total					**225,035**	**1,299,697**

(1) Conditions for Analysis of Flood Control Effect

Conditions of analysis are summarized as follows.

Table 6.4-3 Conditions for Analysis of Flood Control Effect of Small Dams

Item	Description
Flood Case	February 2007 Flood, W=1/50 years
Land Use	Future
Dam Operation	Outlet is always open.
Storage Capacity	①Flood Control Capacity Ciawi Dam（H=20m） $V=518{,}000m^3$ Cibogo Dam（H=20m） $V=104{,}000m^3$ Cisukabirus-1 Dam（H=20m） $V=238{,}000m^3$ Cisukabirus-2 Dam（H=20m） $V=166{,}000m^3$ Cisukabirus-3 Dam（H=20m） $V=149{,}000m^3$ Ciesek Dam（H=20m） $V=123{,}000m^3$ Total $V=1{,}299{,}000m^3$ ②Water Utilization Capacity There is no water utilization capacity assuming dry dam. ③Sediment Storage Capacity Although it shall be examined by hydraulic model test, there is no sediment storage capacity assuming dry dam.
H-V Curve	H-V curves are estimated based on 1/10,000 scale topographic map as shown in Figure 6.4-3
Flood Control Method	Natural control method by orifice
Design of Orifice	Orifice is designed by trial calculation to maximize flood control effect. The following discharge characteristics of orifice are applied. ・Free Flow ($H0+1.2D \geqq H(t)$) $h1=H(t)-H0$ $Qout=C1 \cdot B \cdot h1^{3/2}$ Where, C1=1.8 ・Orifice Flow ($H0+1.8D \leqq H(t)$) $h2=H(t)-(H0+D/2)$ $Qout=C2 \cdot B \cdot D\sqrt{(2gh2)}$ Where, C2=0.6 ・Transition Flow ($H0+1.2D < H(t) < H0+1.8D$) $Q1 = C1 \cdot B \cdot (1.2 \cdot D)^{3/2}$ $Q2 = C2 \cdot B \cdot D \cdot \sqrt{(2g1.3D)}$ $Qout=(Q2-Q1)/(0.6D)\{H(t)-(Ho+1.2D)\}+Q1$

Item	Description
	

(2) Results of Analysis

Flood control effects at each dam site and total flood control volume at the flood control points are summarized in Table 6.4-4 and Table 6.4-5, respectively. Flood discharge allocation and hydrographs at the flood control points and dam sites are shown in Figure 6.4-4, and Figure 6.4-5, respectively. Flood control effect at 37 m^3/s at Manggarai is expected.

Table 6.4-4 Flood Control Effects at Small Dam Sites

Dam	Crest			Peak Inflow (m^3/s)	Outflow at Peak Inflow (m^3/s)	Peak Cut (m^3/s)	Maximum Outflow (m^3/s)	Effect (m^3/s)	Maximum Reservoir Level (El.m)	Flood Control Volume ($1000m^3$)
	Width (m)	Height (m)	Number of Gate							
Ciawi	6.15	6.15	1	353.5	303.1	50.4	336.1	17.3	519.98	517
Cibogo	1.89	1.89	1	46.7	29.6	17.1	35.5	11.3	549.91	103
Cisukabiras 1	2.19	2.19	1	39.3	33.7	5.6	34.6	4.6	579.85	234
Cisukabiras 2	2.17	2.17	1	45.9	37.0	8.9	38.8	7.1	624.88	164
Cisukabiras 3	2.14	2.14	1	56.2	42.1	14.1	45.3	10.9	654.94	148
Ciesek	3.32	3.32	1	116.6	105.2	11.4	106.5	10.1	509.90	122

Table 6.4-5 Total Flood Control Effects of Small Dams at Flood Control Points

Point	Before Control (m^3/s)	After Control (m^3/s)	Effect (m^3/s)
Manggarai	720	683	37
Depok	769	723	46
Katulampa	644	585	59

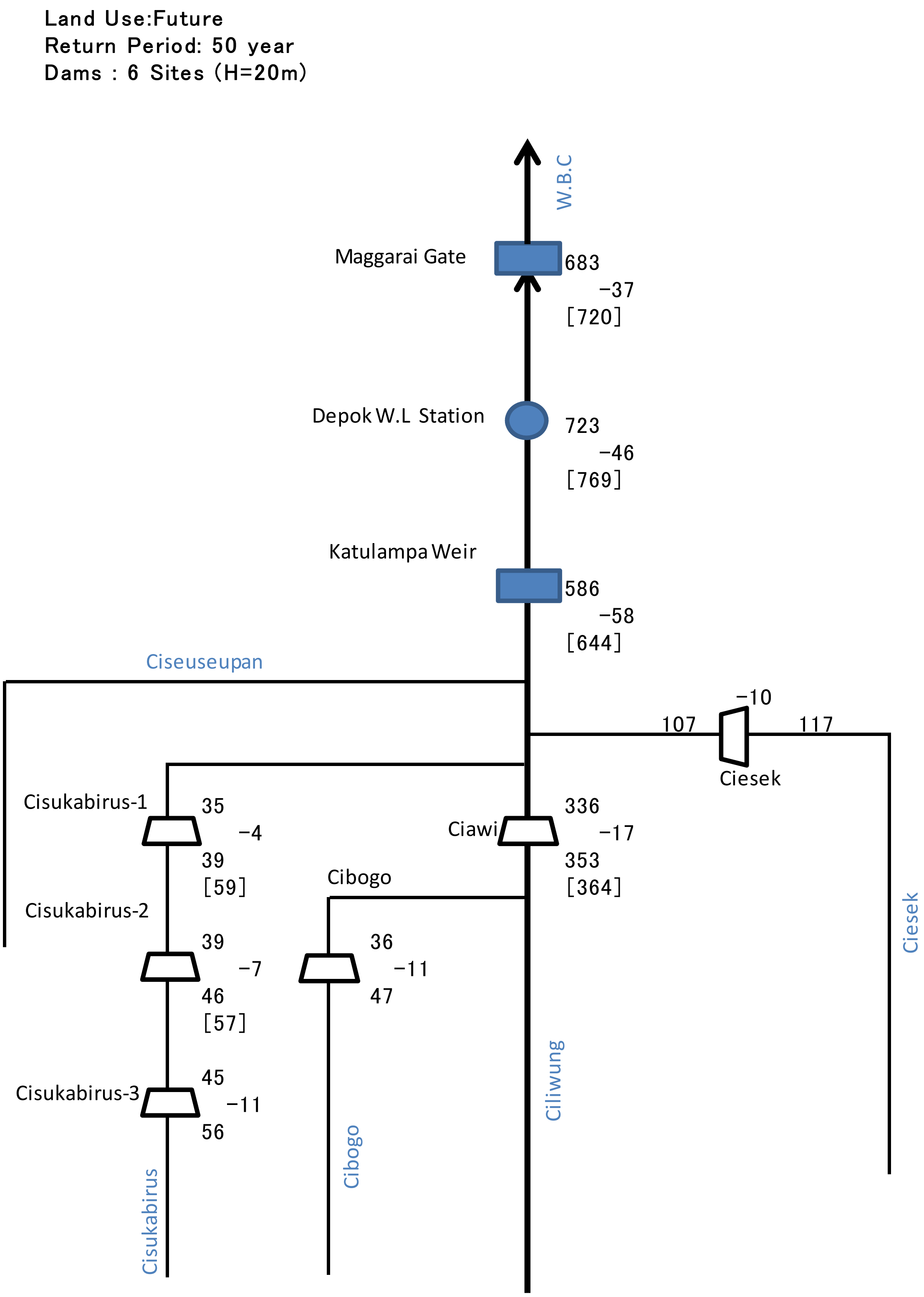


Figure 6.4-4 Flood Discharge Allocation

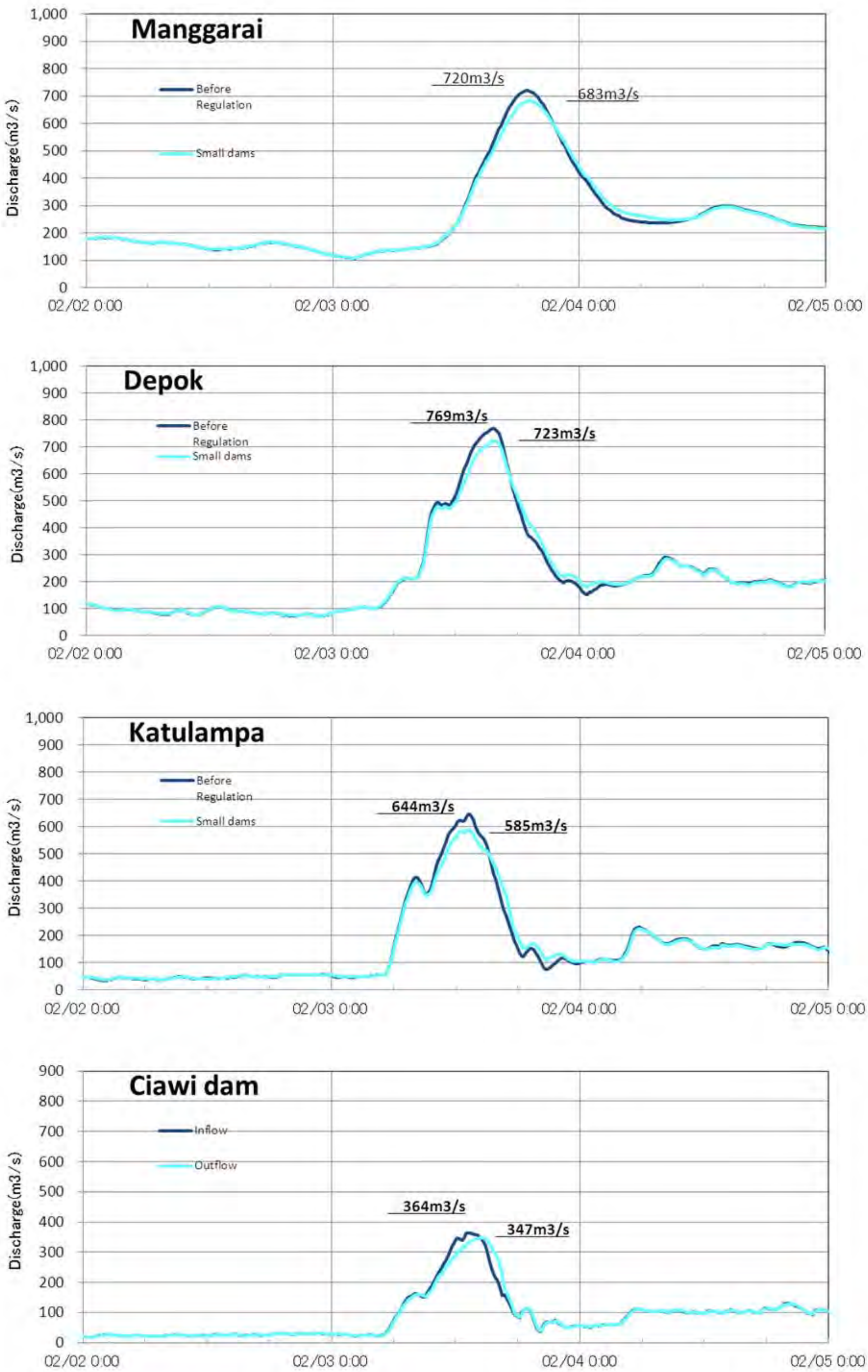


Figure 6.4-5 Hydrographs (1)

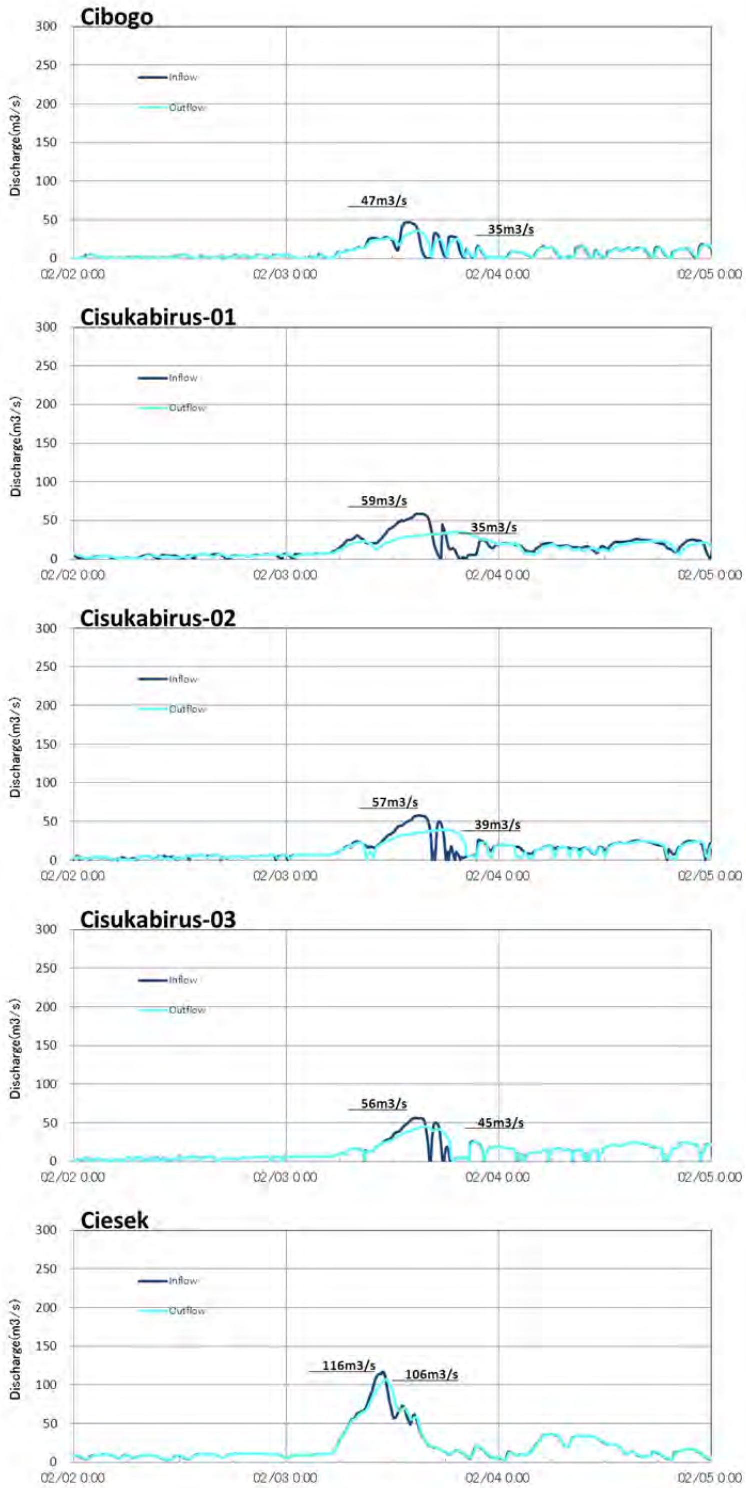


Figure 6.4-5 Hydrographs (2)

6.5 Gate Dams

6.5.1 Plan and Design of Gate Dams

Gate dam is to store flood discharge in river course by controlling of gate, and two gate dams at Pesona Kayangan and Bella Cassa near Depok are planned by BBWSCC. Plan and design of the gates dams are as follows.

Table 6.5-1 Specifications of Gate Dams

Gate Dam	Gate(Before Flood)			Gate(Flood Control)			Flood Control Volume (m^3)
	Width (m)	Height (m)	Number of Gate	Width (m)	Height (m)	Number of Gate	
Pesona Kayangn	6.00	10.00	5	6.00	3.55	5	163,000
Bella Cassa	6.00	10.00	5	6.00	3.66	5	303,000

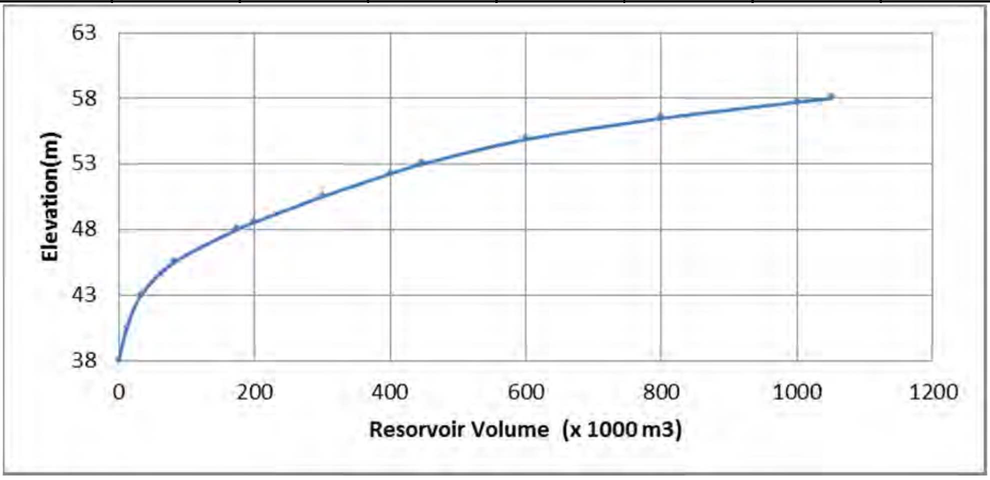


Figure 6.5-1 H-V Curve of Pesona Kayangn Gate Dam

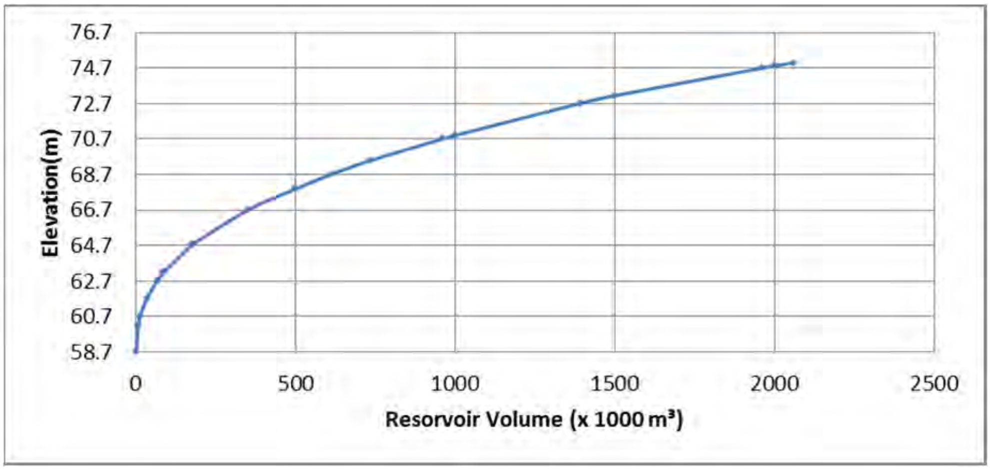


Figure 6.5-2 H-V Curve of Bella Cassa Gate Dam

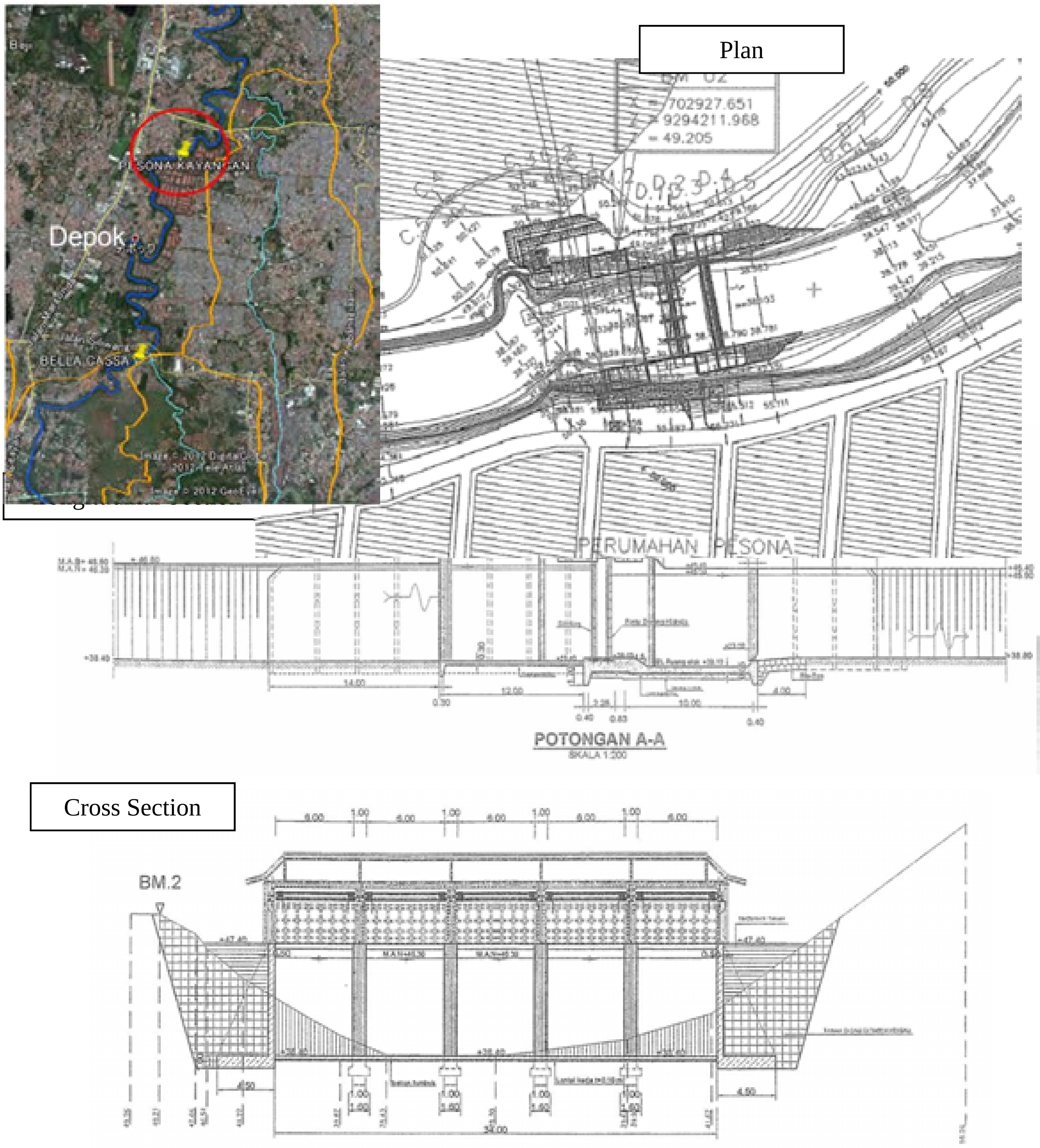


Figure 6.5-3 Design of Pesona Kayangan Gate Dam

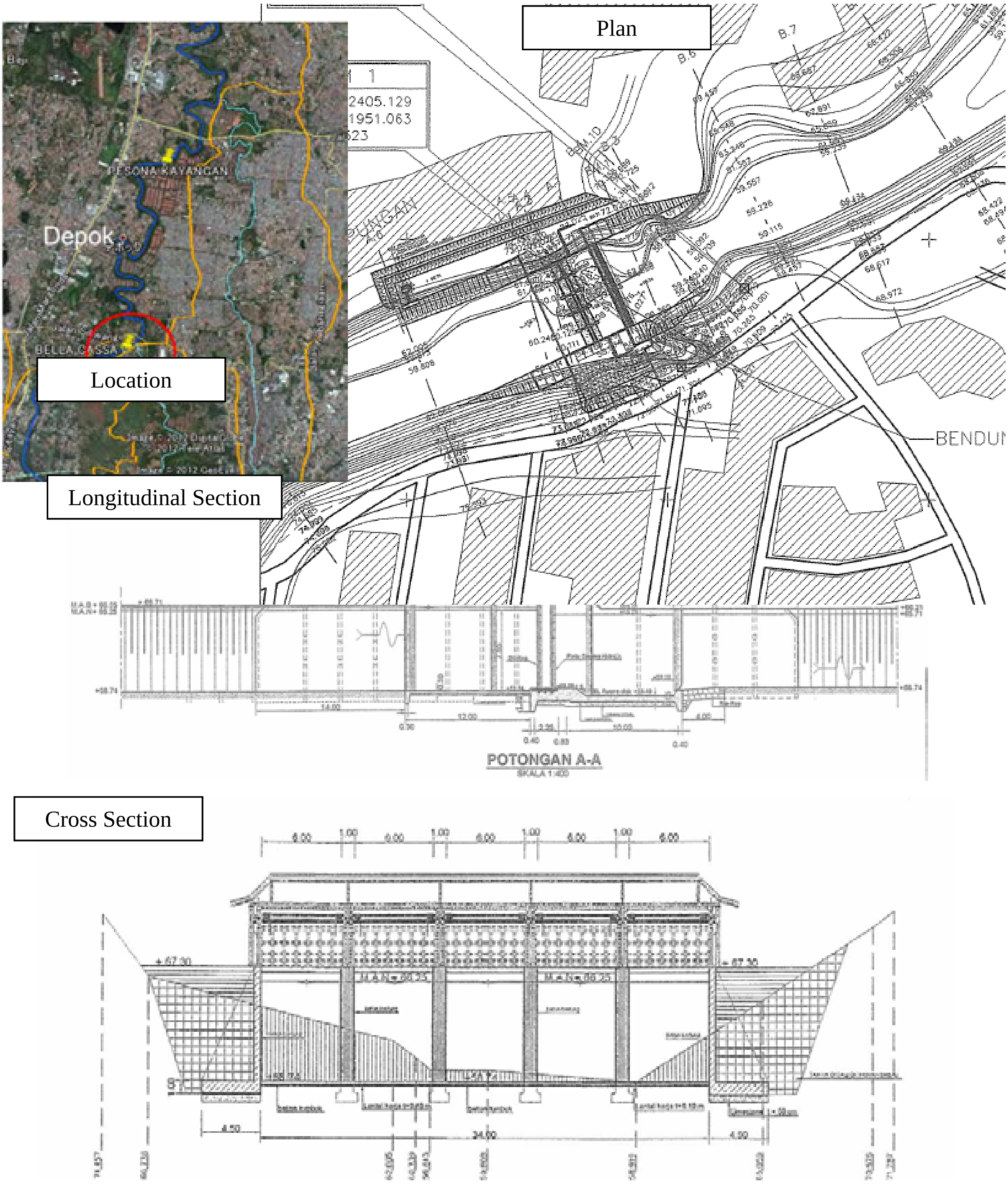


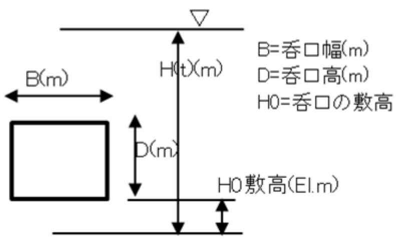
Figure 6.5-4 Design of Bella Cassa Gate Dam

6.5.2 Flood Control Effect of Gate Dams

(1) Conditions for Analysis of Flood Control Effect

Conditions of analysis are summarized as follows.

Table 6.5-2 Conditions for Analysis of Flood Control Effect of Gate Dams

Item	Description
Flood Case	February 2007 Flood, W=1/50 years
Land Use	Future
Flood Control Volume	・Pesona Kayangan V=173,000m^3 ・Bella Cassa V=306,000m^3
H-V Curve	Referring to “Detail Desain Bendung Gerak Sebai Long Storage di Sungai Ciliwung, 2010”
Flood Control Method	Natural control method by orifice
Specification of Gate	Pesona Kayangan: W 6m×H 10m×5 gates, Bottom Elevation 40.0m Bella Cassa: W 6m×H 10m×5 gates, Bottom Elevation 58.9m
Gate Operation	Open height is calculated to maximize flood control volume by trial calculation. The following discharge characteristics of orifice are applied. ・Free Flow ($H0+1.2D \geqq H(t)$) $h1=H(t)-H0$ $Qout=C1 \cdot B \cdot h1^{3/2}$ Where, C1=1.8 ・Orifice Flow ($H0+1.8D \leqq H(t)$) $h2=H(t)-(H0+D/2)$ $Qout=C2 \cdot B \cdot D\sqrt{(2gh2)}$ Where, C2=0.65※ (Note: 0.65 is applied referring to “Detail Desain Bendung Gerak Sebai Long Storage di Sungai Ciliwung, 2010”) ・Transition Flow ($H0+1.2D < H(t) < H0+1.8D$) $Q1 = C1 \cdot B \cdot (1.2 \cdot D)^{3/2}$ $Q2 = C2 \cdot B \cdot D \cdot \sqrt{(2g1.3D)}$ $Qout=(Q2-Q1)/(0.6D)\{H(t)-(Ho+1.2D)\}+Q1$ 

(2) Results of Analysis

Flood control effects at gate dam sites, flood control effects at the flood control points and hydrographs are shown in Table 6.5-3, Table 6.5-4, and Figure 6.5-5, respectively. Flood control effect at 4 m^3/s at Manggarai is expected.

Table 6.5-3 Flood Control Effects at Gate Dam Sites

Gate Dam	Crest			Peak Inflow (m^3/s)	Outflow at Peak Inflow (m^3/s)	Peak Cut (m^3/s)	Maximum Outflow (m^3/s)	Effect (m^3/s)	Maximum Reservoir Level (El.m)	Flood Control Volume (1000m^3)
	Width (m)	Height (m)	Number of Gate							
Pesona Kayangn	6.00	3.55	5	743.0	737.0	6.0	738.0	5.0	66.19	303
Bella Cassa	6.00	3.66	5	764.0	761.0	3.0	762.0	2.0	47.96	163

Table 6.5-4 Total Flood Control Effects of Gate Dams at Flood Control Points

Point	Before Control (m^3/s)	After Control (m^3/s)	Effect (m^3/s)
Manggarai	720	716	4
Depok	769	762	7

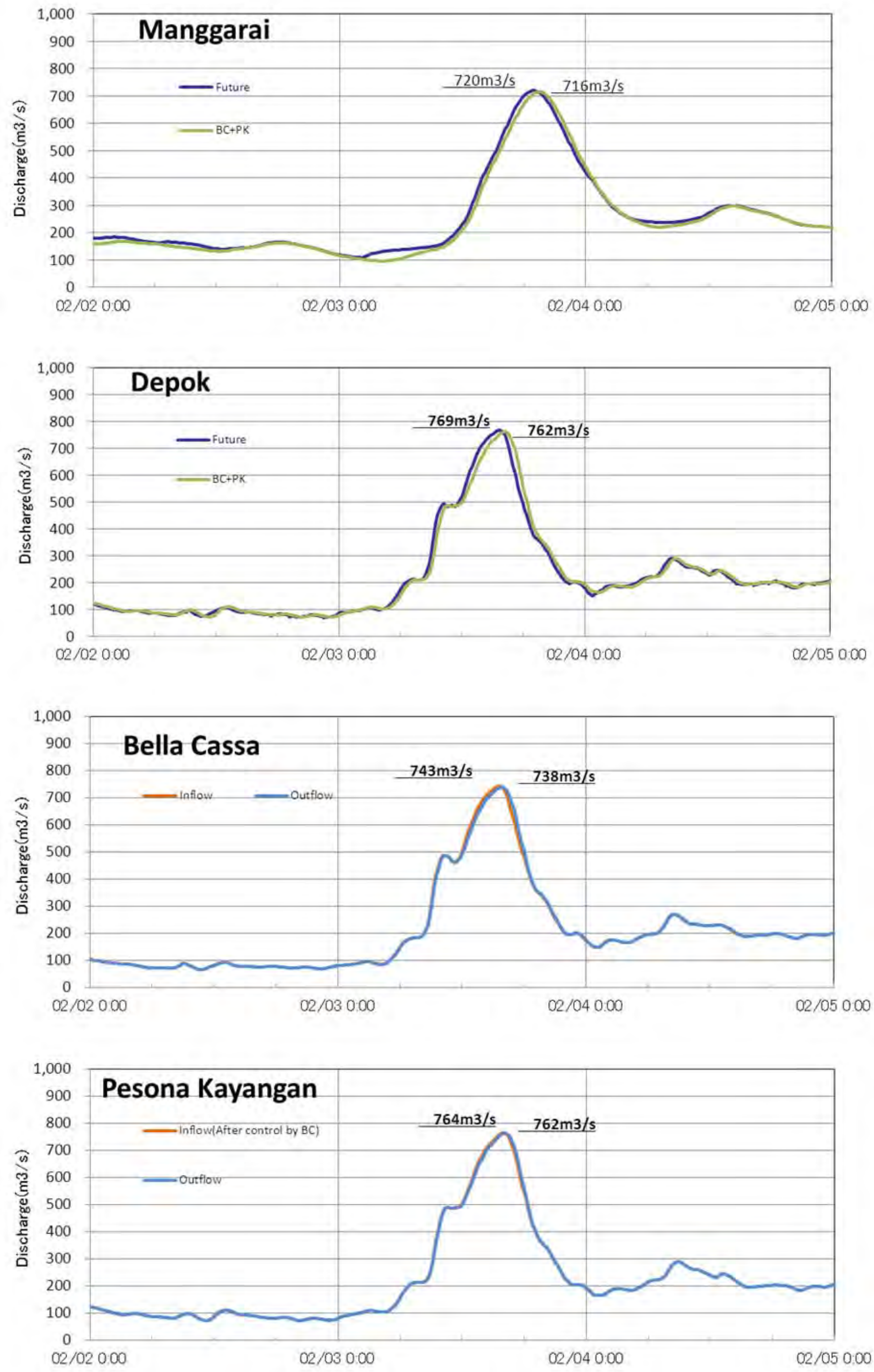


Figure 6.5-5 Hydrographs

6.6 Tunnel Storage

6.6.1 Plan and Design of Tunnel Storage

Tunnel storage is to be constructed under arterial road to mitigate peak discharge downstream by storing a part of flood discharge during flood. After flood, stored water is discharged by pumping. Proposed tunnel route and plan are shown below.

Table 6.6-1 Basic Specifications of Tunnel Storage

Case	Area	Length (km)	Inner diameter (m)	Flood control storage (m^3)
Route 1	MT.Haryono ~ Jawa Sea	20.0	12	1,809,000
Route 2	Outer Ring Road ~ Krukut River	6.1	12	550,000

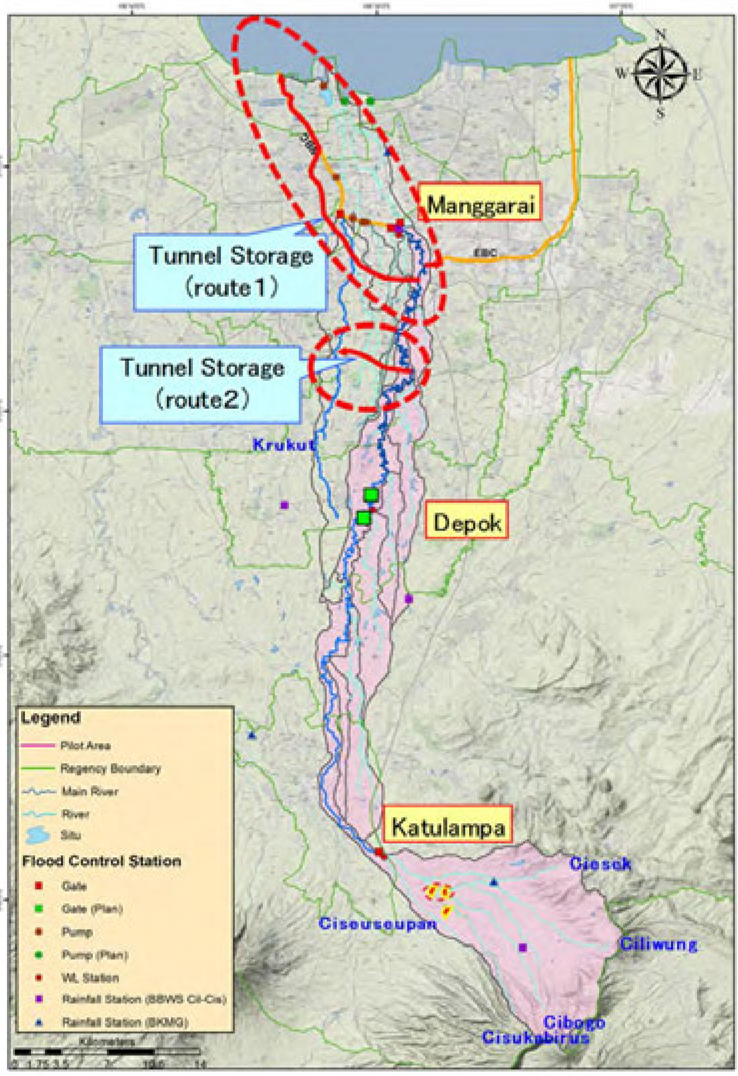


Figure 6.6-1 Location of Tunnel Storage

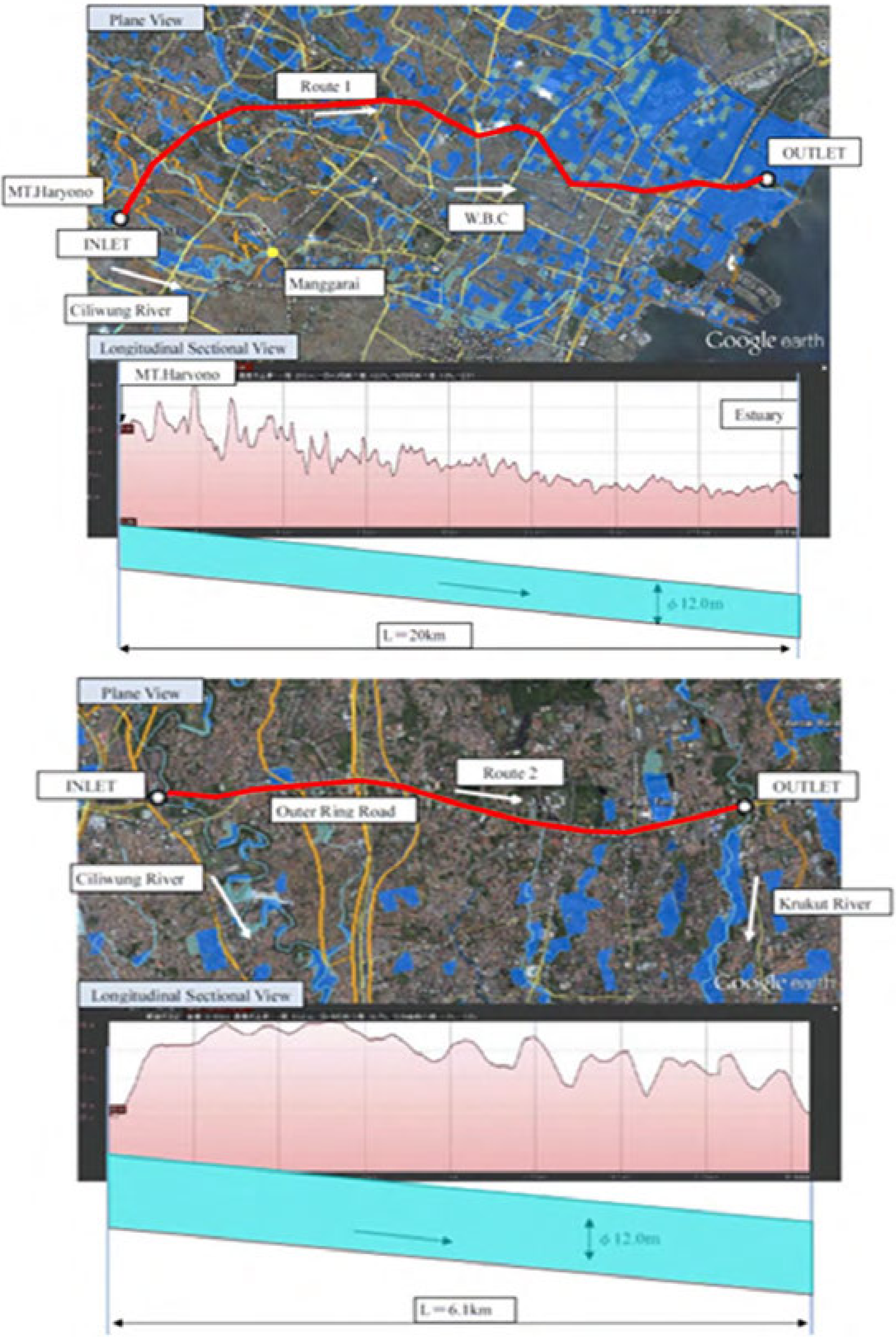


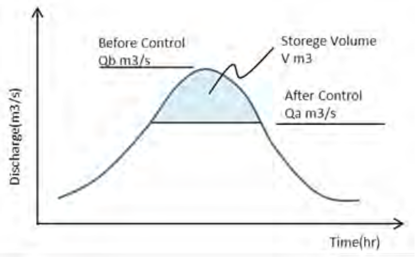
Figure 6.6-2 Outline of Tunnel Storage

6.6.2 Flood Control Effect of Tunnel Storage

(1) Conditions for Analysis of Flood Control Effect

Conditions of analysis are summarized as follows.

Table 6.6-2 Conditions for Analysis of Flood Control Effect of Tunnel Storage

Item	Description
Flood Case	February 2007 Flood, W=1/50 years
Land Use	Future
Storage Volume	Route 1: 1,810,000 m^3 Route 2: 552,000 m^3 Route 1+2: 2,362,000 m^3
Calculation Method of Storage Effect	Flood discharge at Manggarai after control by tunnel storage is calculated by horizontally cutting of original hydrograph. 

(2) Results of Analysis

Storage effects of the tunnel storage and hydrograph at Manggarai before and after control are shown in Table 6.6-3 and Figure 6.6-3, respectively. Flood control effect of 167m^3/s at Manggarai is expected by both Route 1 and 2.

Table 6.6-3 Storage Effect of Tunnel Storage at Manggarai

Case	Before Control (m^3/s)	After Control (m^3/s)	Effect (m^3/s)	Storage Volume (1,000m^3)
Route 1	720	579	141	1,809
Route 2	720	655	65	550
Route 1+2	720	553	167	2,361

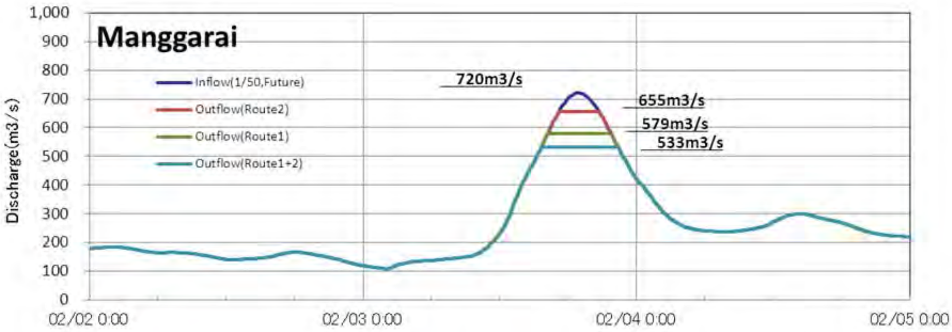


Figure 6.6-3 Hydrograph at Manggarai

6.7 Summary of Flood Control Effects by Candidate Flood Control Facilities

The target flood control discharge was established in the Total Solution as 160m^3/s assuming the following priority structural measures are implemented.

- Channel Improvement (Manggarai Gate～Outer Ring Road) ⇒500m^3/s
- Rehabilitation of Manggarai Gate and Karet Gate ⇒500m^3/s
- Diversion tunnel to EBC ⇒ 60m^3/s

The flood control effects of candidate flood control measures at Manggarai are summarized as follows. Since the small dams and gate dams reach about 20% against the target flood control discharge, they are not recommended.

Table 6.7-1 Flood Control Effects of Candidate Flood Control Measures

Case	Before Control (m^3/s)	After Control (m^3/s)	Effect (m^3/s)
Ciawi Dam 1 (Downstream plan)	720	625	95
Ciawi Dam 2 + (Upstream plan) Cisukabirus Dam		590	130
Small Dam		683	37
Gate Dam		716	4
Tunnel Storage (Route 1)		579	141
Tunnel Storage (Route 1)		655	65

Comparison of flood control effects and estimated costs of the large dams and tunnel storages are summarized in Figure 6.7-1. As shown in Figure 6.7-1, Ciawi Dam-2+Cisukabirus Dam shows the highest feasibility in aspects of flood control effects and costs. Its flood control effect is about 130m^3/s at Manggarai. To implement dam construction, the following main issues shall be solved.

- Design of dam heights based on detailed geotechnical investigations
- Sedimentation in the reservoirs
- Land acquisition/compensation

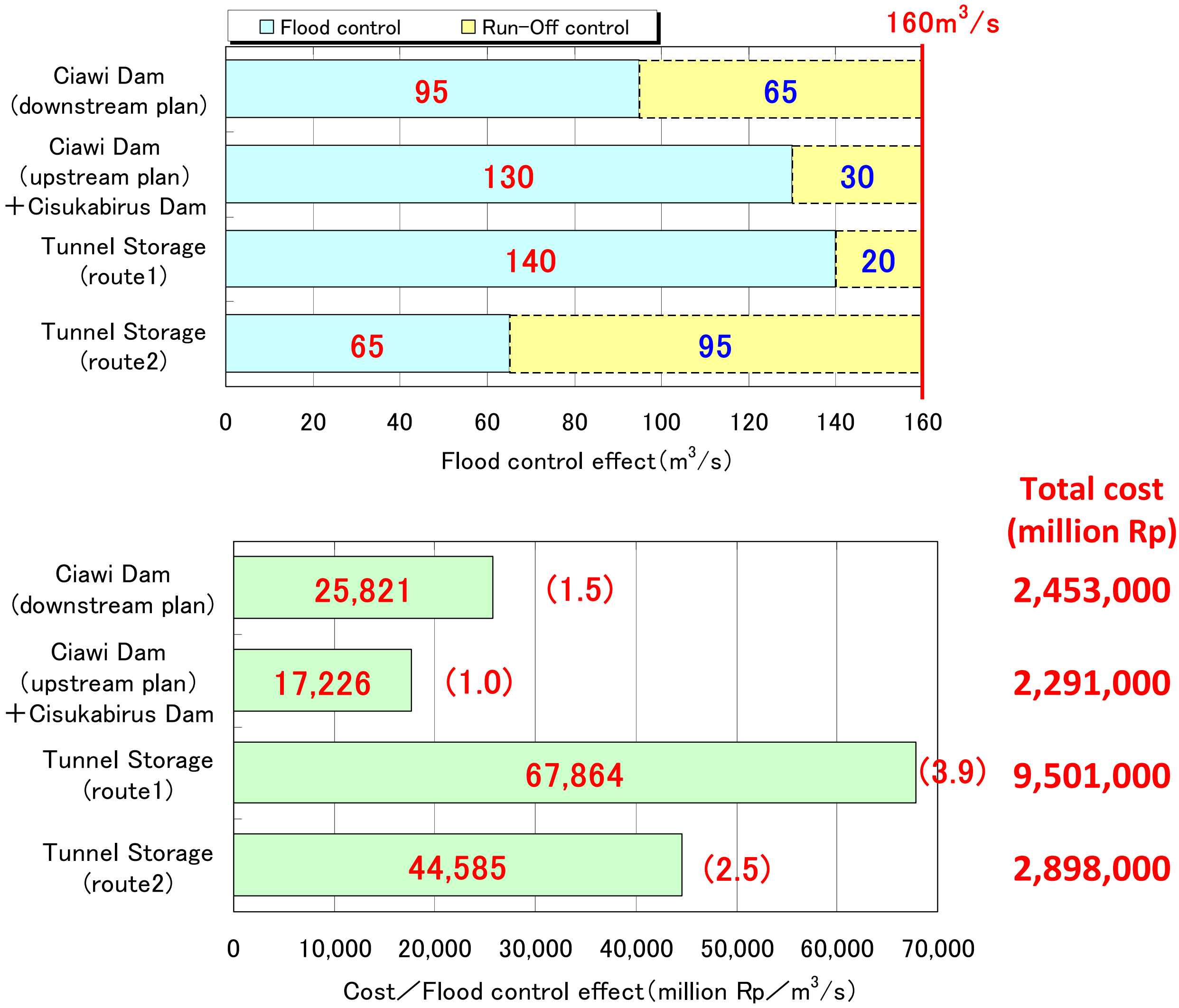
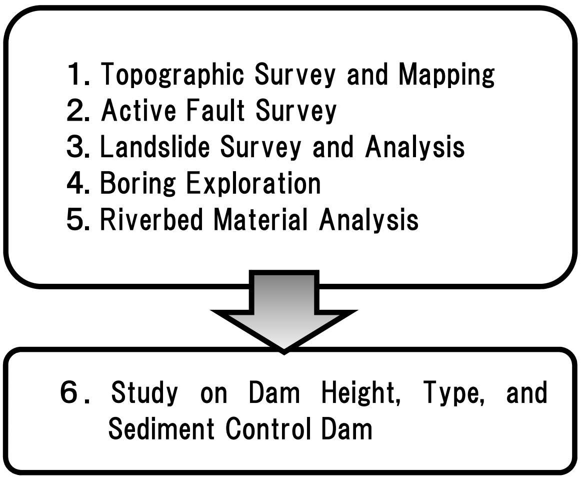


Figure 6.7-1 Flood Control Effects and Costs of Large Dams and Tunnel Storage

6.8 Proposal of Necessary Surveys and Studies

Minimum required survey and study items to design the technically feasible dams are listed as follows.

Item	Purpose	Scope of Works
1) Topographic Survey and Mapping	To obtain basic data for storage capacity calculation, landslide analysis, active fault investigation	Mapping Scale : • Reservoir (1/1,000) • Dam Site (1/500) • Aerial Photo(1/10,000)
2) Active Fault Survey	To check if active faults exist	Reading lineament by means of topographic map and aerial photo
3) Landslide Survey and Analysis	To judge possibility of slope failure due to water level fluctuation	Judging landslide morphology using topographic map and aerial photo, and by site survey
4) Boring Exploration	To assess strength and permeability of foundation ground, and ground water level	Boring (Minimum 3 sites, riverbed and the both bank) Length : 100m (to base rock) Tests : • Standard penetration test • Lugeon test • Ground water observation • Loading tests in bore hole • Rock laboratory test
5) Riverbed Material Analysis	To obtain basic data for study on sediment control dam	Test : Particle size distribution
6) Study on Dam Height, Type, and Sediment Control Dam	To review and determine dam height and dam type based on the result of (1) ~ (5)	Designing the dam and sediment control dam



(1) Topographic Survey and Mapping

Purpose: To obtain basic data for storage capacity calculation, landslide analysis, and active fault investigation
Scope of Works: Reservoir → Topographic Map 1/1,000, Aerial Photo 1/10,000
Dam Site →Topographic Map 1/500

(2) Active Fault Survey

Purpose: To check if active faults exist, since the site with active fault is not suitable for dam site.
Scope of Works: Literature analysis. Reading lineament by means of topographic map and aerial photo

(3) Landslide Survey and Analysis

Purpose: To judge possibility of slope failure due to water level fluctuation
Scope of Works: Judging landslide morphology using topographic map and aerial photo, and by site survey

(4) Boring Exploration

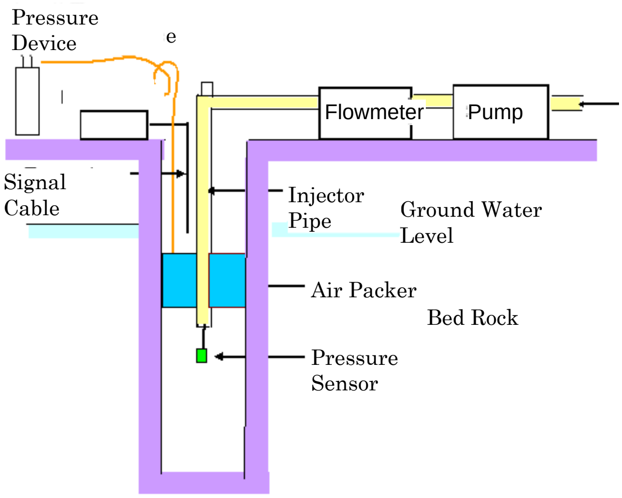
Purpose: To assess strength and permeability of foundation ground and ground water level
Scope of Works: Boring and Geotechnical Tests
Diameter: 66mm
Number: Minimum 3 sites, riverbed and the both bank
Length: To base rock about L=100m
Test:

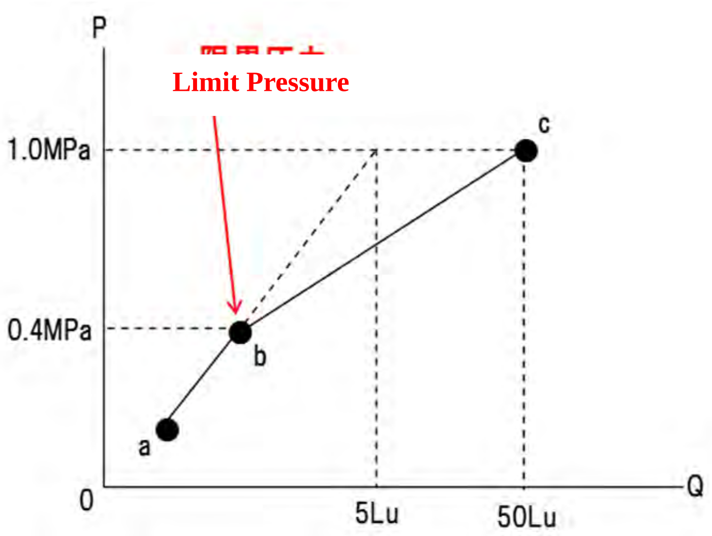
- Standard penetration test
- Lugeon test
- Ground water observation
- Loading tests in bore hole
- Rock laboratory test

*** Lugeon Test**

Test to measure permeability of rock by measuring percolation volume per meter of pressured water with 10kg/cm^2 poured to bore hole, ex. 1L/min = 1 Lu.

Since percolation volume to the foundation will rapidly increase during flood, limit pressure is important. Pressure more than the limit pressure works on the foundation, seepage failure may occur.



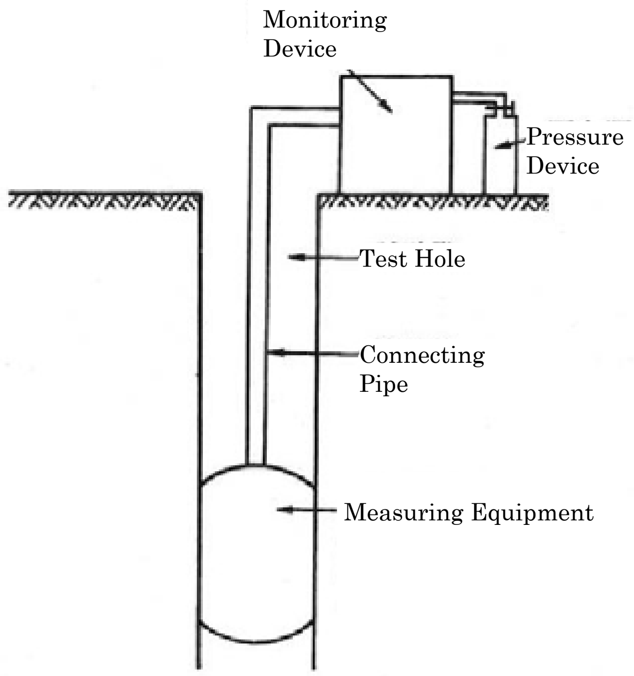


*** Loading Test in Bore Hole, Rock Laboratory Test**

Loading test in bore hole is to analyze horizontal deformation of rocks based on relation of displacement of rocks and pressure by pressurization of bore hole.

Rock laboratory test using boring core is to measure unconfined compression strength, density and water absorption.

Loading test in bore hole and rock laboratory test results gives important parameters for mechanical evaluation of base rock such as sheer strength and elastic coefficient. It is ideal to conduct in situ sheering test and deformation test, however, loading test in bore hole and rock laboratory test is sufficient for preliminary mechanical evaluation of base rock.



(5) Riverbed Material Analysis

Purpose: To obtain basic data for study on sediment control dam, since sediment discharge is large in Ciliwung River induced by surface erosion and debris flows from tributaries which may harms service outlet of the dam by clogging or abrasion.
Scope of Works: Particle size distribution analysis

(6) Study on Dam Height, Type, and Sediment Control Dam

Purpose: To review and determine dam height and dam type based on the result of (1) ~ (5)
Scope of Works: Designing the dam and sediment control dam

CHAPATER 7 EFFECT OF COMPREHENSIVE FLOOD MANAGEMENT PLAN

Flood control effect of the CFMP is verified as follows.

7.1 1/50 Years Flood

7.1.1 Conditions for Verification

Flood control effect is verified by flood analysis before and after flood control measures for 1/50 years flood which proposed in the CFMP. The conditions for analysis are as follows.

Table 7.1-1 Conditions for Verification against 1/50 Flood

Item	Current (Before measures)	After Completion
Land Use	Future (2030)	
Topography	Current (2008)	
Structural Measures	Current (2011)	CFMP Facilities Completion
Flood Control	-	Channel Improvement by BBWS Diversion to EBC 2 Dams（Ciawi-2 + Cisukabirus）
Runoff Control	-	Scenario 2
Hydrograph	February 2007 Flood	
Flood Probability	1/50 Years	

7.1.2 Results of Verification

As the results of flood simulation, water level and discharge hydrograph at Manggarai is shown in Figure 7.1-1 while the difference of inundation depth and area are shown in Figure 7.1-2 and Figure 7.1-3, respectively.

(1) Water Level and Discharge

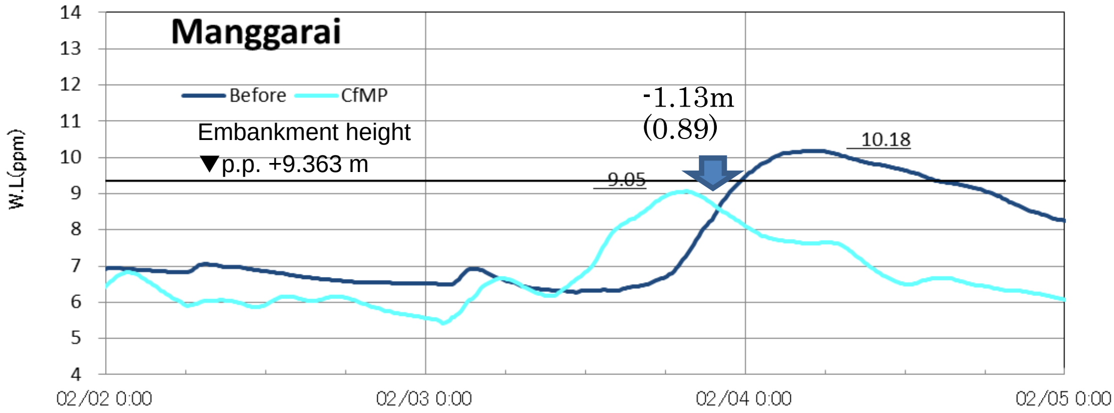
Flood control effects of the CFMP in terms of water level and discharge are summarized as follows.

- Inundation in upstream basin decreases by the channel improvement works resulting increase of river discharge at Manggarai by $114m^3/s$, equivalent to 27% increase.
- Water level at Manggarai decrease by 1.13m, equivalent to 11% decrease by the channel improvement and rehabilitation of Manggarai Gate.

(2) Inundation

Flood control effects of the CFMP in terms of inundation in the upstream basin of Manngarai are summarized as follows.

- Inundation area decreases by $3.12km^2$, equivalent to 36% decrease.
- Inundation water volume decreases by 1.77 million m^3, equivalent to 75% decrease.
- Average inundation depth decreases by 1.66m, equivalent to 62% decrease.



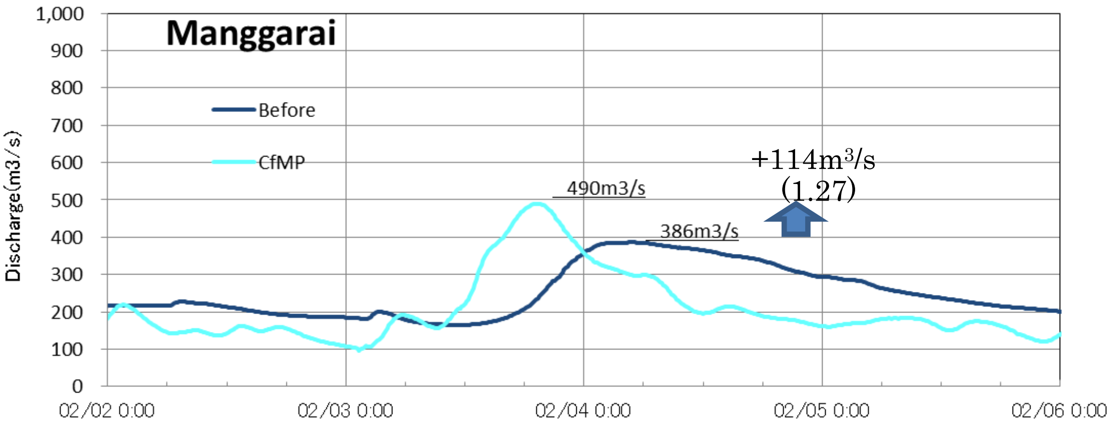


Figure 7.1-1 Hydrograph of 1/50 Years Flood at Manggarai

(3) Estimated Flood Damage

Estimated flood damages by 1/50 years flood before and after flood control measures are summarized below, and the damage reduction is estimated about Rp. 5 billion.

It is noted that the area for estimation of flood damage includes Ciliwung River Basin and Pasanggrahan River Basin, and compound flood damage by both river water and inland water is estimated.

By the structural measures under CFMP, flood damage along Ciliwung River is mitigated, however, the measures cannot mitigate inland water inundation in the low land area directory. Thus, flood damage still remains after the CFMP completion. Besides, flood damages are estimated in Krukut River and Pasanggrahan River basins while flood control measures are not planned in the CFMP. It causes large disaster damage remains even after the CFMP completion.

Table 7.1-2 Estimated Flood Damage by 1/50 Years Flood before and after CFMP Implementation

		Amount (Billion RP.)	
		Before CFMP	After CFMP
Direct Damage on Building	Household	13,325.2	11,403.0
	Business/Office	7,853.8	6,724.2
	Manufacture	1,172.1	1,003.5
Direct Damage on Asset	Household	5,135.1	4,504.6
	Business/Office	3,199.6	2,825.5
	Manufacture	3,053.4	2,902.2
Damage on Infrastructure		7,529.3	6,565.8
Indirect : Income Loss	Business/Office	2,668.1	2,349.0
	Manufacture	641.4	564.7
Indirect : Electricity Loss		84.9	76.1
Indirect : Water Loss		33.1	29.2
Direct and Indirect Damage Total		44,696.0	38,947.6

Note: area for estimation of flood damage includes Ciliwung River Basin and Pasanggrahan River Basin.

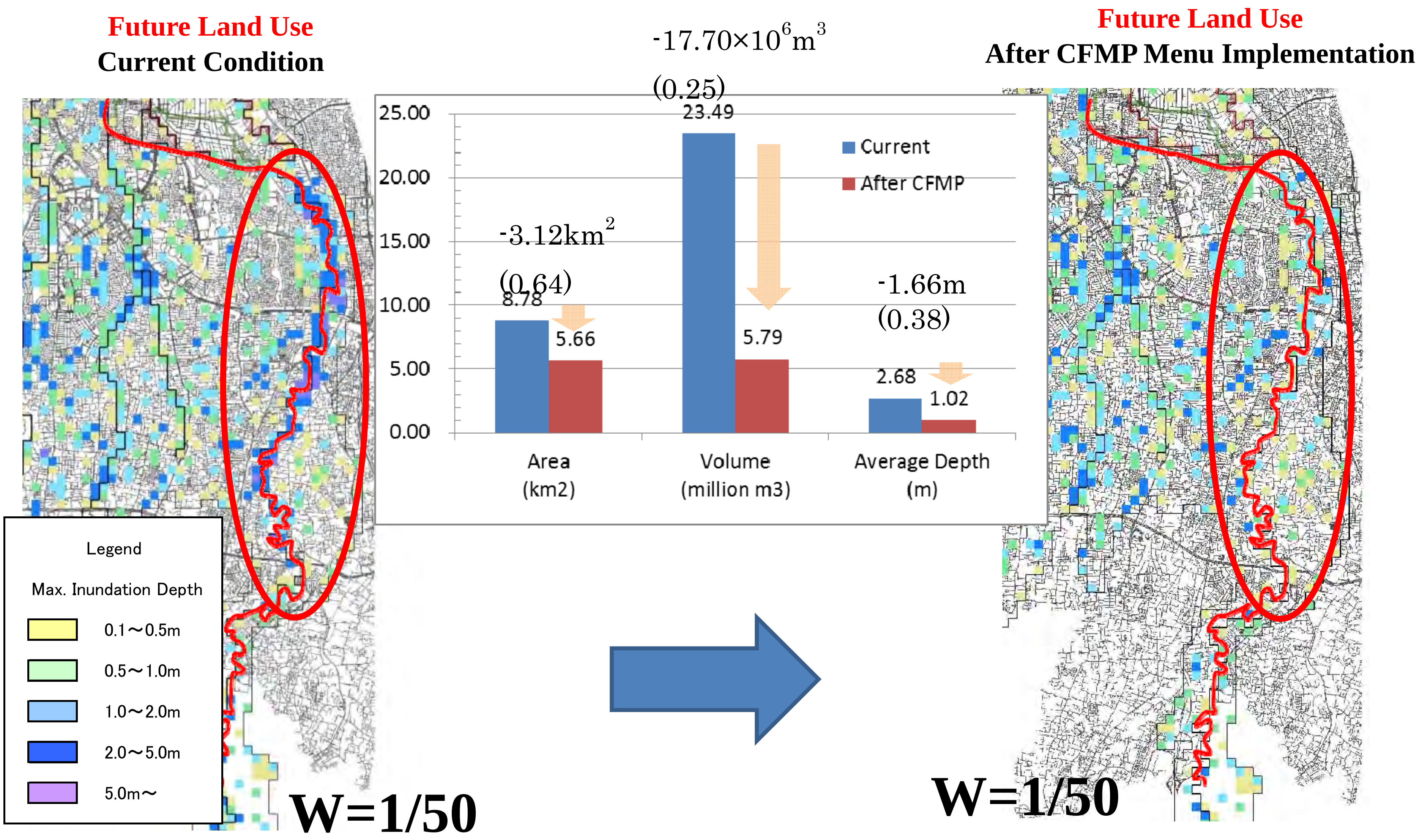


Figure 7.1-2 Estimated Maximum Inundation Depth (Left: Before CFMP, Right: After CFMP)

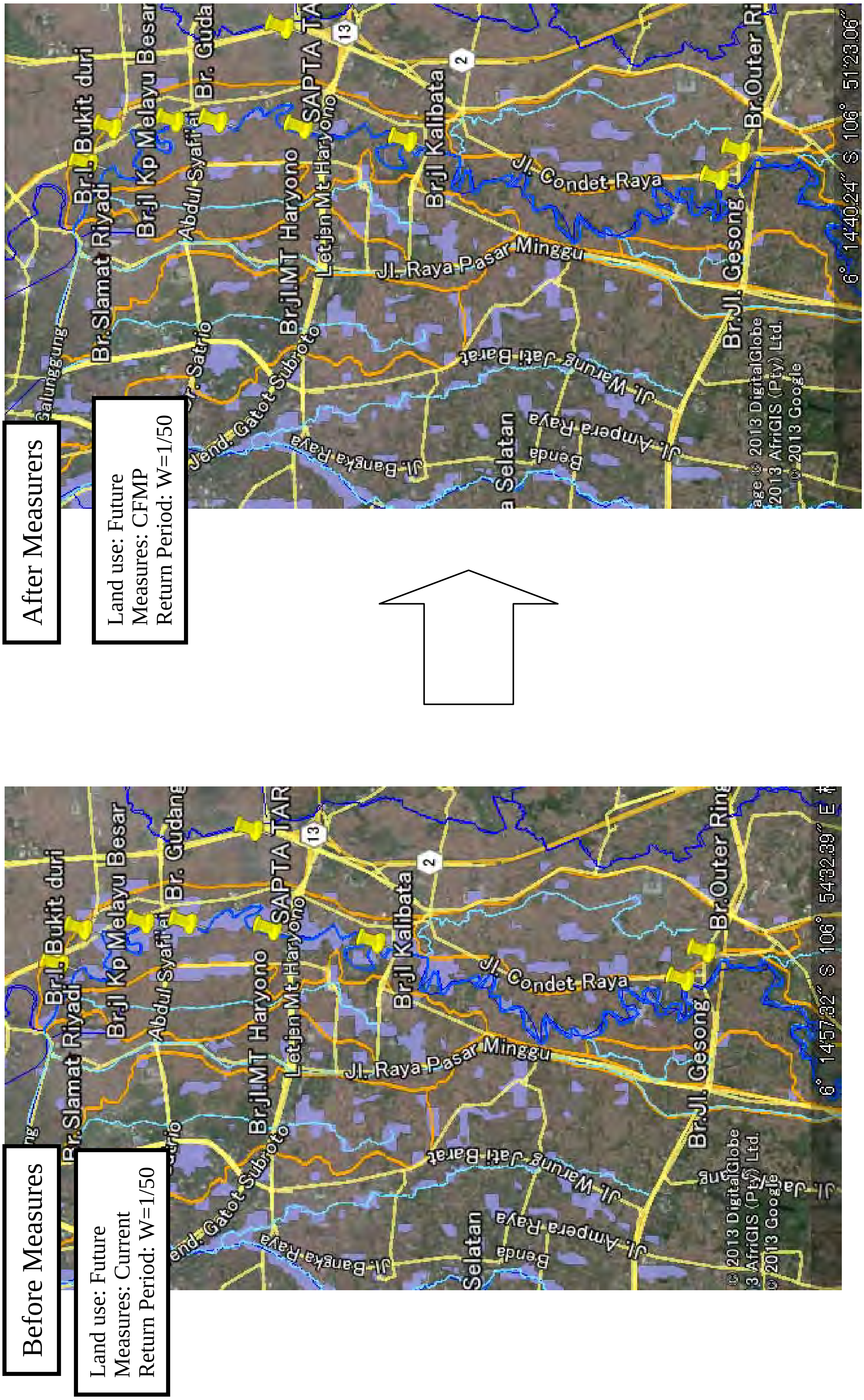


Figure 7.1-3 Estimated Inundation Area before and after CFMP

7.2 1/100 Years Flood

7.2.1 Conditions for Verification

As an excess flood, inundation conditions against 1/100 years flood are simulated. The conditions for analysis are as follows.

Table 7.2-1 Conditions for Simulation of 1/100 Flood

Item	After CFMP Completion with 1/50 Flood	After CFMP Completion with 1/100 Flood
Landuse	Future (2030)	
Topography	Current (2008)	
Structural Measures	CFMP Facilities Completion	CFMP Facilities Completion
Flood Control	Channel Improvement by BBWS Diversion to EBC 2 Dams（Ciawi-2 + Cisukabirus）	Channel Improvement by BBWS Diversion to EBC 2 Dams（Ciawi-2 + Cisukabirus）
Runoff Control	Scenario 2	Scenario 2
Hydrograph	February 2007 Flood	
Flood Probability	1/50 Years	1/100 Years

7.2.2 Results of Verification

As the results of flood simulation, water level and discharge hydrograph at Manggarai is shown in Figure 7.2-1 while the difference of inundation depth and area are shown in Figure 7.2-2 and Figure 7.2-3, respectively.

(1) Water Level and Discharge

Water level and discharge at Manggarai by 1/100 years flood are different from 1/50 years flood as follows.

- Discharge at Manggarai is larger by 39m^3/s, equivalent to 8% larger than 1/50 years flood.
- Water level at Manggarai is larger by 0.28m, equivalent to 3% larger than 1/50 years flood.

(2) Inundation

Inundation conditions comparing 1/50 years flood are summarized as follows.

- Inundation area is larger by 0.37km^2, equivalent to 7% larger than 1/50 years flood.
- Inundation water volume is larger by 0.80 million m^3, equivalent to 14% larger than 1/50 years flood.
- Average inundation depth is larger by 0.08m, equivalent to 14% larger than 1/50 years flood.

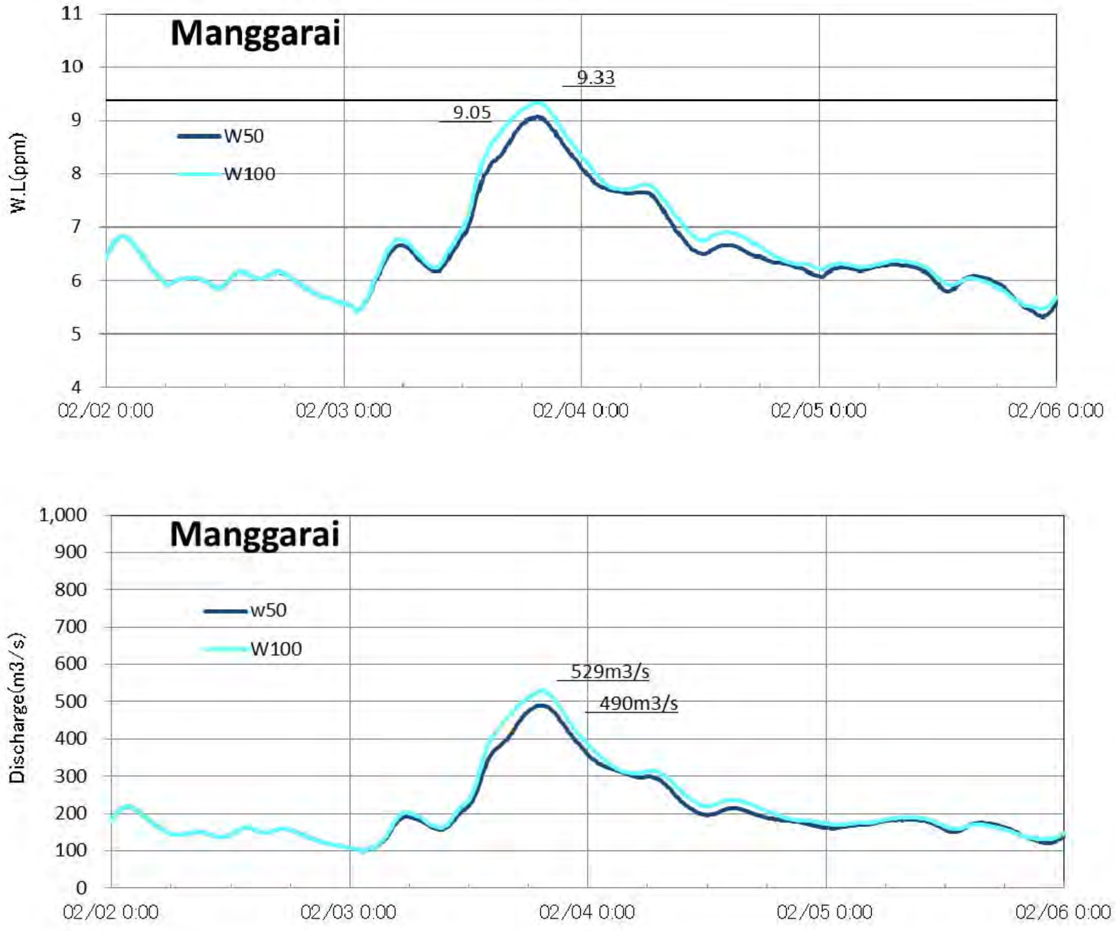


Figure 7.2-1 Hydrograph of 1/50 and 1/100 Years Flood at Manggarai

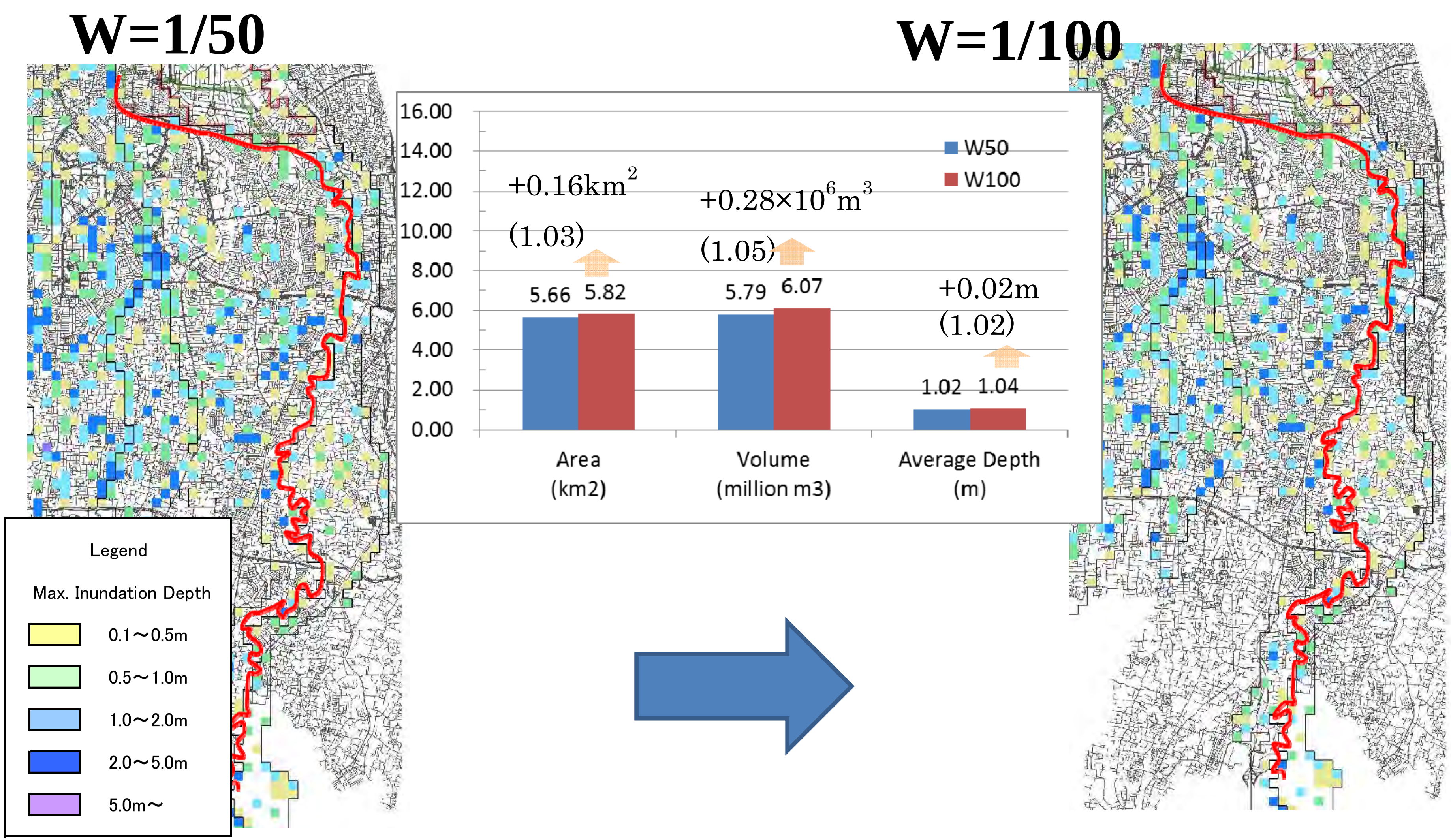


Figure 7.2-2 Estimated Maximum Inundation Depth (Left: 1/50 Years Flood, Right: 1/100 Years Flood)

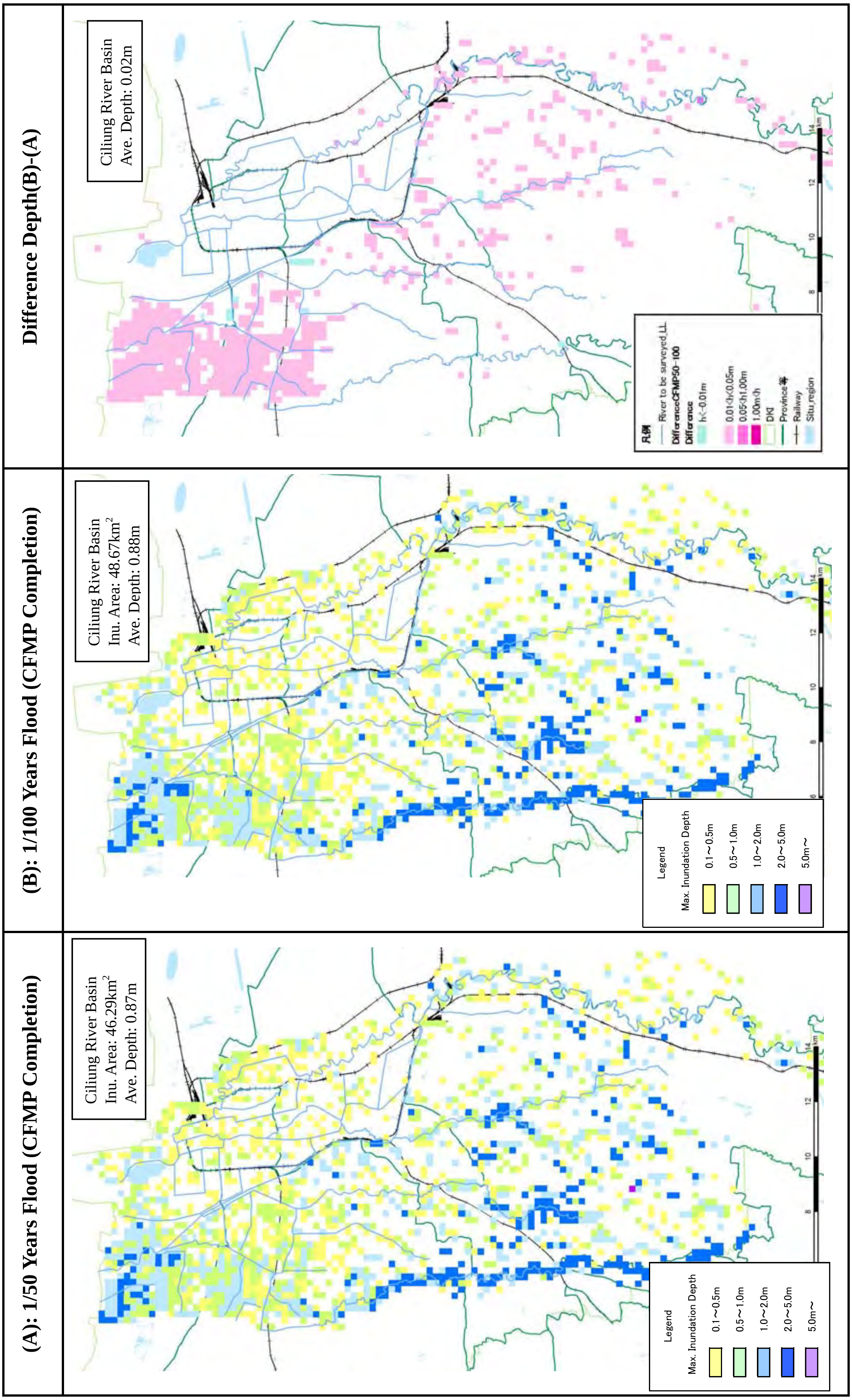


Figure 7.2-3 Comparison of Inundation Conditions between 1/50 and 1/100 Years Floods

7.3 Effect of Climate Change

7.3.1 Objective

The objective is to simulate flood conditions using the distributed type flood analysis model, to evaluate change of flood safety degree due to climate change effect and to propose possible countermeasures against climate change.

7.3.2 Climate Change Scenario and Conditions for Simulation

(1) Climate Change Scenario

Climate change scenarios are built in order to estimate the increase in rainfall and sea level rise in 2030. Following climate change scenarios were settled based on social and economic changes described in the IPCC 4th Assessment Report.
In the “Simulation Study on Climate Change in Jakarta, Indonesia (JICA)”, the following two scenarios are applied to estimate climate change in 2050 as the most possible socio-economic conditions. In this Study, same scenarios are applied to estimate climate change in 2030.

・A1FI Scenario : High Growth Society Scenario Valuing on the Fossil Energy Source
・B1 Scenario : Sustainable Development Society Scenario

Table 7.3-1 Climate Change Scenarios

Scenario*		Application			
		Manila	Bangkok	Ho Chi Minh	**Jakarta**
A1	Growth-oriented Society Scenario				
A1FI	Value on Fossil Energy Resources	●	●	–	●
A1T	Value on Non-Fossil Energy Resources	–	–	–	–
A1B	Value on Balance of Energy Resources	–	–	–	–
A2	Pluralistic Society Scenario	–	–	●	–
B1	Sustainable Development Society Scenario	●	●	–	●
B2	Community Coexistence Scenario	–	–	●	–

Remarks: *)Social and economic changes in IPCC 4th assessment report
Source: The Simulation Study on Climate Change in Jakarta, Indonesia

<Forecast Scenarios (Reference)>

These scenarios do not include the additional global warming measures

■A1 "Growth-oriented Society Scenario"

・World's economy will develop more and great innovation will be come up.
A1FI: Value on Fossil Energy Resources
A1T: Value on Non-Fossil Energy Resources
A1B: Value on Balance of Energy Resources

■A2 " Pluralistic Society Scenario"

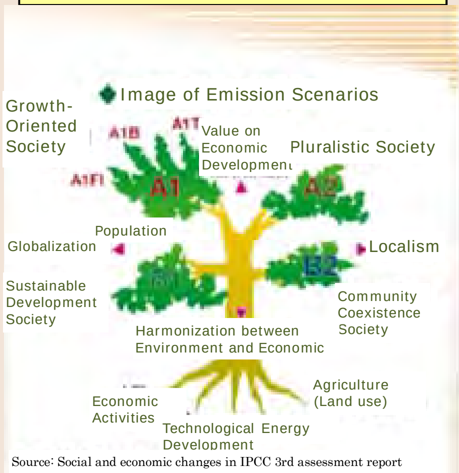
・World's economy and politics will be divided into blocks, and trading and movement of people/technologies will be restricted.
・World's economy will grow slower, and concerns for environment will be relatively scarce.

■B1 "Sustainable Development Society Scenario"

・Environmental protection and economic development will be promoted at the same time.

■B2 "Community Coexistence Scenario"

・Value on the problem solution in the communities and fairness of world, and economic development will be somewhat slow.
・Environmental issues will be resolved within each community.



Source: Ministry of Environment "Global Warming

Source: Summary on IPCC 4th Assessment Report (Official Edition)

Figure 7.3-1 Climate Change Scenarios by IPCC 4th Assessment Report

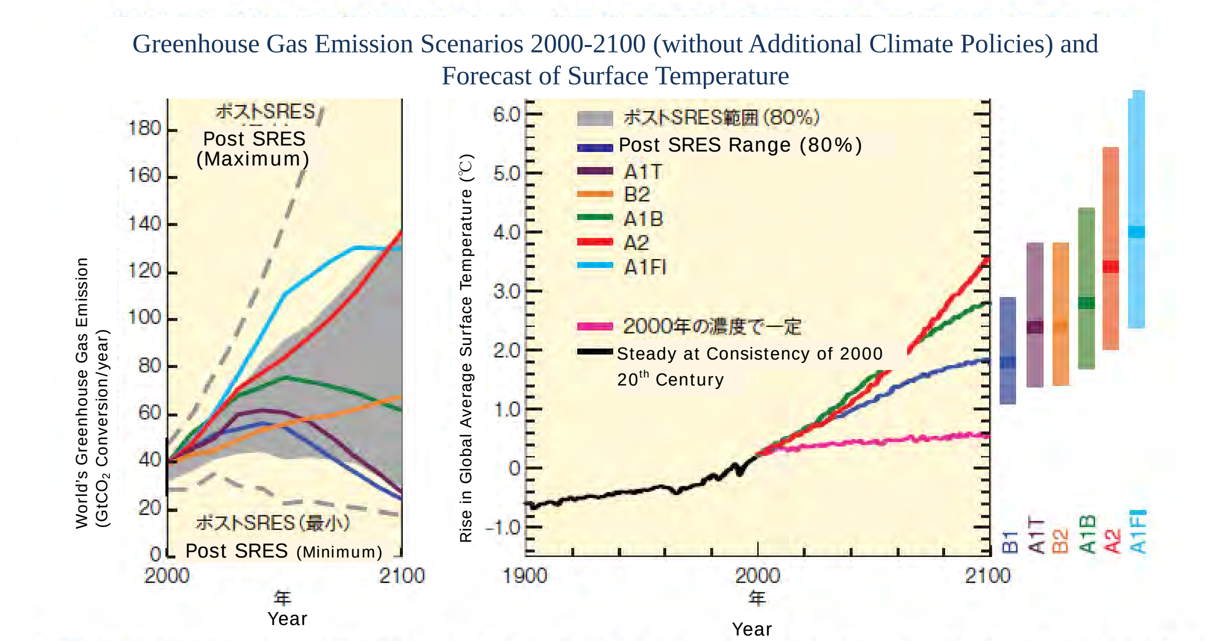


Figure SPM.5. ***Left Figure***: amount of greenhouse emission (CO2 conversion) without additional climate policies: six SRES marker scenarios (colored lines), 80% tile of recent scenarios (post SRES) publicized after SRES (range with grey colored). Dot lines are overall range of results of post SRES scenario. CO2, CH4, N2O and CFC are included in emission amount. ***Right Figure***: solid lines show rise in global average surface temperature continued from the condition of 20th century in models of A2, A1B, B1 scenarios. These forecasts are considered with the effects of short-lived greenhouse gas and aerosol. Pink line represents the simulation of air-sea coupling system model (AOGCM) which is sustained steadily at the atmospheric concentration of year 2000, but the scenario. Right belt of the figure indicates best estimation value (horizontal line of each belt) and forecast spread of high possibility from 2090-2099 of 6 SRES scenarios. All temperatures were comparison with 1980-1999.

Source: Summary on IPCC 4th Assessment Report (Official Edition)

Figure 7.3-2 Forecast Scenarios in IPCC 4th Assessment Report

Table 7.3-2 Forecast of Rise in Global Average Surface Temperature and Sea Level Rise at the End of 21st Century

Scenarios[a)]	Changes in Temperature (difference of year 2090-2099 based on the year 1980-1999 (℃))[C)]		Sea Level Rise (difference of year 2090-2099 based on the year 1980-1999 (℃)) Forecast range by models (exclusive of mechanical changes of rapid ice discharge)
	Best estimate value	Likely forecast range	
Steady at the consistence of 2000[b)]	0.6	0.3-0.9	No data
B1 scenario	1.8	1.1-2.9	0.18-0.38
A1T scenario	2.4	1.4-3.8	0.20-0.45
B2 scenario	2.4	1.4-3.8	0.20-0.43
A1B scenario	2.8	1.7-4.4	0.21-0.48
A2 scenario	3.4	2.0-5.4	0.23-0.51
A1FI scenario	4.0	2.4-6.4	0.26-0.59

Source: Summary on IPCC 4th Assessment Report (Official Edition)
Note: a) Scenarios are six SRES marker scenarios. CO2 conversion consistence (see p.823, 1st working group report of 3rd assessment report) corresponding to the radiative forcing by man-made greenhouse gas and aerosol are SRES marker scenarios of B1, A1T, B2, A1B, A2 and A1FI, and approximately 600, 700, 800, 850, 1250, 1550ppm respectively.
b) Composition of values of steady at the consistence of 2000 is obtained only by air-sea coupling system model (AOGCM).
c) Temperature is the best estimate value and forecast range of uncertainty obtained by models belonging to various hierarchies regarding constraints by observed values and composite degrees. Changes of temperature are presented as the differences between 1980-1999. To present the changes between 1850-1899, 0.5℃ will be added.

(2) Estimation of Climate Change in 2030

Estimated increase in rainfall and sea level rise based on the selected climate change scenarios are summarized in Table 7.3-3.

Table 7.3-3 Summary of Climate Change by 2030

Climate Change Scenario	Temperature rise(℃) (downscaled)	Increased Rate of Rainfall	Sea-Level-Rise (cm)
P	-	0%	0
B1	0.5	4.6%	12
A1FI	1.0	10.3%	18

P: No Climate Change, B1: Sustainable Development Society Scenario, A1FI: Growth-oriented Society Scenario

(3) Rainfall Increment in 2030

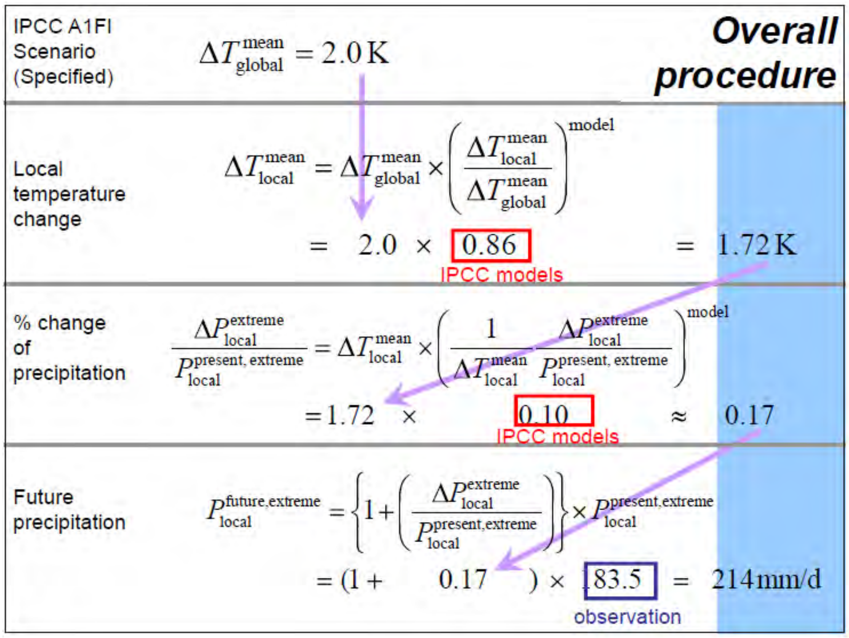
Rainfall increment in 2030 is shown in Table 7.3-4. It was estimated in accordance with downscaling procedure illustrated in Figure 7.3-3. A statistical downscaling method was applied to implement the downscaling in this Study. As a result, rainfall increment in 2030 was estimated at 10.3% in A1FI Scenario, and 4.6% in B1 scenario.

Table 7.3-4 Rainfall Increment in 2030

	A1FI	B1
Global mean temperature increase $\Delta T_{global}\,[K]$	1.2	0.54
$\Delta T_{local}/\Delta T_{global}$	0.86	
Local mean temperature change $\Delta T_{local}\,[K]$	1.03	0.46
$\frac{1}{\Delta T_{local}}\frac{\Delta P^{extreme}_{local}}{P^{present,extreme}_{local}}\,[\%/K]$	10	
Change of precipitation $\Delta P^{extreme}_{local}/P^{present,extreme}_{local}\,[\%]$	**10.3**	**4.6**

$\Delta T_{global} = T^{future}_{global} - T^{present}_{global}$

$\Delta P \;= P^{future} \quad P^{present}$



Source: The Simulation Study on Climate Change in Jakarta, Indonesia」

Figure 7.3-3 Downscaling Procedure

1) Global Mean Temperature Chnage

IPCC provides projections for global mean temperature changes for various IPCC SRES scenarios up to 2100 as shown in Table 7.3-2. Based on the increment up to 2100, the global temperature inclement in 2030 is estimated by rectilinear approximation as shown in Table 7.3-5.

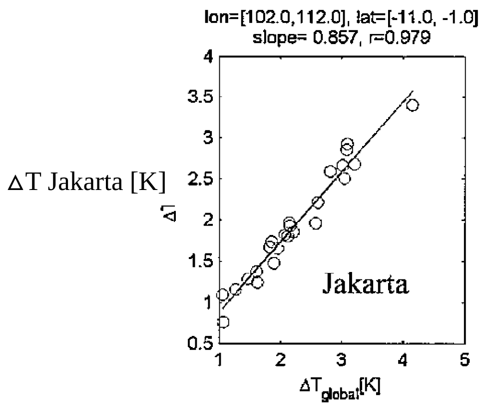
Table 7.3-5 ΔT_{global} for 2030 from IPCC AR4

Scenario	ΔT_{global} for 2100	ΔT_{global} for 2030
A1FI	4.0K	1.2K
B1	1.8K	0.54K

$$\Delta T_{global} = T_{global}^{future} - T_{global}^{present}$$

2) Local Temperature Change in Jakarta

Relation between the global mean temperature change and local temperature change in Jakarta under the A1F1 and B1 Scenarios are shown in Figure 7.3-4. The local temperature increase in Jakarta is about 90 percent of the global average temperature increase.

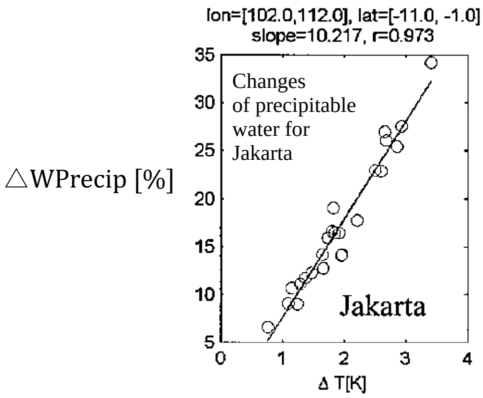


Source: The Simulation Study on Climate Change in Jakarta, Indonesia

Figure 7.3-4 Relation between Global and Local Temperature Change in Jakarta

3) Precipitable Water and Temperature Change in Jakarta

Precipitable water and temperature change in Jakarta is summarized as shown in Figure 7.3-5. Increase of precipitable water is estimated around 10% of temperature change.



Source: The Simulation Study on Climate Change in Jakarta, Indonesia」

Figure 7.3-5 Relation between Precipitable Water and Temperature Change in Jakarta

(4) Estimation of Sea Level Rise in 2030

IPCC provides projections for sea level rise for various IPCC SRES scenarios up to 2100 as shown in Table 7.3-2. Based on the increment up to 2100, the sea level rise in 2030 is estimated by rectilinear approximation as shown in Table 7.3-6.

Table 7.3-6 Sea Level Rise in 2030 (cm)

Scenario	2100	**2030**
P	0	**0 cm**
B1	38cm	**19 cm**
A1FI	59cm	**29 cm**

P: No Climate Change, B1: Sustainable Development Society Scenario, A1FI: Growth-oriented Society Scenario

7.3.3 Case of Analysis

Analysis cases and conditions are summarized as below.

Table 7.3-7 Analysis Cases and Conditions

Item	Before Climate Change	After Climate Change	After Climate Change
Scenario	Current	B1 Scenario	A1FI Scenario
Increment of Rainfall	—	1.046	1.103
Sea Level Rise	—	12cm	18cm
Hydrograph	February 2007 Flood		
Probability	1/50 Years Flood		
Landuse	Future (2030)		
Topography	Current (2008)		
Structural Measures	After CFMP Completion		
Flood Control Measures	Channel Improvement by BBWS Diversion to EBC 2 Dams（Ciawi-2 + Cisukabirus）		
Runoff Control Measures	Scenario 2		

7.3.4 Results of Analysis

(1) Summary of Results

1) Water Level and Discharge

- Discharge at Manggarai increases by $11m^3/s$ (2%) by B1 Scenario and by $57m^3/s$ (12%) by A1F1 Scenario.
- Water level at Manggarai increases by 0.08m (1%) by B1 Scenario and by 0.47m (5%) by A1F1 Scenario.

2) Inundation

<Upstream of Manggarai>

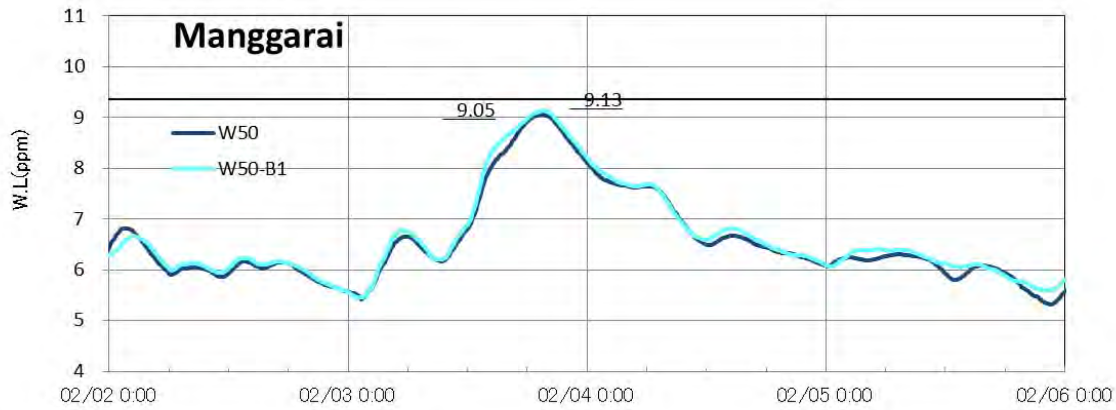
- Inundation area increases $0.21km^2$ (4%) by B1 Scenario and $0.37km^2$ (7%) by A1F1 Scenario.
- Inundation volume increases 0.29 million m^3 (5%) by B1 Scenario and 0.80 million m^3 (14%) by A1F1 Scenario.
- Average inundation depth increases 0.02m (2%) by B1 Scenario and 0.08m (14%) by A1F1 Scenario.

<Whole Ciliung River Basin>

- Inundation depth increases 0.04m by B1 Scenario and 0.08m by A1F1 Scenario.

(2) B1 Scenario

1) Water Level and Discharge Hydrograph



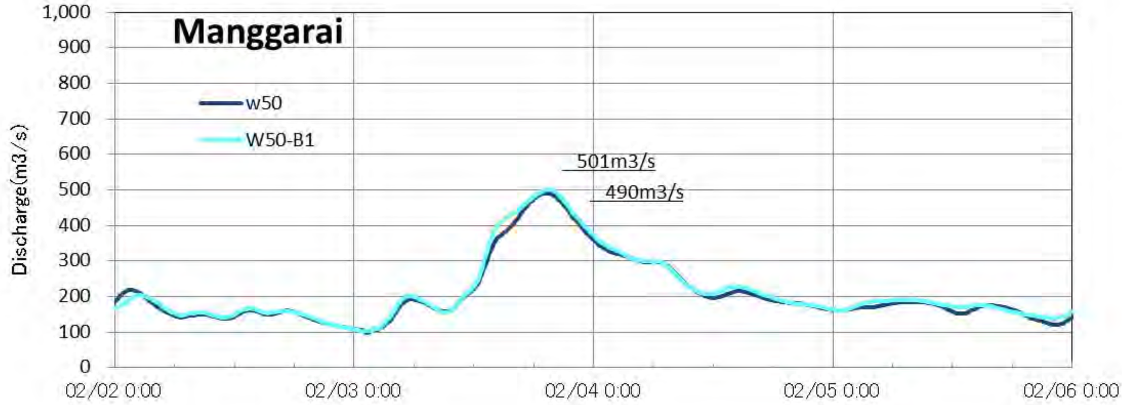


Figure 7.3-6 Water Level and Discharge Hydrograph (B1 Scenario)

2) Inundation

Simulation results of B1 Scenario are shown in Figure 7.3-7 and Figure 7.3-8.

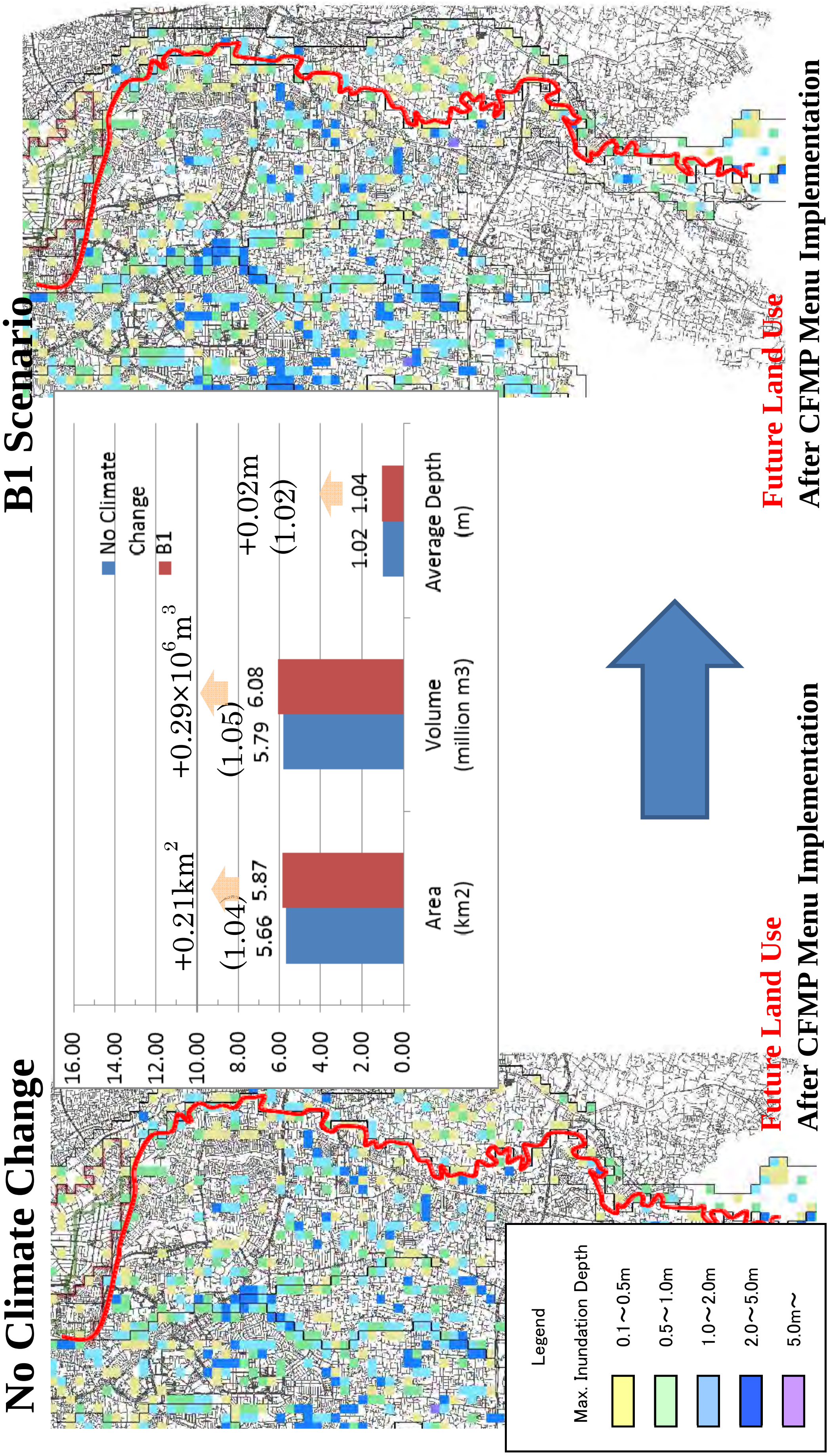


Figure 7.3-7 Estimated Maximum Inundation Depth (Left: Before Climate Change, Right: B1 Scenario)

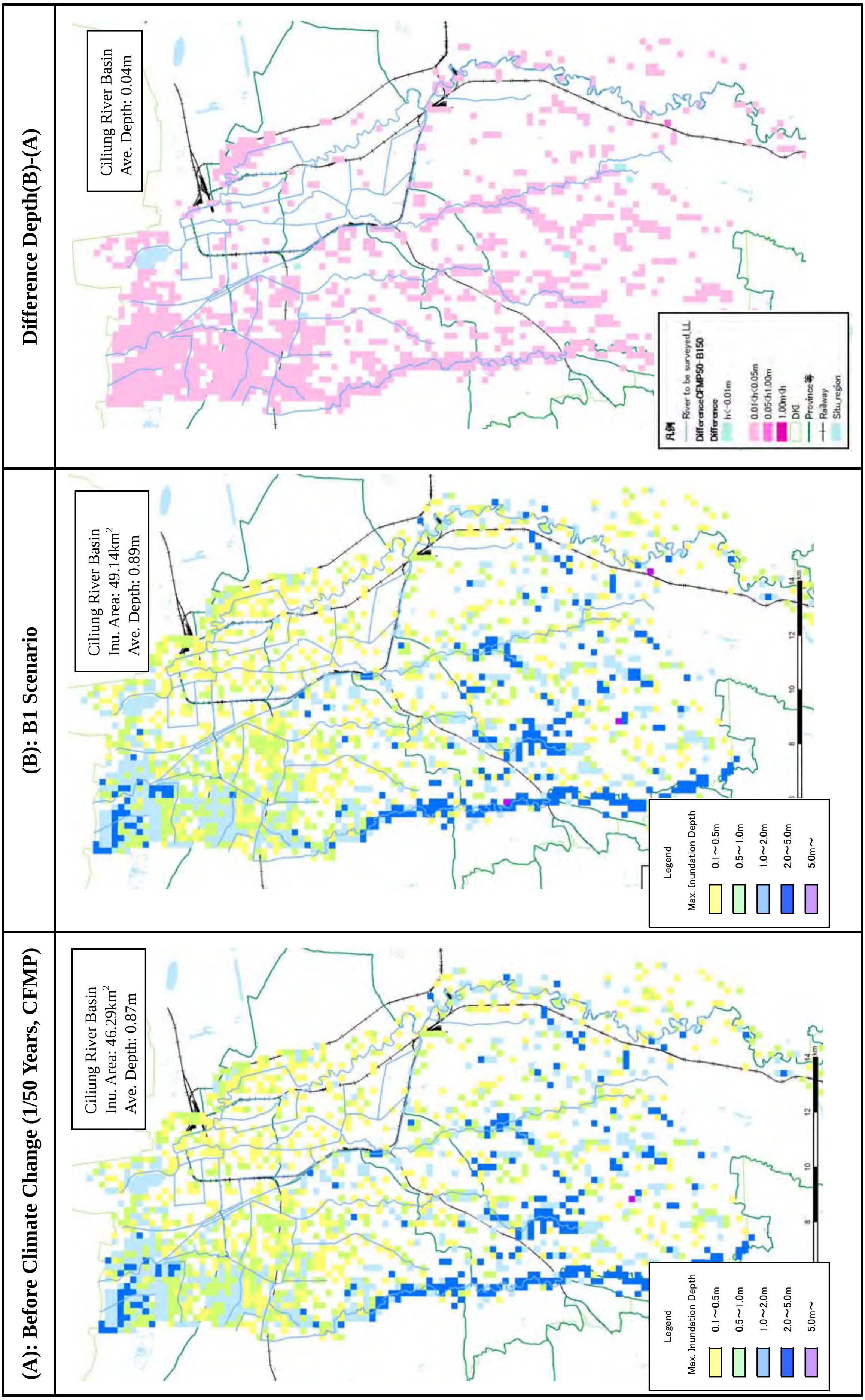


Figure 7.3-8 Comparison of Inundation Conditions between Before Climate Change and B1 Scenario

(3) A1FI Scenario

1) Water Level and Discharge Hydrograph

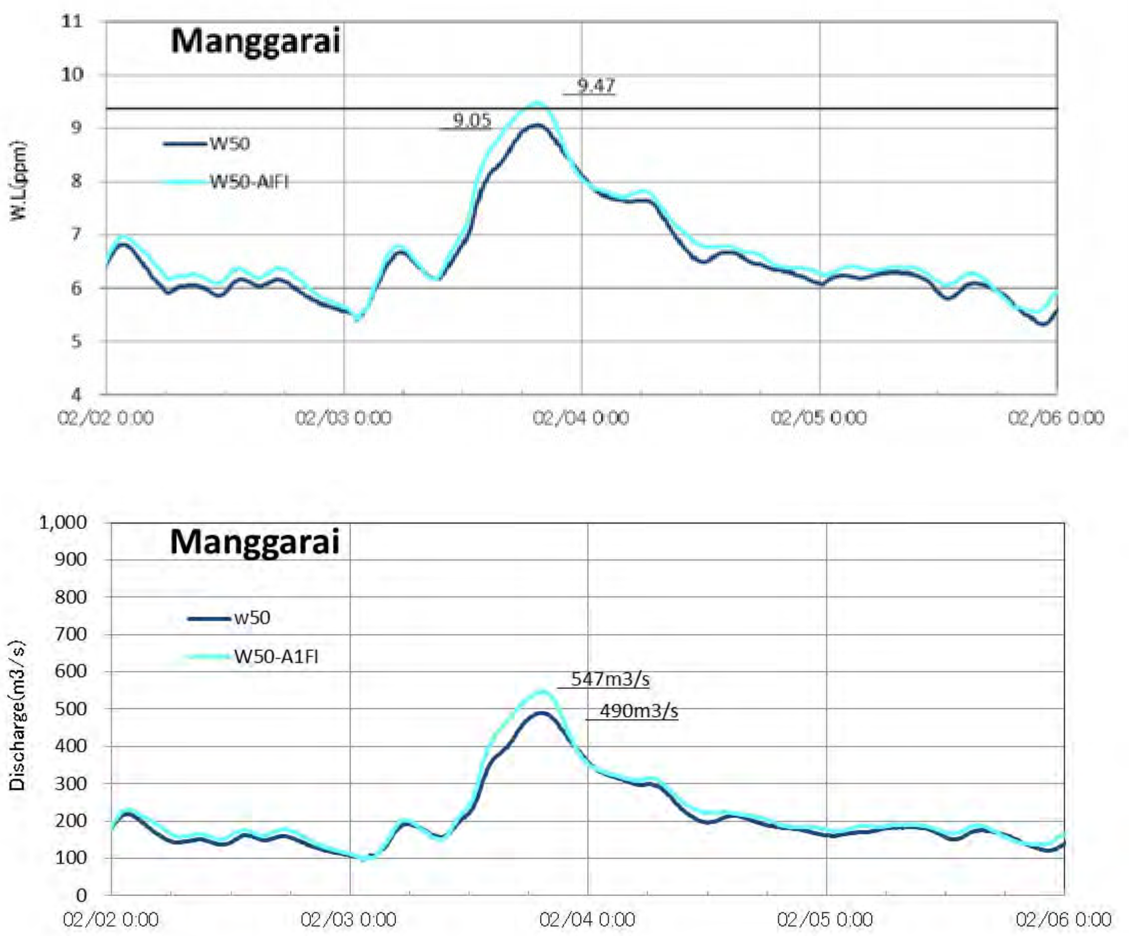


Figure 7.3-9 Water Level and Discharge Hydrograph (A1F1 Scenario)

2) Inundation

Simulation results of A1F1 Scenario are shown in Figure 7.3-10 and Figure 7.3-11.

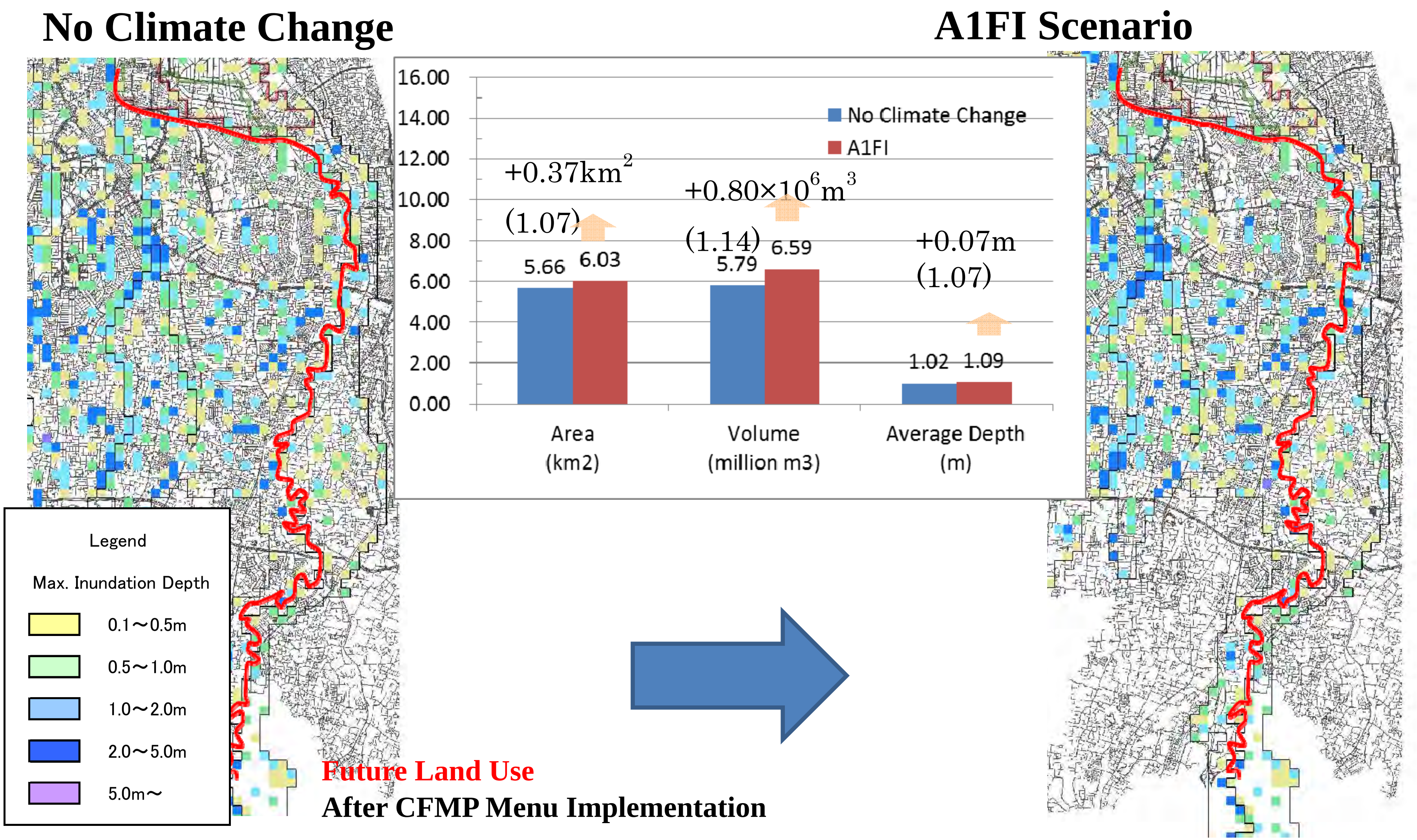


Figure 7.3-10 Estimated Maximum Inundation Depth (Left: Before Climate Change, Right: A1F1 Scenario)

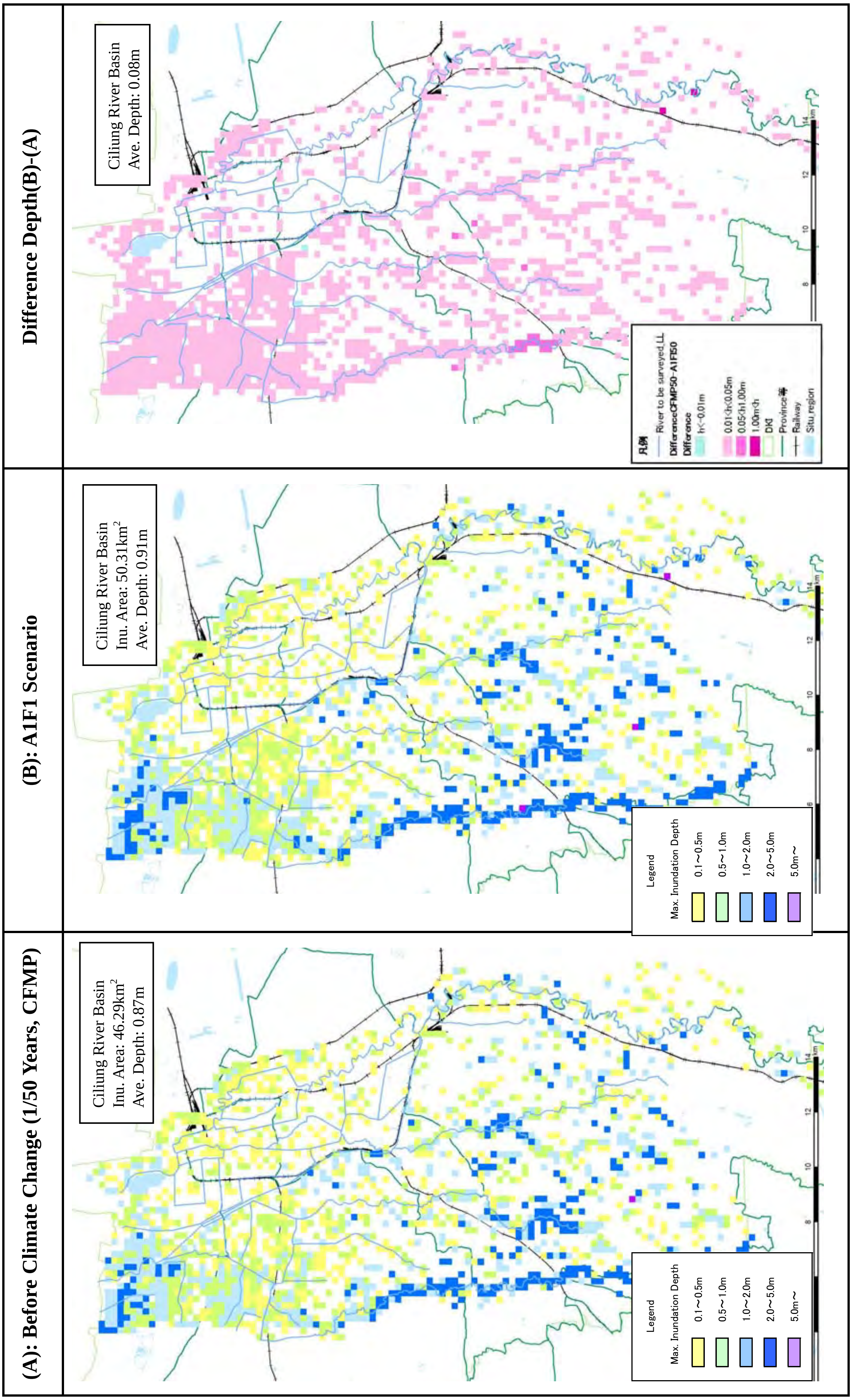


Figure 7.3-11 Comparison of Inundation Conditions between Before Climate Change and A1F1 Scenario

7.3.5 Flood Safety Degree after Climate Change

Reflecting the effect of climate change, basic flood discharge in Ciliwung River Basin will be changed as shown in Table 7.3-8. The target basic flood discharge at Manggarai in the CFMP of 720m^3/s, which is originally 1/50 years flood, is evaluated as 1/25-32 years flood.

Table 7.3-8 Probable Basic Flood Discharges at Manggarai

Return Period of Rainfall	Scenario		
	No Climate Change (Current)	B1	A1FI
100	801	853	891
50	720	775	799
25	655	695	718
10	563	596	615
5	494	517	546

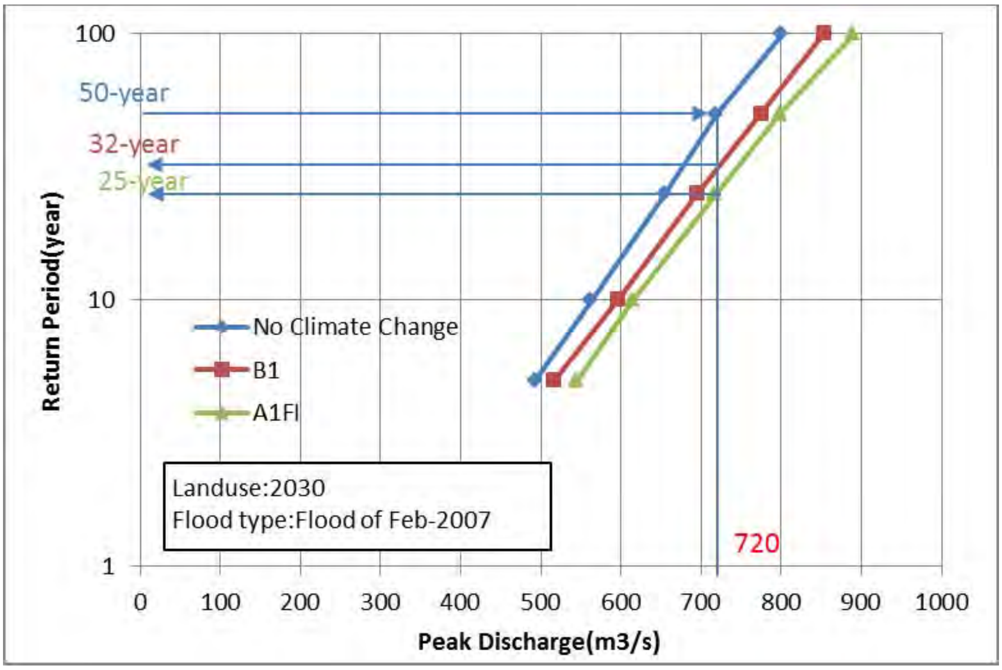


Figure 7.3-12 Probable Basic Flood Discharges at Manggarai

7.4 Adaptation Measures against Excess Floods and Climate Change

The following non-structural measures are recommended as the adaptation measures against excess floods and climate change.

- Improvement of Flood Forecasting and Warning
- Preparation of Hazard Map

(1) Improvement of Flood Forecasting and Warning

Flood warning is conducted by BNPB in Ciliwung River Basin. Since flood disaster occurs by excess flood seven after the CFMP is completed, warning activity shall be continuously conducted. It is noted that warning level shall be reviewed according to the progress of river improvement works.

For accurate and appropriate forecasting and warning, improvement of monitoring system is required.

- Secure monitoring of rainfall and water level and its communications
- Discharge monitoring
- Accumulation and management of monitoring data

(2) Preparation of Hazard Map

For appropriate evacuation and disaster relief activities, hazard map shall be prepared.

CHAPTER 8 REVIEW OF 2013 FLOOD SCALE

In this section, scale of 2013 Floods will be reviewed using Flood Analysis Model which is distributed type, and it will arrange about the subject in the future measure.

8.1 Arrangement of Flood Damages by 2013 Flood

8.1.1 Arrangement of Rainfall Situation

(1) Determination of Amount of Rainfall and Return Period of Rainfall

By arranging the collecting data of 6 (six) observation stations, the amount of rainfall is evaluated as followings and return period of rainfall are evaluated.
- Amount of rainfall for each 24hr and 48hr at each observation stations
- Arrangement of average amount of rainfall in the river basin of 24hr , 48hr and 96hr at Manggarai point, Depok point and Katulampa point

The Location of Observation Stations of Rainfall is as shown in Figure 8.1-1. Collected and sorted data of rainfall, and examined return period of rainfall are as shown in Table 8.1-1 - Figure 8.1-2.

- Point Rainfall

Amount of Rainfall (48hr) reached to 346mm at Bedong Gadog located upstream of Katulampa, and which is evaluated 1/200 return period*. At other points, rainfall amount are evaluated around 1/2 to 1/4 return period.
*Because of lack of rainfall data, return period is evaluated by using that of Citeko point located in the vicinity.

- Average Amount of Rainfall in River Basin

Average Amount of Rainfall in River Basin (48hr) reached to around 146mm at Manggarai point, and which is evaluated 1/4 return period*. At Katulampa point, Average Amount of Rainfall in River Basin (48hr) is evaluated around 1/3 return period, and at Depok point, that is evaluated around 1/4 return period.

Table 8.1-1 Evacuation of Amount and Return Period of Rainfall at each Observation Station

Unit: mm

Station Name	24hr	48hr
Citeko	145.5 (1/3)	205.8 (1/4)
Bend. Gadog*	266.0 (1/250)	346.0 (1/200)
Cibinong	114.5	121.5
Cilember*	119.5 (<1/2)	199.5 (1/4)
Jakarta OBS	236.1 (1/20)	256.0 (1/4)
PONDOK BETUNG CILEDUG	126.2 (1/3)	131.5 (<1/2)

*Estimated by Citeko, the number in parentheses represents return period of rainfall

Table 8.1-2 Evaluation of Average Amount and Return Period of Rainfall in River Basin

Unit: mm

CA	24hr	48hr	96hr
Katulampa	120.3 (<1/2)	197.1 (1/3)	301.0 (1/4)
Depok	106.5 (1/2)	182.0 (1/4)	284.1 (1/6)
Manggarai	90.7 (1/2)	161.6 (1/4)	265.2 (1/7)

*The number in parentheses represents return period of rainfall

(2) Time Distribution of Rainfall Amount

Time distribution of Average Amount of Rainfall in River Basin at each observation stations and main points are analyzed as follows.

Rainfall Hyetograph of each observation stations and main points from Jan 8th to Jan 20th are as shown in Figure 8.1-2, and Time distribution of Average Amount of Rainfall in River Basin are as shown in Figure 8.1-3.

It is suggested that in upstream, it rained from Jan 15th to Jan 16th and in downstream, it rained on Jan 17th.

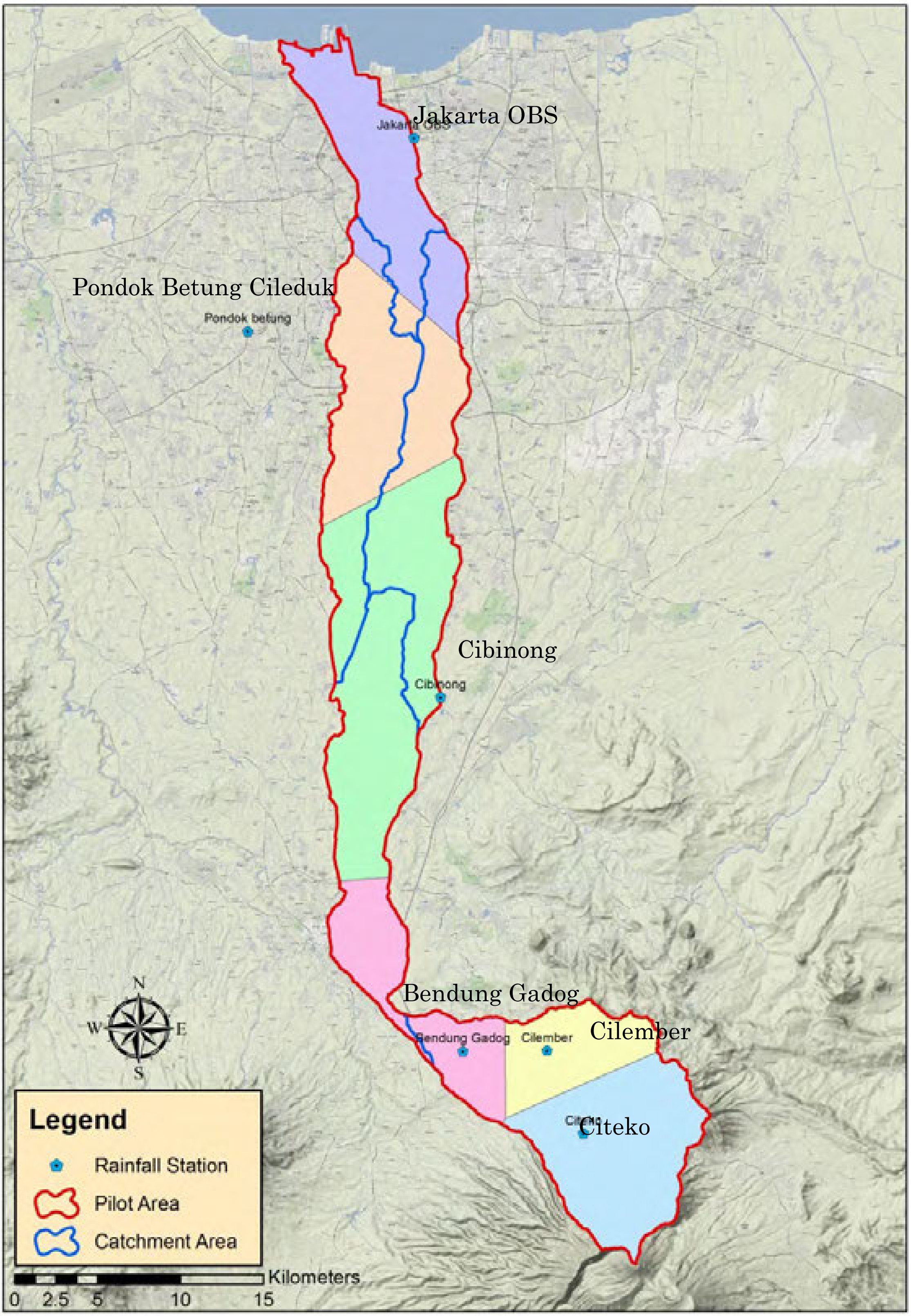


Figure 8.1-1 Location of Observation Stations of Rainfall (Hourly Rainfall)

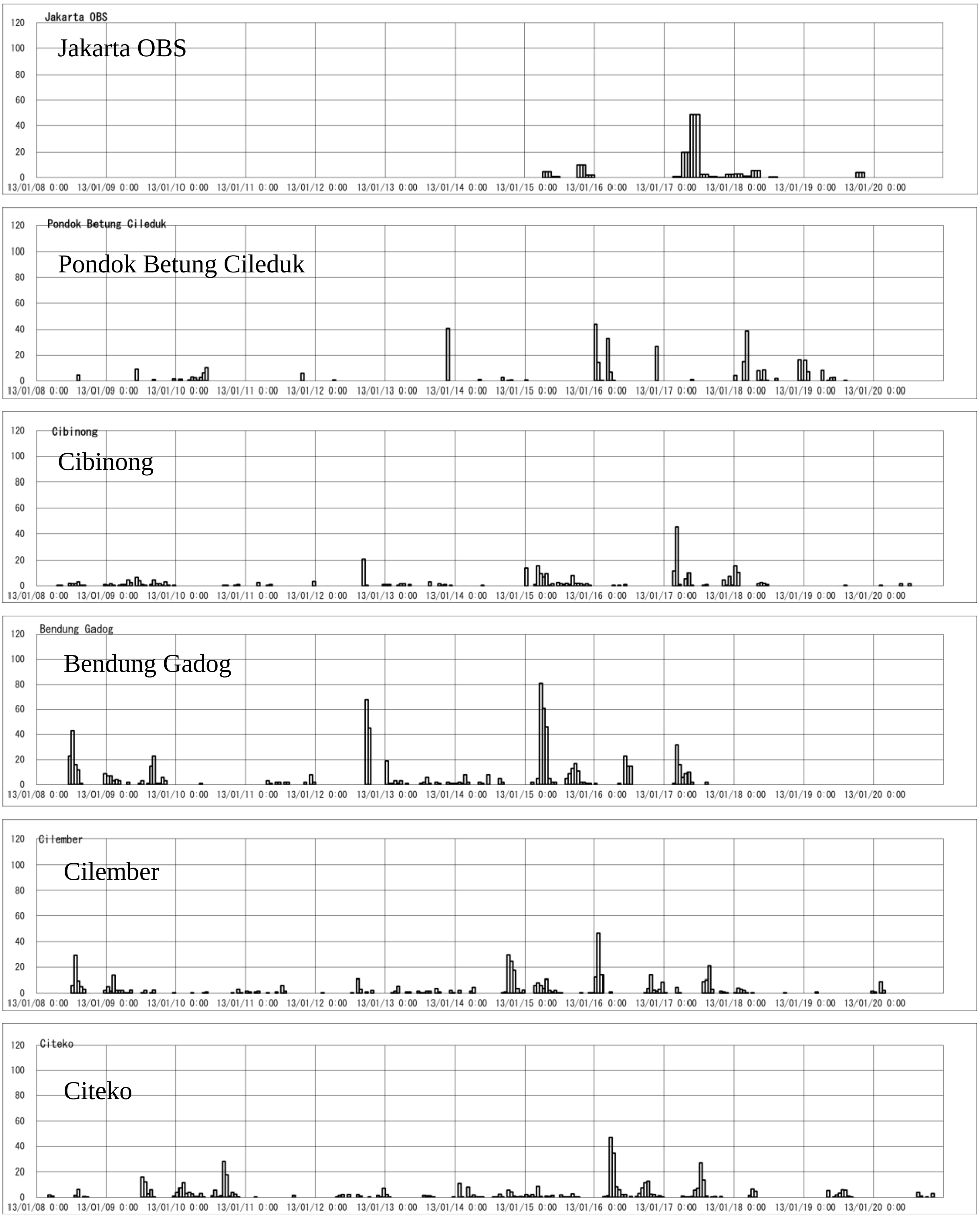


Figure 8.1-2 Time Distribution of Rainfall Amount at each Observation Station

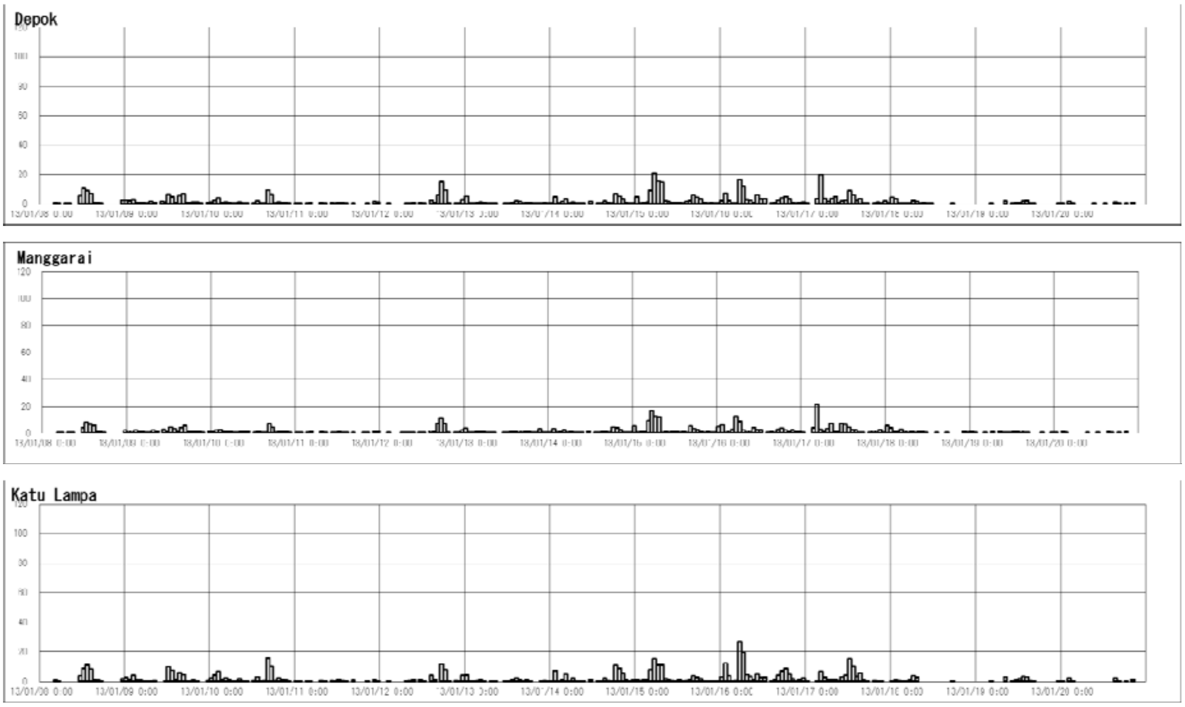


Figure 8.1-3 Time Variation of Average Rainfall Amount in River Basin at main points

(3) Spatial Distribution of Rainfall（Daily Rainfall）

Based on collected rainfall data (daily), Spatial Distribution of rainfall from Jan 15th to Jan 17th is analyzed. Spatial Distribution of rainfall is as shown in Figure 8.1-4, and which is created as an Isohyetal diagram by analyzing the collected rainfall data (daily) of 18 observation stations. Followings are suggested in Figure 8.1-4.

- On Jan 15th, more than 100mm of rain fell in upstream of Katulampa point, then on Jan 17th, more than 200mm of rain fell in downstream of Manggarai point
- On Jan 15th, rainfall in upstream area, and then, rain fell in downstream area up to Jan 17th.
- It is assumed that rain fell in upstream on Jan 15th, which flowed to downstream, and river water level at Manggarai point rose up due to heavy rainfall in downstream and water flow from upstream.

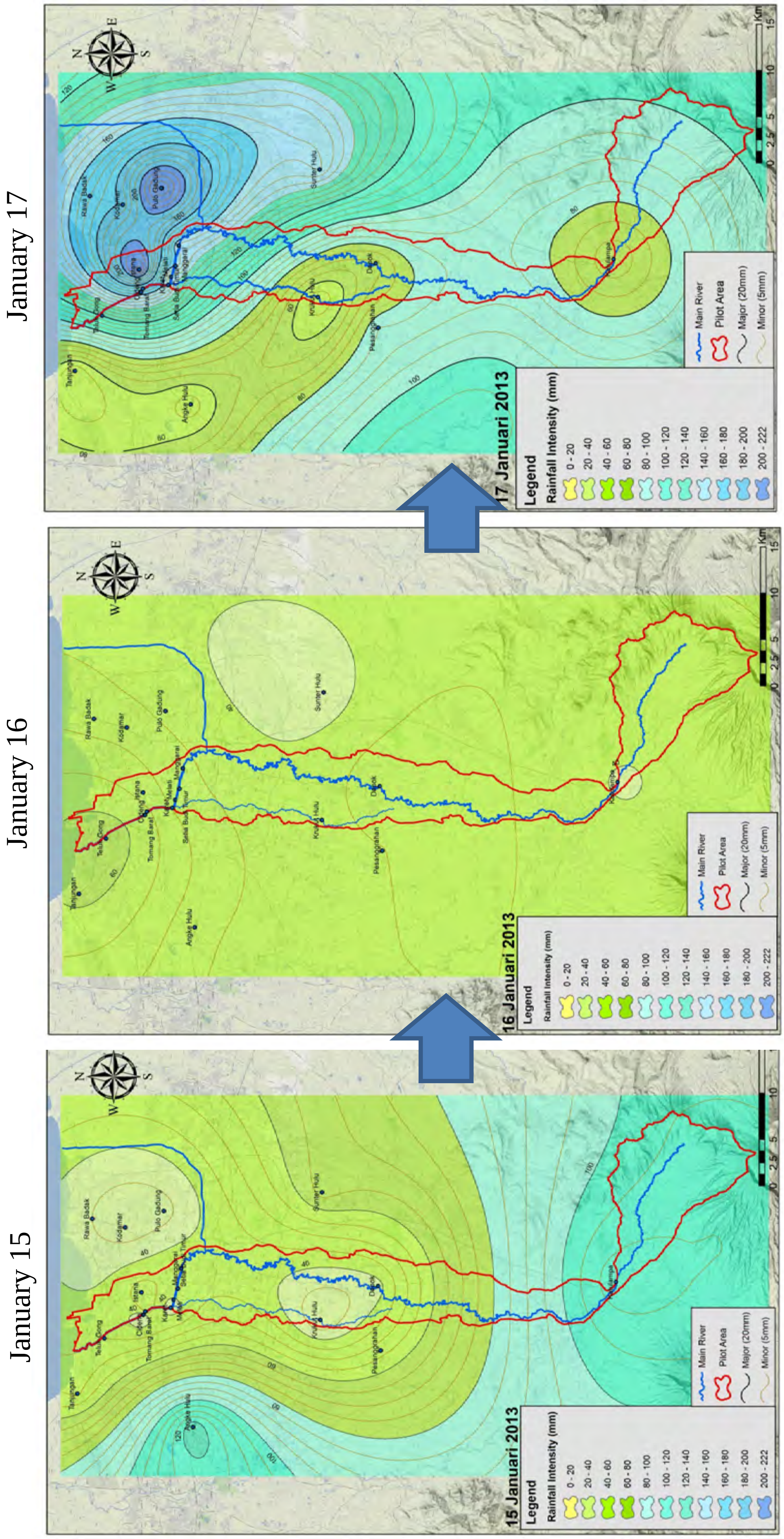


Figure 8.1-4 Spatial Distribution of Rainfall（Daily Rainfall)

8.1.2 Arrangement of Water Level Situation

Recording data of river water level gauging station at Manggarai, Depok and Katulampa from Jan 15 to Jan 20th are as shown in Figure 8.1-5. Recording data are readings of staff gauges.

- At Katulampa point, peak water level recorded 1.95m on January 15th and water level exceeded Siaga3 for around 3 hours.
- At Depok point, peak water level recorded 3.80m on January 15th and water level exceeded Siaga1 for around 3 hours.
- At Manggarai point, peak water level recorded 10.00m on January 17th and water level exceeded Siaga1 for around 4 hours.
- It is assumed that rain fell in upstream on Jan 15th, which flowed to downstream on Jan 16th, and river water level at Manggarai point rose up due to heavy rainfall in downstream and water flow from upstream.

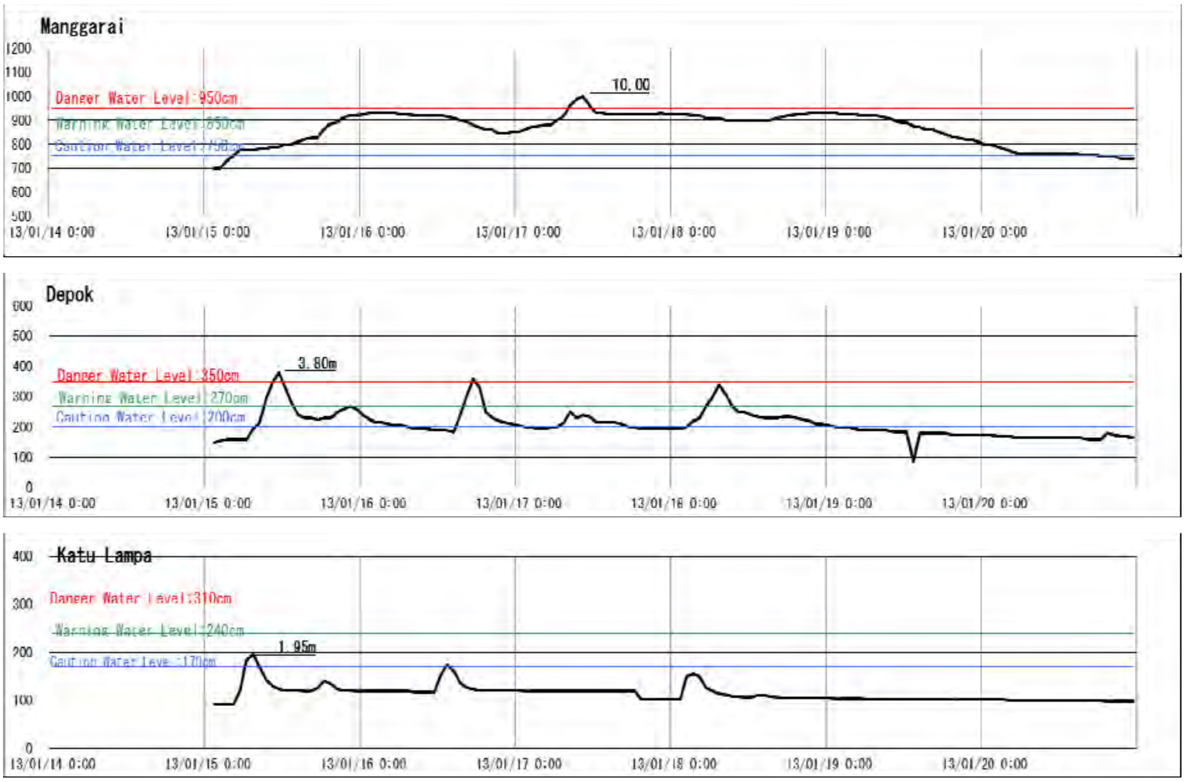


Figure 8.1-5 Water Level Records at main points

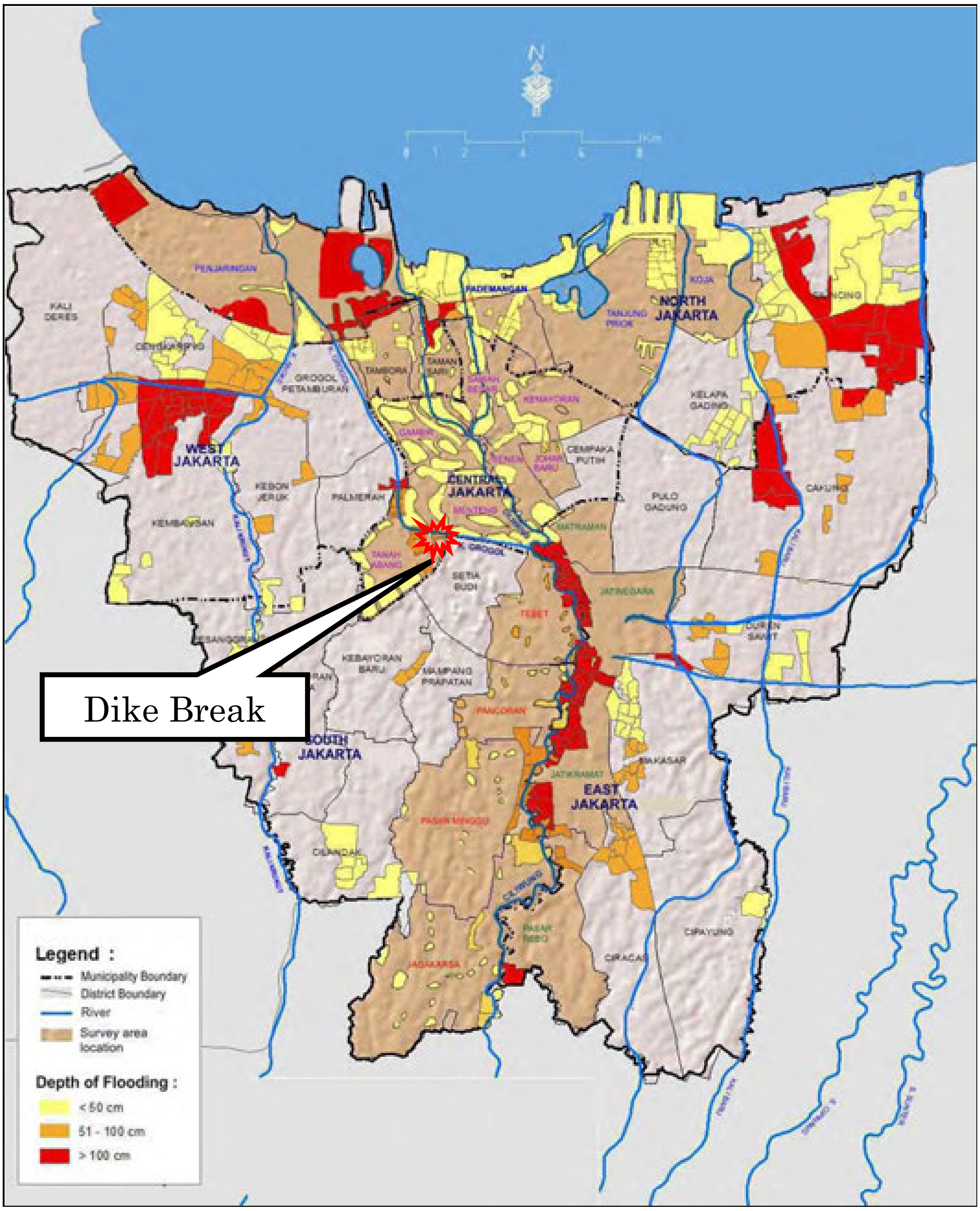
8.1.3 Arrangement of Flood Damages Situation

Inundated Area and Inundation Depth are as shown in Figure 8.1-6. This Inundated Area is one which is created in this project based on field survey after flood.

- In DKI area, Inundated Area is about 140 km^2 (about 21% of DKI area), in the Ciliwung river basin, Inundated Area is about 36.7km^2 (8%), of which upstream area of Manggarai point is about 11.6km^2 (4%)
- It is assumed that, in inland water area on right side of WBC, Dike break occurred in the morning of Jan 17th and flood damage expanded by not only inland water but also flooding water.
- It is assumed that, Inundation damage along the Ciliwung river located upstream of Manggarai point, is due to lack of flow capacity of the Ciliwung river.

Table 8.1-3 Inundation Area

	(A)Inundation Area (km^2)	(B)Total Area (km^2)	Ratio A/B
DKI	140.068	662.33	21.1%
WBC/Ciliwung River Basin	36.701	485.13	7.6%
Manggarai	11.638	337.09	3.5%
Krukut	4.959	84.96	5.8%
Lowland	20.104	63.08	31.9%



Source: Survey Area on 2013 Jakarta Flood Disaster

Figure 8.1-6 Distribution of Inundation Area

Figure 8.1-7 Past Flood Damages Photos

8.2 Verification of Flood Analysis of 2013 Flood

Scale of 2013 Flood is verified with Flood Analysis Model using distributed type.

8.2.1 Conditions of Verification

Conditions for flood verification is shown as below.

Table 8.2-1 Conditions for Flood Analysis

Item	Conditions
Land Use	Present condition
Topographical condition	Present condition
River Channel	Present condition (2011) For WBC, reflecting the riverbed deposition situation of about 1m, which was estimated by field survey after dredging (As-Built Drawings). River channel roughness coefficient of the WBC: n = 0.030.
Drainage Facility	Present condition (2011) Operation of the pump follows an operating rules of the prescribed
Target Flood	2013 Flood Rainfall is distributed by Thiessen method
River Basin	Standard condition

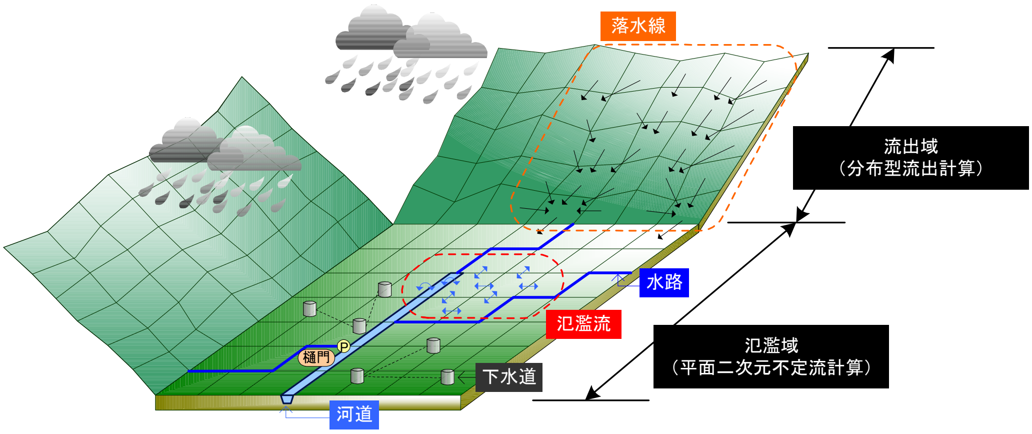


Figure 8.2-1 Image of Flood Analysis Model using Distributed Type

8.2.2 Result of Verification

2013 Flood is analyzed through the comparison with calculation result by Flood Analysis Model using distributed type and measured flow discharge (Conversion value by the HQ formula) at Katulampa point and Depok point, and actual water level at Manggarai point.

(1) River Flow Discharge

Calculation result of river flow discharge at Katulampa point and at Depok is shown as below. Peak flow discharge and form around peak wave are similar to the actual. Flood situation is generally reproduced by Flood Analysis Model using Distributed Type. HQ formula is shown as below.

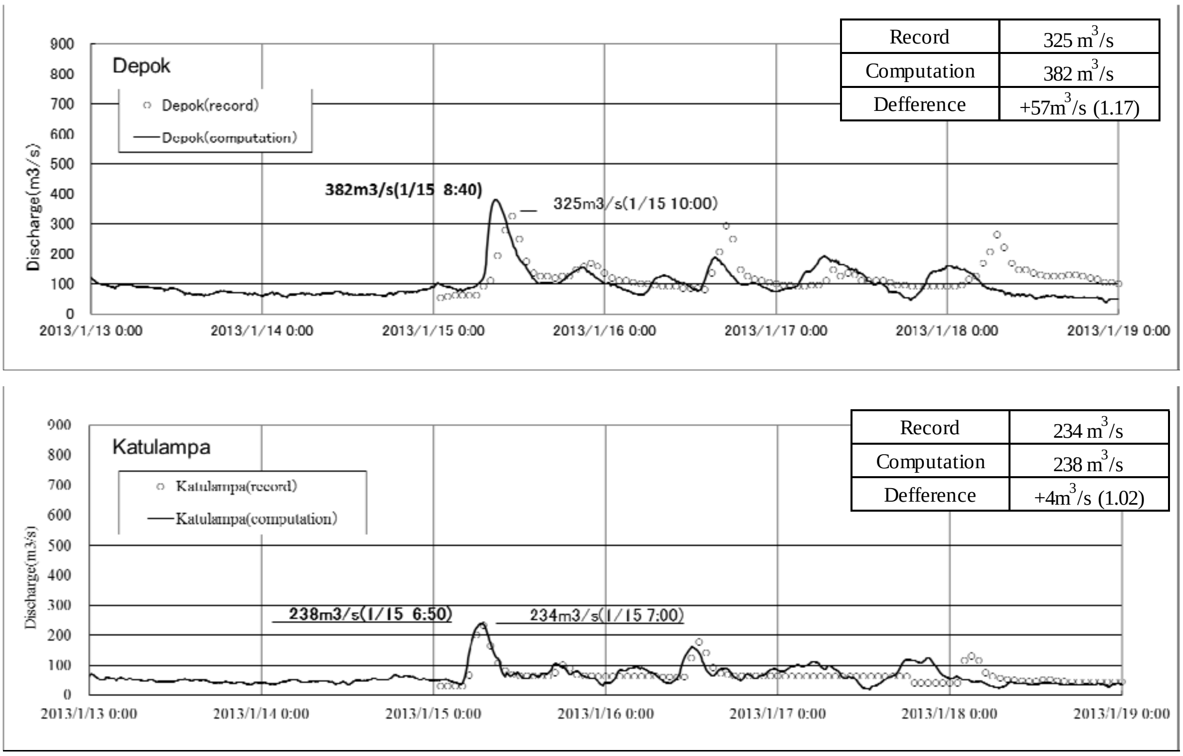


Figure 8.2-2 Discharge Hydrograph (Katulampa, Depok)

H-Q formula

H-Q at Depok: Non-uniform computation (by JFM)

H-Q at Katulampa: Flow computation (by JFM)

Depok WL Station

C.A.= 245.51 km2

H-Q formula by JFM (Dutch Assistance)

Q=31.52*(h-0.160)^1.805

Katulampa WL Station

C.A.= 149.79 km2

H-Q formula by JFM (Dutch Assistance)

for 1.05<h<3.50

Q=76.76*(h-0.27)^2.145

(2) Results of River Water Level

1) Water Level Hydrograph

Calculation result of river water hyetograph at Manggarai point is shown as below. Calculation result of peak water level and measured water level at Manggarai point was different about 1.8m. However, flood situation is highly reproduced by Flood Analysis Model using Distributed Type because form of whole peak wave is similar to the actual.

Meanwhile, cause of the difference in calculation result and the measured water level is assumed due to the following.

- It is assumed that, calculation result is calculated rather lower than measured one because it is impossible to reflect the situation such as sedimentation of the riverbed in downstream section (WBC) and deterioration of flow capacity of gate due to garbage or driftwood caught during flood.

Record	9.316 p.p.m
Computation	7.48 p.p.m
Defference	-1.84m (0.80)

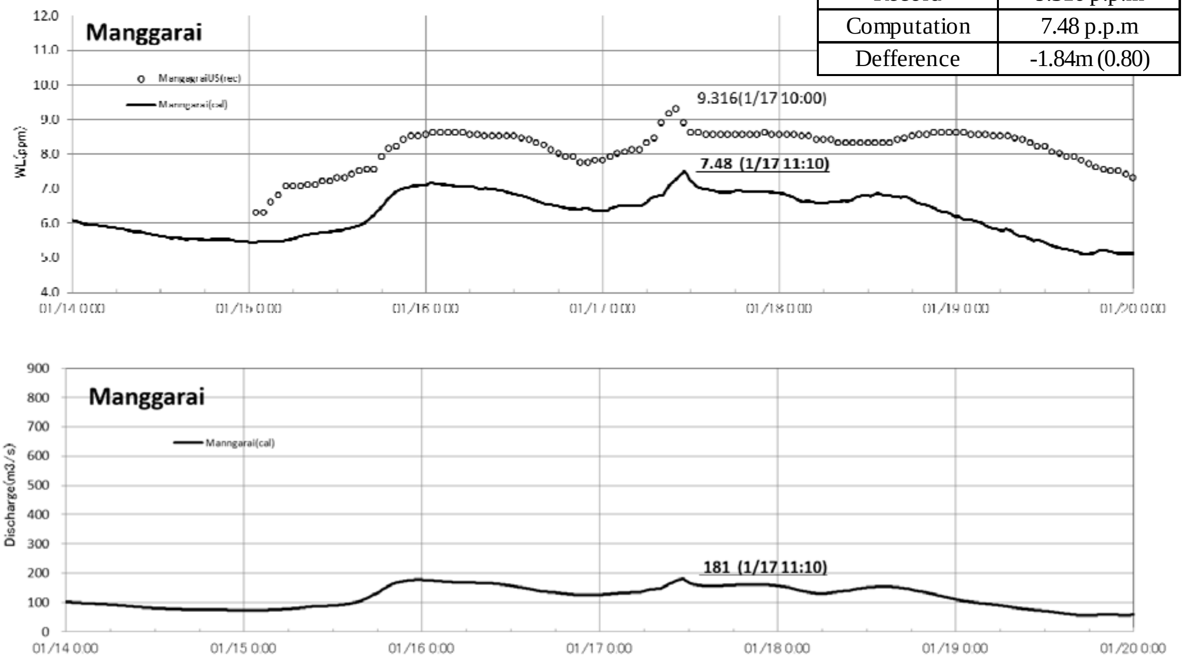


Figure 8.2-3 Water Level Hydrograph (Manggarai)

2) Longitudinal Water Level

Longitudinal water level of WBC is shown in Figure 8.2-4. For WBC, from Karet Gate to Manggarai point, the calculation is reflecting the riverbed deposition situation of about 1m, which was estimated by field survey. In addition, the calculation is reflecting some obstruction of Karet Gate, which information is obtained in the hearing.

- At Karet point, calculation result of water level is around 4.68ppm, and lower than assumed measured water level, “6.84ppm(720) m” by around 2.16m.
- At near the dike break point, calculation result of water level is around 6.08ppm, and lower than assumed actual water level by around 0.7m.
- At Manggarai point, calculation result of water level is lower than measured peak water level, “9.316ppm” by 7.48ppm (around 2.16m).

This is assumed to have produced, when the following factors cannot be reflected well calculative.

- Sedimentation after dredging of WBC has not been able to fully grasp.
- Deterioration of flow capacity of gate due to garbage or driftwood caught during flood has not been able to fully grasp.
- Water level rise around the bridge is not taken into consideration.

(3) Inundated Area

Calculation result of flooding region of the above is as shown on the next page.
For flood area in upstream area of Manggarai point which is not affected by dike break, the difference is about 10% comparing the calculation results with the actual. Therefore, Flood Area seems to be generally reproduced

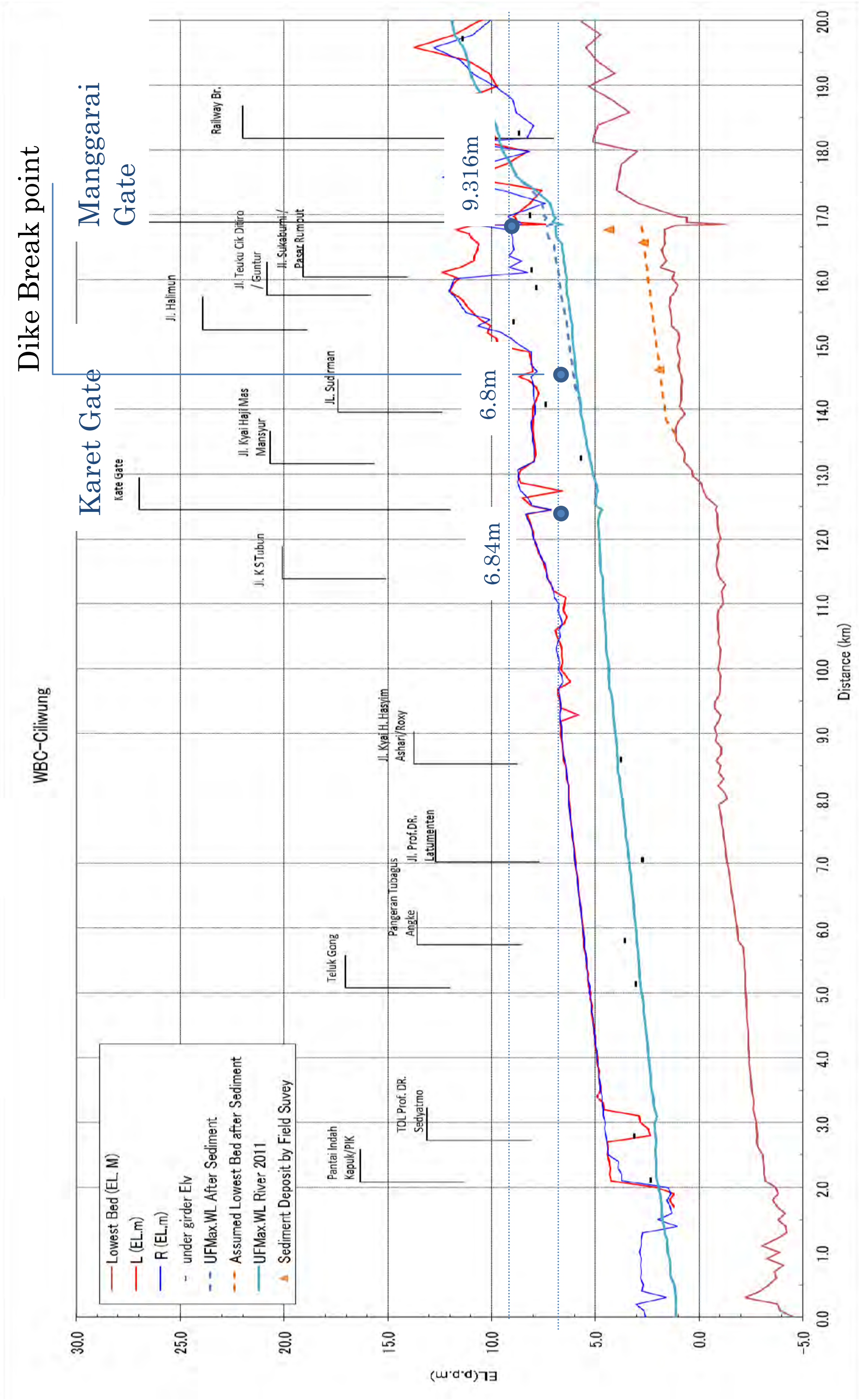
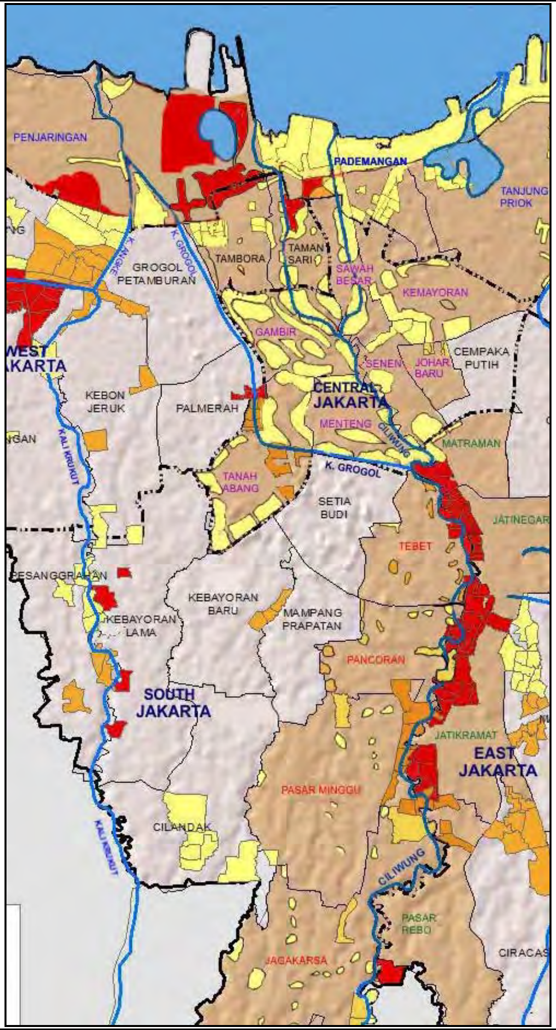
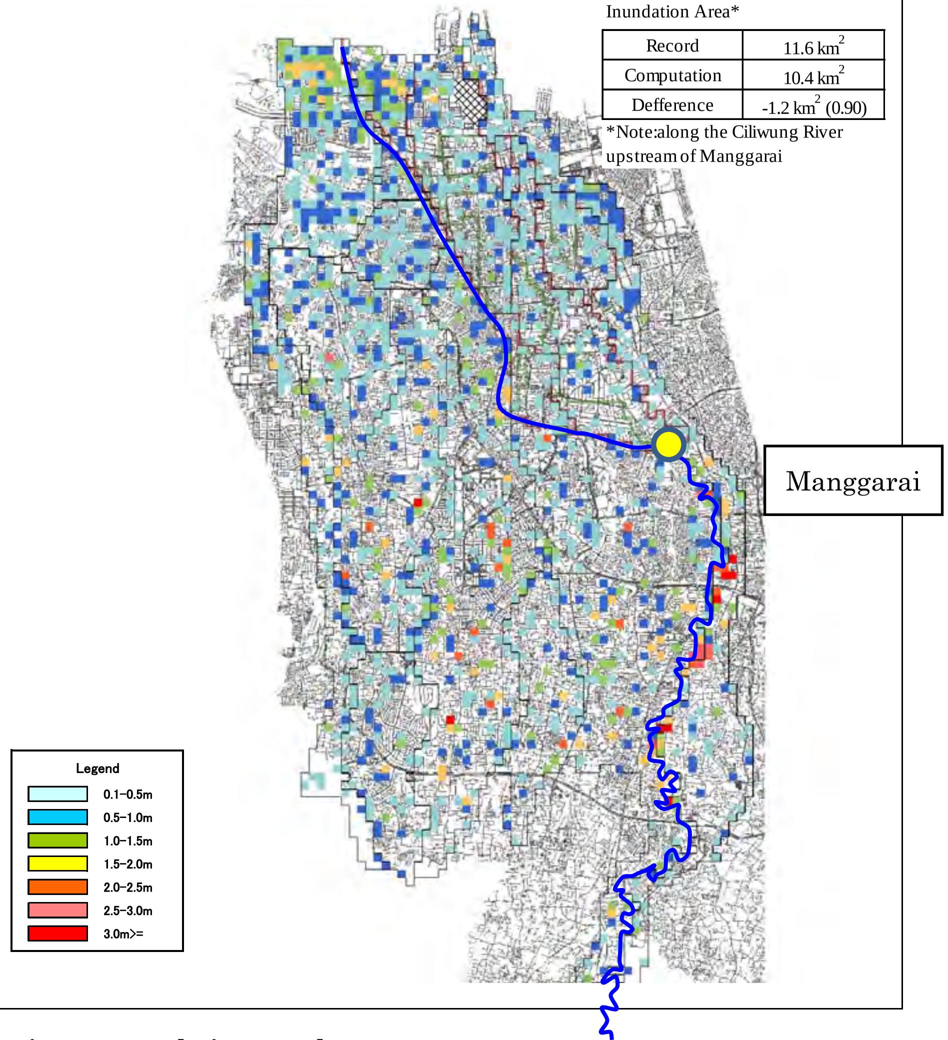


Figure 8.2-4 Longitudinal Section of Maximum Water Level

Record	Verification
	

Inundation Area*

Record	11.6 km^2
Computation	10.4 km^2
Defference	-1.2 km^2 (0.90)

*Note:along the Ciliwung River upstream of Manggarai

Figure 8.2-5 Maximum Inundation Depth

8.2.3 Evaluation of 2013 Flood based on Flood Analysis

Based on the results of evaluation so far, scale of 2013 Flood is evaluated as followings.

(1) Scale of Return Period of Rainfall

- Average Amount of Rainfall (48hr) in River Basin at Manggarai point is evaluated to be around 1/5 year return period.
- Average Amount of Rainfall (48hr) in River Basin at Katulampa point is evaluated to be around 1/200 year return period.

(2) Scale of River Flow Discharge at Manggarai point

- Flow discharge through Manggarai point is around 180m3/s, which is evaluated to be around 1/2 year return period.

8.3 Attribution Analysis of 2013 Flood

Based on examination results of above, attribution analysis result of 2013 Flood is summarized as followings.

■Upstream of Katulampa point
- River Water level at Katulampa point rose up by rainfall in excess of locally 1/200 year return period.
- River water level rose up in downstream area by overlapping with the peak of the rainfall in downstream area and arrival of flood water caused by rainfall in upstream

■Upstream of Manggarai point
- River Water level at Katulampa point rose up by rainfall in excess of locally 1/200 year return period.
- Along the Ciliwung river, flooding occurred due to lack of flow capacity

■Upstream of Manggarai point (Lowland area along the WBC)
- Lack of Flow Capacity is assumed due to sedimentation after dredging in WBC.
- Unexpected overflowing dike break occurred from weak part of dike

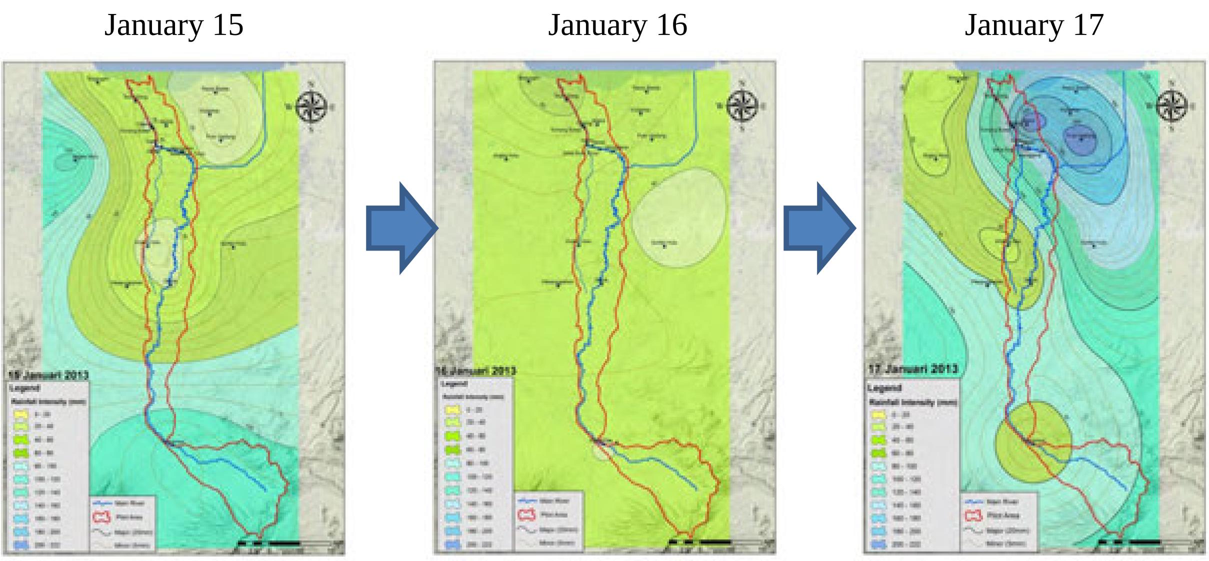


Figure 8.3-1 Isohyetal Map (Daily Rainfall) from Jan 15th to Jan 17th

8.4 Future Issues

Based on the attribution analysis of 2013 Flood, future issues are summarized as below.
Issues of WBC

- To implement Periodically Monitoring (To confirm flow capacity and sedimentation of the river channel, To grasp of driftwood around gate and ensure gate function)
- To keep enough height of dike and elevation of dike crest against flood level (To review Design High Water Level and Existing Dike Elevation)
- To keep quality of material of Dike body
- To heighten bridges to keep clearance against flood level

Downstream of Manggarai

Downstream of Manggarai

Figure 8.4-1 Deposition of Sediment, Garbage in River Channel at Downstream of Manggarai Point

Condition of Manggarai Gate(June25,2013)

Condition of Katet Gate(June25,2013)

Figure 8.4-2 Condition of Existing Manggarai Gate and Kate Gate

Securing the Height of Riverbank

Securing the safety of the embankment

Heightening of the bridges

Figure 8.4-3 Current Situations and Issues