5.5.5 Optimization of River Improvement

Design discharge for the Kampar Kanan River is 4,000 m^3/s as presented in Fig. VI.5.4. Optimization of river improvement for this design discharge has been conducted as follows:

(1) Alignment and Longitudinal Profile

Optimization of alignment and longitudinal profiles was conducted for the 142 km stretch of the Kampar Kanan River. The following three alternatives have been compared. Fig. VI.5.5 shows plans for a representative stretch for each case.

Particulars	Case 1	Case 2	Case 3
Alignment	Creates shortest channel with many shortcuts	Shortcut at heavily meandering parts	Shortcut only at extremely meandering parts
Longitudinal Profile	1/1,350 - 1/5,500 Ave.: 1/2,350	1/1,400 - 1/6,000 Ave.: 1/2,820	1/1,500 - 1/8,000 Ave.: 1/3,800
River Length	100 km	113 km	142 km
Shortcut Length	48.5 km	24.8 km	10.3 km
Flow Velocity	1.8 - 2.8 m/s	1.6 - 2.5 m/s	1.4 - 2.2 m/s
Direct Construction Cost	Rp. 650×10^9	Rp. 543×10^9	Rp. 549×10^9

Note: Channel with embankment cross section of 300 m wide (upstream) and 400 m wide (midstream) and 600 m wide (downstream) is applied.

Case 2 is finally selected based on the table above considering the following reasons:

- Case 2 requires the minimum construction cost as shown in the above table and in Fig. VI.5.6.
- In view of the stability of the river, the case that can maintain the present condition as much as possible is preferable. If shortcuts are applied only at extreme meandering points (Case 3), however, the flow capacity is small and the cost is comparatively high.
- In Case 2, the river profile will be steepened to 1/2,820 in average. Even in this case, an average flow velocity is maintained below 2 m/s.
- Case 1 requires the highest cost, and longitudinal profile becomes very steep at 1/2,350 in average and flow velocity reaches 3 m/s. It is not preferable to river morphology.

(2) Cross Sections

The following four alternatives have been compared (refer to Fig. VI.5.7). The sample stretch is of 41.7 km in the Kampar Kanan River from the

68.1 km point to the 109.8 km point from the confluence with the Kampar Kiri River.

Proposed longitudinal gradient in this stretch is 1/4,000 and the design discharge is 4,000 m^3/s.

	Case 1	Case 2	Case 3	Case 4
Description	Assure flow capacity mainly by excavation. Embankment is considered only for the extremely low banks.	Assure flow capacity mainly by embankment. Narrow channel will be created. Excavation is considered only for the extremely narrowed sections.	Assure flow capacity mainly by embankment. Channel of moderate width will be created. Excavation is considered only for the extremely narrowed sections.	Assure flow capacity mainly by embankment. Wide channel will be created. Excavation is considered only for the extremely narrowed sections.
River Width (m)	380	200	300	600
Height of High Water Level from present land (m)	- 0.5	3.7	2.6	2.2
Excavation (1,000 m^3)	48,000	1,500	1,500	1,500
Embankment (1,000 m^3)	500	9,340	6,420	4,740
Land Acquisition (ha)	960	570	940	2,130
Cost (Rp. 10^9)	398	136	114	119

Case 3 is finally selected based on the table above, considering the following reasons:

- Case 3 requires the minimum construction cost.
- It is preferable to maintain the high water level as low as possible because a higher water level will give potential risks in case of bank collapse. In this sense, Case 1 is preferable. However, the cost of Case 1 is extremely high since it requires much excavation.
- Land to be acquired is smallest in Case 2. However, high water level is very high.
- Rise of high water level from Case 4 to Case 3 is not so large compared to Case 3 to Case 2.
- Since the difference of cost in the case of 300 m width and 600 m width is small, the widths of 400 m and 600 m are applied for the downstream part of the Kampar Kanan River and the Kampar River, respectively, considering the present land use along the river channel.

- One side bank is introduced for the Kampar Kiri River and for the Kampar River in the downstream part from Kerinci in due consideration of the topographic conditions.

5.5.6 Optimum Flood Control Plan

As a result of the above optimization, the optimum flood control plan for the Kampar Kanan River has been established. Plan, proposed longitudinal profiles and typical cross sections are shown in Figs. VI.5.8 to VI.5.10, respectively.

5.6 Optimization of Flood Control Structure for Kampar and Kampar Kiri Rivers

The optimization of flood control structures for the Kampar and Kampar Kiri Rivers is presented in this section.

As discussed in applicable alternative measures, the following structures are applicable for the flood control of the Kampar and Kampar Kiri Rivers.

- Kampar Kiri No.1 Multipurpose Dam
- Kampar Kiri No.2 Multipurpose Dam
- Retarding Basin in the downstream stretch of the Kampar Kiri River
- River improvement of the Kampar and Kampar Kiri Rivers

5.6.1 Major Characteristics for Flood Control Planning

Present conditions of the Kampar and Kampar Kiri rivers are described in CHAPTER 1. Characteristics related to flood control planning are summarized as follows:

- Flow capacity of the present channel of the Kampar Kiri River is about 500 m^3/s in the upstream stretch from around Kampung Dalam. It is much smaller at around 200 m^3/s in the downstream stretch from the same point to the confluence with the Kampar Kanan River.
- The flood inundation area, namely, the area to be protected, is long and narrow along the river. The width of the area is about 5 to 10 km.
- The rivers, except the Kampar River in the downstream stretch from Kerinci, form heavily meandering channels. The width of the meandering is about 1.5 to 2 km. This means that if the improved channel is formed to comprehend the meandering channel, many portions of the area to be protected will be included in the river channel.
- The development of meandering is not so rapid.

Optimization study is conducted as described below on the basis of the above mentioned major characteristics.

5.6.2 Dam

Possible dams for flood control in the basin are two, namely, Kampar Kiri No.1 on the Sibayang River and Kampar Kiri No. 2 on the Singingi River. These are in the upstream basin of the Kampar Kiri River. Hydropower generation is also considered at these two dams, the optimization of dam and river improvement combination has been accordingly conducted in SECTOR XI, MULTIPURPOSE DEVELOPMENT PLAN.

The capacity allocation for Kampar Kiri No.1 and No.2 dams has been determined as presented as follows:

Unit: $10^6 m^3$

Capacity	Kampar Kiri No. 1 Dam	Kampar Kiri No. 2 Dam
Flood Control	250	150
Hydropower Generation	646	438
Dead Storage	1,350	1,612
Gross Storage	2,246	2,200

5.6.3 Retarding Basin

The river improvement plan is to be established based on the future land use plan proposed in REPELITA VI. In accordance with the land use plan of REPELITA VI, the area along the Kampar Kiri River in the stretch upstream from the confluence with the Kampar Kanan River is designated as forest area.

As discussed under the characteristics for flood control planning, the flow capacity of the present channel in this section is very small and this area is always inundated. Accordingly, this area can be considered as a retarding basin. The following table compares the design discharge with and without this retarding basin.

Stretch	Design Discharge (m^3/s)	
	With Retarding Basin	Without Retarding Basin
Langgam - Kerinci	4,850	5,450
Kerinci - downstream	5,100	5,700

Note: Design discharges consider Kampar Kiri No. 1 and No. 2 dams.

Since the future land use in this area is forest, no benefit will be obtained even if the area is not considered as a retarding basin and is protected from flood. Accordingly, this retarding basin has been taken into consideration.

5.6.4 Floodway

There are no realistic sites for floodways in the Kampar and Kampar Kiri river areas.

5.6.5 Design Discharge Distribution

As discussed in the previous sub-sections, Kampar Kiri No.1 and No.2 dams and Kampar Kiri Retarding Basin together with river improvement of Kampar and Kampar Kiri rivers have been considered. The design discharge distribution for the Kampar and Kampar Kiri rivers has been accordingly determined based on the standard flood discharge for 50-year return period as presented in Fig. VI.5.4 together with those for the Kampar Kanan River.

5.6.6 Optimization of River Improvement

The lower reaches of the Kampar Kiri River is designated as Kampar Kiri Retarding Basin. Accordingly, the following stretches have been considered for river improvement.

- Sibayang River
- Singingi River
- Kampar Kiri River
- Kampar River

The Sibayang River is located in the downstream stretch of Kampar Kiri No.1 Dam. In accordance with the design discharge distribution, the design discharge for the Sibayang River is 500 m^3/s. The flow capacity of the present channel of the Sibayang River is larger than 500 m^3/s. No improvement is required for this stretch.

The design discharge for the lower reaches of the Singingi River is 1,200 m^3/s. No improvement has been considered for the Singingi River since the area is designated as the forest areas in the future land use.

The necessary improvement stretch of the Kampar Kiri River is from Kampung Dalam in the downstream end to Lipat Kain in the upstream end. As discussed for the Kampar Kanan River, improvement mainly by excavation is costly and thus embankment has been considered. In accordance with cross sections and topographical maps, the right bank area in this stretch consists of relatively high natural levees. One side bank (left bank) has thus been considered for this stretch. Design high water level and crest level have been determined based on non-uniform calculation.

(1) Alignment and Longitudinal Profiles

On the basis of the optimization as conducted for the Kampar River (refer to Subsection 5.5.6), alignment and longitudinal profiles are determined as follows:

(a) Kampar Kiri River

- Alignment: The present alignment has been basically maintained since wider channel can be considered by application of one side bank.
- Longitudinal Profile: Basically follow the present profile.

(b) Kampar River

- Alignment: To create smoother channels by shortcut at heavily meandering points
- Longitudinal Profile: Determined on the basis of the new alignment as follows: 1/5,500 - 1/17,000 (Average: 1/11,000).

(2) Cross Sections

On the basis of the optimization as conducted for the Kampar River, the following cross sections are selected:

(a) Kampar Kiri River

- Lipat Kain - Kampung Dalam : Left bank only

(b) Kampar River

- Langgam - Kerinci : B=600 m
- Kerinci - downstream : Right bank only

5.6.7 Optimum Flood Control Plan

The optimum flood control plan for the Kampar and Kampar Kiri rivers is established as a result of the above optimization. Plan, proposed longitudinal profiles and typical cross sections are shown in Figs. VI.5.8 to VI.5.10, respectively.

5.7 Optimization of Flood Control Structure for Middle and Lower Reaches of Indragiri River

The optimization of flood control structure for the middle and lower reaches of the Indragiri River is described below.

5.7.1 Major Characteristics for Flood Control Planning

The present conditions of the middle and lower reaches of the Indragiri River, the Kuantan-Indragiri River, are described in CHAPTER 1. Characteristics related to flood control planning are summarized as follows:

- Flow capacity of the present channel of the Kuantan-Indragiri River is about 1,000-1,500 m^3/s. This flow capacity is less than the 2-year return period value and floods occur almost every year.
- The flood inundation area, namely, the area to be protected along the river, is long and narrow. This implies river improvement is costly and flood control dam is generally advantageous.
- The rivers form heavily meandering channels especially in the upstream area from Japura. The width of the meandering is about 2 to 3 km. This means that if the improved channel is formed to minimize the meandering channel, many portions of the area to be protected will be included in the river channel.
- Development of the meandering is not so fast.

Optimization study is conducted as described below based on the major characteristics mentioned above.

5.7.2 Dam

Possible dams for flood control of the Kuantan-Indragiri River are the Upper Sinamar, the Sukam and the Kuantan. The study revealed that flood control by the Upper Sinamar and Sukam dams is not advantageous because the peak cut effect is small.

The Kuantan Dam is studied for optimization in combination with the river improvement in the downstream stretches. Since the Kuantan Dam is planned as a multipurpose dam, the optimization study is conducted in SECTOR XI, MULTIPURPOSE DEVELOPMENT PLAN. As a result, the flood control capacity of 400×10^6 m^3 is allocated to the Kuantan Dam.

5.7.3 Retarding Basin

The river improvement plan is established based on the future land use plan proposed in REPELITA VI. In accordance with the land use plan of REPELITA VI, the left bank area along the Indragiri River in the stretch downstream from Japura is designated as forest area and non-designated area. Accordingly, this area can be considered as a retarding basin.

The following table compares the design discharge with and without this retarding basin.

Stretch	Design Discharge (m^3/s)	
	With Retarding Basin	Without Retarding Basin
Japura - Downstream	5,000	5,500

Note: Design discharges consider Kuantan Dam.

Since the future land use in this area is forest, no benefit is obtained even if the area is not considered as a retarding basin and protected from flood. Accordingly, this retarding basin is taken into consideration.

5.7.4 Floodway

As discussed in Section 3.2, the floodway that diverts a maximum discharge of 500 m^3/s from the Indragiri River to the Gaung River is considered for the present planning.

5.7.5 Design Discharge Distribution

As discussed in the previous sub-sections, Kuantan Dam and Indragiri Retarding Basin together with river improvement of Kuantan-Indragiri River have been considered. The design discharge distribution for the Kuantan-Indragiri River has been accordingly determined based on the standard flood discharge for 50-year return period as presented in Fig. VI.5.11.

5.7.6 Optimization of River Improvement

Optimization of alignment, longitudinal profiles and cross sections for the Kuantan-Indragiri river improvement have been conducted in this subsection.

(1) Alignment and Longitudinal Profiles

On the basis of the optimization as conducted for the Kampar River (refer to Subsection 5.5.6), alignment and longitudinal profiles are determined as follows:

- Alignment: To create smoother channels by shortcut at heavily meandering parts
- Longitudinal Profile: Determined on the basis of the new alignment as follows: 1/2,400 - 1/5,000 (Average: 1/3,400).

(2) Cross Sections

On the basis of the optimization as conducted for the Kampar River, the following cross sections are selected:

- Lubukjambi - Peranap : B=300 m
- Peranap - Japura : B=600 m
- Japura - Downstream : Right bank only (Left bank is proposed as a retarding basin.)

5.7.7 Optimum Flood Control Plan

As a result of the above optimization considering the results of optimization for multipurpose structures as presented in SECTOR XI, the optimum flood control plan for the middle and lower reaches of the Kuantan-Indragiri rivers has been established. Plan, proposed longitudinal profiles and typical cross sections are shown in Figs. VI.5.12 to VI.5.14, respectively.

5.8 Optimization of Flood Control Structure for Payakumbuh Area

The following rivers have been studied in Payakumbuh Area:

- Sinamar River
- Agam River
- Lampasi River

5.8.1 Major Characteristics for Flood Control Planning

The present conditions of rivers of Payakumbuh Area in the upper reaches of the Indragiri River are described in Section 1.3. Characteristics related to flood control planning are summarized as follows:

- Although the rivers are in the upper reaches, river gradients are generally gentle because they are in the flat valley.
- Flow capacity of the present channel is generally less than the 2-year return period and floods occur almost every year.
- The flood inundation area, namely, the area to be protected, is composed of agricultural lands and urban areas. Lands are extensively used compared to the areas in the lower reaches. This implies that a narrower channel is beneficial in this area.
- Meandering is not so heavy compared to the lower reaches.

5.8.2 Alternative Measures

In the Sinamar, Agam and Lampasi river basins, there are no realistic sites for dam, retarding basin and floodway construction.

5.8.3 Design Discharge Distribution

As discussed in the previous sub-sections, dams, retarding basins and floodways have not been proposed for flood control of Payakumbuh area. Accordingly, the design discharge distribution for the Sinamar, Lampasi and Agam rivers has been determined based on the standard flood discharge for 50-year return period as presented in Fig. VI.5.15.

5.8.4 Optimization of River Improvement

(1) Alignment and Longitudinal Profiles

After the optimization study, the alignment is determined by basically maintaining the present channel and applying shortcuts at only the extreme meandering portions. The proposed longitudinal gradients are determined as follows:

- Sinamar River : 1/450 - 1/2,050
- Agam River : 1/1,000
- Lampasi River : 1/600

(2) Cross Section

The following three alternatives have been compared as presented below. The sample stretch is of 21.3 km in the Sinamar River.

The proposed longitudinal gradient in this stretch is 1/1,000 and the design discharge is 950 m^3/s.

Particulars	Case 1	Case 2	Case 3
Description	Assure flow capacity mainly by excavation. Embankment is considered only for the extremely low banks.	Assure flow capacity by a combination of excavation and embankment.	Assure flow capacity mainly by embankment. Excavation is considered only for extremely narrowed sections
River Width (m)	140	107	93
Height of High Water Level from present land (m)	0	1.3	3.1
Excavation (1,000 m^3)	10,500	2,900	500
Embankment (1,000 m^3)	100	950	2,500
Land Acquisition (ha)	240	170	140
Cost(Rp. 10^9)	87	37	38

Case 2 is finally selected based on the table above and considering the following reasons:

- Case 2 requires the minimum construction cost.
- It is preferable to maintain the high water level as low as possible because a higher water level will give potential risks in case of bank

collapse. In this sense, Case 1 is preferable. However, the cost of Case 1 is extremely high because it requires much excavation.

- The required width is large in Case 1. Case 2 and Case 3 are much smaller.
- The high water level of 1.3 m from the ground in Case 2 is in the allowable extent considering the historical inundation level.
- Although the difference of costs in Case 2 and Case 3 is small, high water level is as high as 3.1 m in Case 3 and this is not preferable.

5.8.5 Optimum Flood Control Plan

As a result of the above optimization, the optimum flood control plan for rivers in the Payakumbuh Area is established. Plan, proposed longitudinal profiles and typical cross sections are shown in Figs. VI.5.16 to VI.5.18, respectively.

5.9 Optimization of Flood Control Structure for Solok Area

The following rivers are studied in Solok Area:

- Lembang River
- Sumani River

5.9.1 Major Characteristics for Flood Control Planning

The present conditions of rivers in the upper reaches of Solok Area are described in Section 1.3. Characteristics related to flood control planning are summarized as follows:

- Although the rivers are in the upper reaches, river gradients are generally gentle because they are in the flat valley.
- Flow capacity of the present channel near Solok City is small at about 400 m^3/s. This corresponds to about 3 to 4-year return period.
- In the Lembang River, there is a longitudinal bottleneck at about 11 km from Singkarak Lake. This seems to be causing a rise in riverbeds resulting in small flow capacity in the upstream reaches.
- The flood inundation area, namely, the area to be protected, is composed of urban areas (Solok City) and agricultural lands. Lands are extensively used compared to the areas in the lower reaches. This implies that a narrower channel is beneficial in this area.

5.9.2 Floodway

The possible floodway in Solok Area is as shown in Fig. VI.5.19. After the cost comparison, the floodway is not adopted because improvement of the present channel is much more advantageous, as follows:

Particulars	Direct Construction Cost for Lembang River
With Floodway	Rp. 52×10^9
With Improvement of Present Channel	Rp. 29×10^9

5.9.3 Other Alternative Measures

In the Lembang and Sumani river basins, there are no realistic sites for dam and retarding basin construction.

5.9.4 Design Discharge Distribution

As discussed in preceding subsections, dams, retarding basins and floodways were not proposed for flood control of the Lembang and Sumani rivers. Accordingly, the design discharge distribution for the Kampar Kanan River were determined based on the standard flood discharge for 50-year return period as presented in Fig. VI.5.15.

5.9.5 Optimization of River Improvement

(1) Alignment and Longitudinal Profiles

After the optimization study, the alignment is basically determined by maintaining the present channel and applying shortcuts at only the extreme meandering portions. The proposed longitudinal gradients are given as follows:

- Lembang River : 1/140 - 1/1,480
- Sumani River : 1/800

(2) Cross Sections

The excavated channel is applied to the Lembang River for the upstream stretch from the 11 km point from Singkarak Lake since it is physically and economically advantageous. For the stretch downstream from the same point, normal section with embankment and channelization as determined for the Sinamar River is applied.

5.9.6 Optimum Flood Control Plan

As a result of the above optimization, the optimum flood control plan of the Lembang and Sumani rivers is established. Plan, proposed longitudinal profiles and typical cross sections are shown in Figs. VI.5.16 to VI.5.18, respectively.

5.10 Optimization of Flood Control Structure for Sijunjung/Muara Area

The following rivers are studied in Sijunjung/Muara Area:

- Sukam River
- Palangki River
- Kuantan River

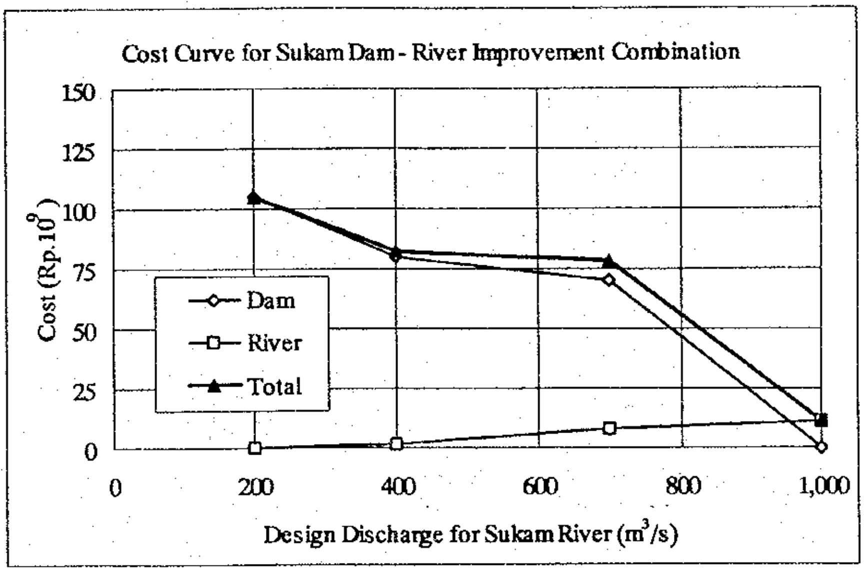
5.10.1 Major Characteristics for Flood Control Planning

The present conditions of rivers in Sijunjung/Muara Area are described in Section 1.3. Characteristics related to flood control planning are summarized as follows:

- Flow capacity of the present channel of the Sukam River is generally less than the 2-year return period and floods occur almost every year.
- The flood inundation area, namely, the area to be protected, is composed of agricultural lands and urban areas. Lands are extensively used compared to the areas in the lower reaches. This implies that a narrower channel is beneficial in this area.
- Meandering is not so heavy compared to the lower reaches.
- Inundation of the area in the south of Muara Town is due to backwater by a bottleneck in the Kuantan River.

5.10.2 Dam

The possible damsite in this area is the Sukam damsite for the flood control in the Sukam River in Sijunjung Area. The optimization is conducted comparing the combination of the cost for the dam and downstream river improvement. It is finally concluded that the Sukam Dam is not considered for flood control in the Sukam River because it is much more costly compared with the river improvement, as shown below:



5.10.3 Other Alternative Measures

There are no realistic sites for retarding basin and floodway in Sijunjung/Muara Area.

5.10.4 Design Discharge Distribution

As discussed in the previous sub-sections, dams, retarding basins and floodways have not been proposed for flood control of Sijunjung/Muara Area. Accordingly, the design discharge distribution has been determined based on the standard flood discharge for 50-year return period as presented in Fig. VI.5.15.

5.10.5 Optimization of River Improvement

As discussed in Subsection 5.10.1, the cause of the flooding problems in Sijunjung/Muara Area can be categorized into two:

- Poor flow capacity of Main Stream: Floods occur due to the poor flow capacity of the present channel of the Sukam and Palangki rivers.
- Backwater by Kuantan River: Inundation in the lower reaches of the Sukam and Palangki rivers due to the backwater from the Kuantan River. This inundation takes place when the flood occur in the Ombilin-Kuantan River.

River improvement plan has been optimized by the above cause as presented below:

(1) Poor Flow Capacity of Main Stream

Alignment, longitudinal profiles and cross sections have been determined following the optimization result conducted for Payakumbuh Area since the conditions are similar.

(a) Alignment and Longitudinal Profiles

The alignment is basically determined by maintaining the present channel and applying shortcuts at only the extreme meandering portions. The proposed longitudinal gradients are given as follows:

- Sukam River : 1/650 - 1/1,400
- Palangki River : approximately 1/1,500

(b) Cross Sections

Normal section with embankment and channelization of low water channel as determined for the Sinamar River is applied.

(2) Backwater by Kuantan River

(a) Present Condition

The plan, longitudinal profile and representative cross sections of the Kuantan River in the downstream stretch of Muara Town are presented in Fig. VI.5.20 to VI.5.22. The longitudinal profile (refer to Fig. VI.5.21) also shows water levels when the design discharge occur.

The water levels at Muara Town during floods are as presented as follows:

Return Period of Flood	Discharge (m^3/s)	Water Level at Muara Town (EL m)
10-year	3,950	146.9
50-year	5,450	149.0

(b) Countermeasures

To cope with the inundation the following countermeasures are conceived, namely, excavation of the Kuantan River and backwater levee in the Sukam and Palangki rivers.

Backwater levees for the Sukam and Palangki rivers have been finally selected in due consideration of the following comparison:

Particulars	Excavation of Kuantan River	Backwater Levee of Sukam and Palangki rivers
Outline of Works	Channelize the Kuantan River for the stretch of 11.3 km to create smoother and steeper channel.	Construct backwater levee in the Sukam and Palangki rivers
Construction Cost (Rp. 10^9)	205	78
Remarks	Rock excavation works is in steep mountainous areas and access to the site is very difficult. After excavation, slope collapse and/or landslide might occur and maintenance would be a problem.	Construction works is in the flat plain. Backwater levee function is obtained by additional embankment to the main channel of Sukam and Palangki rivers.

5.10.6 Optimum Flood Control Plan

As a result of the above optimization, the optimum flood control plan for rivers in Sijunjung/Muara Area is established. Plan, proposed longitudinal profiles and typical cross sections are shown in Figs. VI.5.16 to VI.5.18, respectively.

CHAPTER 6 FEASIBILITY STUDY

6.1 General

(1) Objective Priority Projects for Feasibility Study

The objective priority projects for feasibility study of flood control component are the following two:

- Bangkinang Area River Improvement Works
- Rengat Area Flood Protection Works

(2) Target Completion Year

The target year for the completion of implementation of the priority projects is set at 2004. The year 2004 is the last year of the Seventh 5-Year Development Plan (REPELITA VII). The implementation period of nine years (1996-2004) is deemed appropriate for the feasibility study.

6.2 Bangkinang Area River Improvement Works

6.2.1 Planning Criteria

(1) Objective River Stretch

The objective stretch for the feasibility study is the Kampar Kanan River from Rantauberangin Bridge (just upstream of Kuok Intake Weir) down to Danaubingkuang Bridge with a total length of approximately 49 km. This stretch corresponds to the irrigation development area for the priority project of Kampar Kanan Water Supply Project.

(2) Design Scale

Design scale for the feasibility study stage is set at 5-year return period considering that the area consists mainly of agricultural lands. Consideration should be given, however, that the structures are to be upgraded in the future in accordance with the Overall Development Plan of 50-year return period scale. Accordingly, structures difficult for upgrading in the future, e.g., bridges and sluice gates, are to be designed for 50-year return period scale.

(3) Standard Flood Discharge

Standard flood discharges by design scale are as follows:

- 5-year Return Period: 2,800 m^3/s
- 50-year Return Period: 4,000 m^3/s

6.2.2 Optimization Study

The following optimization study and preliminary design have been conducted.

(1) Alignment and Longitudinal Profile

The plan and profile have been determined basically following the Overall Plan. The 1/50,000 topographical maps and the river survey results prepared during the Overall Development Plan stage have been used. The proposed plan and longitudinal profile are presented in Fig. VI.6.1 and VI.6.2.

(2) Cross Sections

In the Overall Plan stage, optimization study has been conducted and the optimum plan has been determined as follows: Assure flow capacity mainly by embankment. Channel of moderate width will be created. Excavation is considered only for the extremely narrowed sections. The width of the channel has been determined at 300 m comparing 200 m to 400 m.

On the basis of the cross sections determined in the Overall Plan stage, more detailed optimization has been conducted comparing the width of the following five cases and it has been finally determined at 300 m as the same results with the Overall Plan:

	Case 1	Case 2	Case 3	Case 4	Case 5
River Width (m)	200	250	300	350	400
Height of High Water Level from present land (m)	3.7	3.0	2.6	2.5	2.4
Excavation (1,000 m^3)	1,500	1,500	1,500	1,500	1,500
Embankment (1,000 m^3)	9,340	7,420	6,420	6,180	5,950
Land Acquisition (ha)	570	750	940	1,140	1.350
Cost (Rp. 10^9)	136	121	114	115	117

Design crest elevation has been determined based on a 5-year return period flood. Future raising of the dike will be made on the land side. Proposed cross sections are presented in Fig. VI.6.3.

(3) Related Structure

Preliminary design of related structures is presented in SECTOR XIII, RIPARIAN STRUCTURE ENGINEERING.

6.3 Rengat Area Flood Protection Works

6.3.1 Planning Criteria

(1) Objective Area for Flood Control

The objective area for flood control is determined as shown in Fig. VI.6.4 based on the Detailed City Layout Plan of Rengat as well as considering the discussion with PU officials.

(2) Design Scale

Design scale of 10-year return period is to be applied for the flood control in the area since this area is considered as the urban area. The historical maximum flooding level will also be considered for the determination of the design high water level. The design scale for the interior drainage has been determined at 5-year return period (for detail, refer to sub-section 6.3.3).

(3) Related Other Projects

The following projects are related for the formulation of flood protection in Rengat Area.

- Kuantan Dam Construction Works
- Kuantan-Indragiri River Improvement Project (Lubukjambi-Peranap Area and Peranap-Japura Area River Improvement Works)
- Indragiri-Gaung Floodway

Of these, Kuantan Dam Construction Works and Lubukjambi-Peranap Area and Peranap-Japura Area River Improvement Works are scheduled to be implemented after the completion of Rengat Area Flood Control Project (refer to Implementation Schedule of Overall Development Plan). Accordingly, the condition without these two projects has been considered.

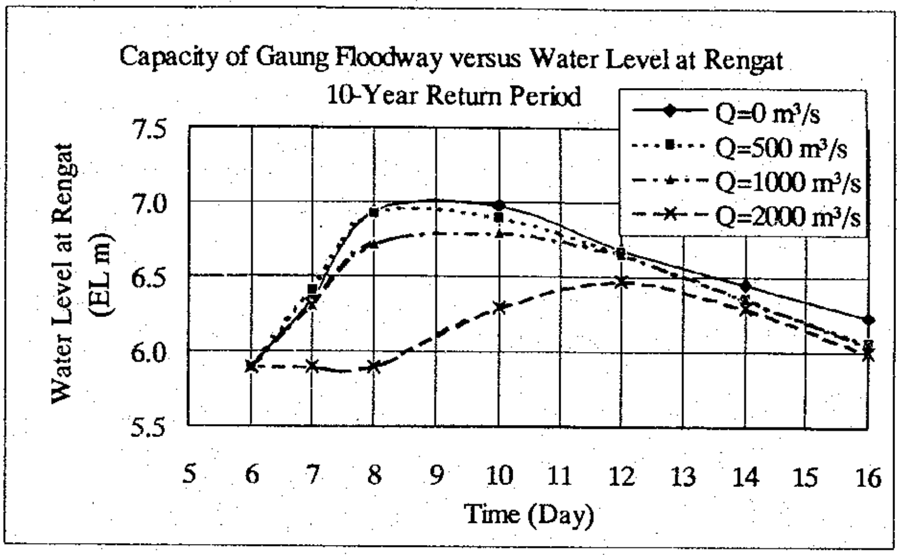
With regard to Indragiri-Gaung Floodway, implementation schedule has not been finalized by the Indonesian Government. Accordingly, the condition without the floodway has been considered for the planning of Rengat Area Flood Protection Works. However, the effect of the floodway to the Rengat Area Flood Protection Works has been studied in the subsequent paragraphs.

(4) Design High Water Level and Design Discharge

Design high water level for the design scale of 10-year return period is to be determined based on the above condition. When the condition as mentioned above is considered, the flood in the Kuantan-Indragiri River flows down by flooding to the riverine areas. Accordingly, the result of two-dimensional non-uniform flow calculation has been used to determined the high water level.

As presented in 6.4.1 of SECTOR I METEOROLOGY AND HYDROLOGY, the highest flood water level at the pier of Rengat City when the 10-year return period flood flows is EL 7.0 m.

The effect of floodway to the Gaung River has been evaluated hereunder. The following illustration shows curves of the water level at Rengat for the different capacity of the Indragiri-Gaung floodway. As shown in the illustration, the highest water level of EL 7.0 m in the case of no floodway is lowered to EL 6.90 m when the capacity of 500 m^3/s is given to the floodway. The difference is very small. Accordingly, the effect of Indragiri-Gaung floodway has not been considered for the determination of high water level.



Historical high water levels at the same point are as follows: The flood water level has never exceeded the road surface of EL 7.240 m in the last 20 years but almost exceeded several times. The water level during the flood in March 1995 (the highest in the past 10 years) was EL 6.744 m. The calculation result of EL 7.0 m of high water level is accordingly deemed appropriate. Design discharge of the Indragiri River is obtained from H-Q curve at Q=2,850 m^3/s.

6.3.2 Optimization Study of Ring Dike

(1) Dike Alignment

Dike alignment is as shown in Fig. VI.6.4. The alignment has been so determined as to follow the Indragiri River improvement plan in the Overall Plan in the river side, and considering the existing road alignment and the possible new road to Tembilahan in the land side.

(2) Longitudinal Profile and Cross Sections

Longitudinal profile and cross sections will be planned on the basis of the determined design high water level.

In accordance with the Overall Plan of Indragiri River Improvement Project, design high water gradient is 1/5,500. The longitudinal profile of the bank in the Indragiri River side has been thus determined at 1/5,500.

The land side dike can be divided into two, namely, the stretch from No. L-10+300 to No.L-15+200 and the stretch from No. L-0 to No. L-10+300.

The stretch from No. L-10+300 to No. L-15+200 runs along the existing road connecting Japura and Rengat City. The result of two-dimensional non-uniform flow calculation revealed that along this alignment, water gradient during floods is minimal. Accordingly, high water level of this stretch has been considered level.

The desigh high water level for the stretch from No. L-0 to No. L-10+300 has been determined by connecting the high water level at No. L-0 and the same at No. L-10+300.

Cross sections have been determined based on the same condition with the Overall Development Plan.

Proposed longitudinal profile and cross sections are presented in Fig. XI.6.5 and VI.6.6, respectively.

6.3.3 Optimization Study for Interior Drainage

The proposed ring dike line inevitably crosses the existing drainage channels and creates an interior drainage problem. Most streams in the Rengat area can discharge by gravity during low flows of the Indragiri River. During floods, however, the inland water is collected to lower areas and inundate the area since gravity flow through outlet gates is impossible due to continuing high water level of the Indragiri River

As a solution of the interior drainage problem, a pumping station is thus required to discharge the interior water over the dike during floods. This sub-section studies necessity and optimum scale of the pumping station based on a benefits and costs analysis.

(1) Design Scale

The design scale for the planning has been determined at 5-year return period. This has been determined based on Flood Control Manual, CIDA-DPU, June 1993 in due consideration of the case of the initial phase of the primary drainage system for urban areas with a population less than 500,000.

(2) Design Rainfall

Design rainfall for the interior drainage analysis has been determined based on the three factors, namely, duration, probable amount and temporal distribution.

(a) Duration of Rainfall

The inundation analysis presented in SECTOR I, METEOROLOGY AND HYDROLOGY shows that the water level of Indragiri River at Rengat keeps up the elevation higher than 6.5 m for approximately one week during the 5-year return period flood. Therefore, the duration of design rainfall is considered to be ten days.

(b) Probable Rainfall Amount

The rainfall station at Japura, Rengat is the nearest one to the objective area. Accordingly, the rainfall record at Japura has been used for the study. As presented in SECTOR I, METEOROLOGY AND HYDROLOGY, probable 10-day rainfall at Japura station is as follows:

Unit: mm

Return Period	Probable 10-Day Rainfall
2-year	205
5-year	230

(c) Temporal Distribution of Rainfall

Actually observed rainfall has been applied for temporal distribution of design rainfall. For the 13-year period from 1981-93, annual maximum 10-day rainfalls at Japura station have been picked up for temporal distribution of design rainfall. Actual values are presented in SECTOR I METEOROLOGY AND HYDROLOGY.

(3) Establishment of Inundation Model

An inundation model has been established as follows: For this project, the interior drainage area is considered as a reservoir when a flood occurs. Under the present condition, there is no outlets to discharge rainfall in the area, but the rainfall is simply stored in the area. Under the with-a-pumping-station condition, the pumping station drain stored water.

Topographical data of the flood inundation model as presented in SECTOR I, can result in clarifying relation between inundated elevation and its volume contained below that elevation. Fig. VI.6.7 illustrates this relation. Fig. VI.6.7 is to be utilized to obtain water level from the stored water volume.

Spatial distribution of inundation depth has been obtained using the same mesh blocks developed for flood inundation simulation in SECTOR I.

(4) Optimization of Pump Capacity

Necessity and optimum capacity of the pumping station has been determined hereunder based on a benefits and costs analysis.

(a) Calculation of Benefits

The benefits for a flood mitigation project are generally calculated by flood damage reductions which are determined based on the difference in average annual flood damage with and without a pumping station.

The average annual flood damage can be computed by summation of damages which is obtained by multiplying damage expected at a given stage by the probability of occurrence of that stage.

(i) Damage Without a Pumping Station

Expected damages have been firstly estimated for 2-year and 5-year return period rainfall and an average annual damage has been accordingly calculated.

The catchment area is measured at 2,337.5 ha by the flood inundation model. Rainfall losses in terms of basin retention, evapotranspiration, interception, infiltration and percolation are omitted because of this small catchment area.

By entering the inundated elevation-volume curve (refer to Fig. VI.6.7) on the horizontal axis at a runoff volume, the inundation elevation can be determined by projecting to the up and left from the intersection with the curve. The maximum inundated elevation for each design flood is accordingly obtained.

The inundation depth is given by the difference between the inundation elevation and the ground height of each mesh. For each design flood, Fig. VI.6.8 illustrates the calculated inundation depth.

Resulting from the same calculation model used in the inundation analysis, the above calculated depth and duration bring the expected damage without a pumping station.

The calculation procedure is as follows:

Particulars	Unit	Return Period	
		2-year	5-year
Probable Rainfall	mm	205	230
Runoff Volume	$\times 10^3 m^3$	4,792	5,376
Max. Inundation Elevation	EL m	4.79	4.89
Expected Damage	Rp. 10^6	5,210	5,700

An average annual damage has been accordingly calculated at Rp. 3,530 million.

(ii) Damage with a Pumping Station

The expected damage with a pumping station is also estimated by using inundation depth and duration.

Calculation procedure for these two factors is almost the same as the above except that the runoff volume for each return period are measured in considering pump capacity; that is, the runoff volume stored in the inundated area can be computed by the runoff volume without a pumping station minus discharge from a pumping plant.

This computation is carried out day by day for the maximum 10-day rainfall patterns in 13-year records from 1981 to 1993. The model rainfall patterns are extended in accordance with the probable value for each return period (refer to Table VI.6.1).

If the daily discharge from a pumping station is larger than the runoff volume in a day, no inundation can show up. But, if the runoff volume exceeds the pumping discharge, difference between the pumping discharge and runoff volume must carry over and add to the next day runoff volume to be calculated.

Assuming several pump capacities, the day-by-day simulation gives results of the stored runoff volume as shown in Table VI.6.2. The following table summarize the inundated elevation and its duration for each return period.

Return Period	Inundation Condition	Unit	Pump Capacity (m^3/s)						
			1	3	5	8	10	20	30
2-Year	Max. Elevation	m	4.6	4.4	4.4	4.3	4.3	4.1	3.9
	Duration	day	5	1	1	1	1	1	1
5-Year	Max. Elevation	m	4.7	4.5	4.5	4.4	4.4	4.2	4.0
	Duration	day	5	1	1	1	1	1	1

Based on the prescribed calculation results, the inundation depth is measured as shown in Fig. VI.6.9, and the expected damage with a pumping station is estimated as the following table. The average annual flood damage is computed based on the expected damage with the probability of occurrence in any year. Table VI.6.3 shows these computation results.

Expected Damage

Unit: Rp. 10^6

Return Period	Pump Capacity (m^3/s)						
	1	3	5	8	10	20	30
2-Year	4,819	3,447	3,445	3,127	3,127	2,471	1,262
5-Year	5,154	4,111	4,111	3,447	3,447	2,700	1,859

(iii) Average Annual Flood Damage Reduction

The average annual flood damage reductions for each option are given by the average annual flood damage for different pump capacity and summarized below.

Unit: Rp. 10^6

Particulars	Pump Capacity (m^3/s)							
	0	1	3	5	8	10	20	30
Average Annual Flood Damage	2,939	2,701	1,995	1,994	1,768	1,768	1,393	784
Average Annual Flood Damage Reduction	-	238	944	945	1,171	1,171	1,546	2,155

The benefits is computed by the average annual damage reduction minus annual operation and maintenance which is assumed as 0.5 percent of total project costs (the estimation of total project costs is discussed later). The computation results are listed as follows:

Unit: Rp. 10^6

Particulars	Pump Capacity (m^3/s)						
	1	3	5	8	10	20	30
Average Annual Flood Damage Reduction	238	944	945	1,171	1,171	1,546	2,155
Annual Operation and Maintenance Cost	5	15	25	40	50	100	150
Annual Benefit	243	959	970	1,211	1,221	1,646	2,305

(b) Calculation of Costs

The calculation of costs for each option is found from the equation,

$$C = I \times \left[i + \frac{i}{(1+i)^n - 1} \right]$$

where,

C : annual cost
I : total project cost
I : interest rate (set at 10%)
n : project life (assumed at 20 years considering the life of a pump)

The total project costs are estimated based on the assumption that the project costs per pump capacity (cubic meter per second) are one billion Indonesian Rupiah.

In the above equation,

$$\left[i+\frac{i}{(1+i)^n-1}\right]=\left[0.10+\frac{0.10}{(1+0.10)^{20}-1}\right]=0.1176$$

Consequently, the annual costs are then computed as follows:

Unit: Rp. 10^6

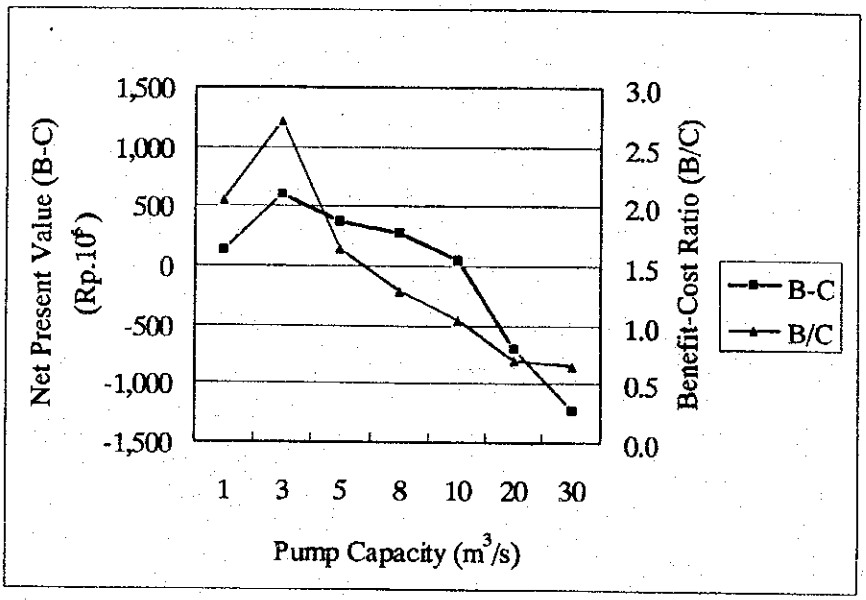
Particulars	Pump Capacity (m^3/s)						
	1	3	5	8	10	20	30
Annual Cost	118	353	588	941	1,176	2,352	3,528

(c) Benefit and Cost Analysis

The Net Present Value (B-C) and Benefit Cost Ratio (B/C) are employed in evaluating alternatives for the interior drainage. Prescribed calculations of the annual benefits and costs are used for selecting the optimum pump capacity as shown below.

Particulars	Pump Capacity (m^3/s)						
	1	3	5	8	10	20	30
Annual Benefit (B) (Rp. 10^6	243	959	970	1,211	1,221	1,646	2,305
Annual Cost (C) (Rp. 10^6)	118	353	588	941	1,176	2,352	3,528
Net Present Value (B-C) (Rp. 10^6)	125	606	382	270	45	-706	-1,223
Benefit-Cost Ratio (B/C)	2.06	2.72	1.65	1.29	1.04	0.70	0.65

Relation between the pump capacity and the two indicators, i.e., Net Present Value (B-C) and Benefit-Cost Ratio (B/C) is illustrated below. This figure shows that both the B-C and B/C are the maximum at the pump capacity of 3 m^3/s and concludes that the 3 m^3/s pump is the best choice in these alternatives. The area of about 20 ha with the elevations less than EL 3.0 m in the downstreammost area of the Senggeris River has been utilized as a retarding basin as shown in Fig. VI.6.4.



TABLES

VI FLOOD CONTROL PLAN

Table VI.3.1 OUTLINE OF EXISTING RIVER IMPROVEMENT PROJECT

Item \ Project		Lembang	Sinamar
1. Project Site		Solok, West Sumatra	Payakumbuh, West Sumatra
2. Catchment Area	(km^2)	408.5	
3. Protected River Stretch			
(1) River Name		Lembang	Sinamar/Lampasi/Agam
(2) Length	(km)	11.6	21.5 / 11.0 / 7.8
(3) Downstream End		12.3 km upstream of Singkarak Lake	8.9 km downstream of Agam junction
(4) Upstream End		500 m upstream of Sumani junction	
4. Design Scale			
(1) Design Return Period	(Year)	5 / 20	20
(2) Design Discharge	(m^3/s)	357.0 / 496.5	
(3) Specific Discharge	($m^3/s/km^2$)	0.87 / 1.22	
5. Flood Control Measures			
(1) Embankment		Yes for 20–year return period	
(2) Normalization		Yes	Yes
(3) Short Cut		Yes	Yes
(4) Floodway		No	No
(5) Others		No	Revetment/Check dam Groundsill/Rubber dam
6. Completion Target Year		after 1998/99	after 1998/99
7. Implemented/Proposed by		DPU SUMBAR	DPU SUMBAR

Table VI.5.1 PROTECTION LEVEL FOR FLOOD CONTROL WORKS IN INDONESIA

No.	Name of River	Province	Catchment Area (km^2)	Design Flood (m^3/s)	Return Period (Year)	Remarks
1	Blorong	Central Jawa	157.0	435	20	Completed
2	Silandak	Central Jawa	8.5	92	50	Completed
3	Babon	Central Jawa	77.0	320	5	Completed
				460	25	On-going
4	East Floodway	Central Jawa	29.7	333	25	On-going
5	Cimanuk	West Jawa	3,006.0	1,440	25	
6	Serang	Central Jawa	937.0	900	25	
7	Citandui	West Jawa	3,680.0	1,900	25	
8	Ular	North Sumatra	1,080.0	800	30	
9	Pemali	Central Jawa	1,228.0	1,300	25	
10	Cipanas	West Jawa	220.0	385	25	
11	Solo	Central/East Jawa	3,320.0	1,500	10	Urgent Plan
				2,000	40	
12	Madiun	East Jawa	2,400.0	1,100	10	Urgent Plan
				2,300	40	
13	Wanpu	North Sumatra	3,840.0	1,320	20	
14	Arakundo	Aceh	5,495.0	2,100	50	
15	Krung Aceh	Aceh	1,775.0	1,960	50	
16	Brantas	East Jawa	10,000.0	1,350	10	Urgent Plan
				1,500	50	
17	Bah Bolon	North Sumatra	2,776.0	1,200	20	
18	Walanae	South Sulawesi	3,190.0	2,900	20	
19	Bila	South Sulawesi	1,368.0	1,900	20	
20	Jeneberang	South Sulawesi	729.0	3,700	50	
21	Ciujung	North Banten	1,850.0	1,100	10	Urgent Plan
				1,600	50	
22	Kuranji	West Sumatra	213.0	870	25	Urgent Plan
				1,000	50	
23	Air Dingin	West Sumatra	131.0	600	25	Urgent Plan
				700	50	
24	Marmoyo	East Jawa	290.0	230	20	
25	Surabaya	East Jawa	631.0	370	50	

Table VI.5.2 ANNUAL MAXIMUM DISCHARGE WITH/WITHOUT KOTAPANJANG RESERVOIR

Year	Annual Maximum Discharge (m^3/s)		Ratio
	Without Reservoir	With Reservoir	With/Without
1971	508.3	215.6	0.42
1972	876.7	215.6	0.25
1973	656.6	215.6	0.33
1974	677.4	215.6	0.32
1975	885.5	215.6	0.24
1976	828.6	215.6	0.26
1977	733.6	596.3	0.81
1978	882.4	831.6	0.94
1979	544.3	215.6	0.40
1980	624.9	215.6	0.35
1981	552.9	215.6	0.39
1982	997.4	997.4	1.00
1983	932.7	747.2	0.80
1984	981.8	878.8	0.90
1985	1,048.0	1,003.7	0.96
1986	1,422.4	1,192.9	0.84
1987	1,187.4	668.2	0.56
1988	697.4	697.4	1.00
1989	938.4	215.6	0.23
1990	1,005.1	463.7	0.46
1991	1,483.5	1,037.9	0.70
1992	678.2	585.6	0.86

Table VI.6.1 MODEL RAINFALL PATTERNS EXTENDED IN ACCORDANCE WITH DESIGN FLOOD VOLUME FOR EACH RETURN PERIOD

2-year Return Period

Unit: $10^6 m^3$

Day	Date and Year												
	1993	1988	1991	1992	1985	1986	1982	1983	1984	1989	1987	1990	1981
	4/28- 5/7	1/17- 1/26	3/7- 3/16	10/25- 11/3	11/27- 12/6	4/29- 5/8	12/15- 12/24	11/26- 12/5	4/7- 4/16	11/29- 12/8	4/22- 4/3	11/19- 11/28	4/29- 5/[illegible]
1	0.618	0.977	0.075	0.861	0.596	1.894	0.460	0.557	0.574	0.900	0.428	1.287	0.255
2	1.891	0.132	1.641	0.685	0.000	0.000	0.242	0.557	0.050	0.079	0.134	0.055	0.425
3	0.000	0.207	0.000	0.020	0.596	0.357	0.411	0.290	0.349	0.132	0.321	2.026	0.227
4	0.356	0.226	0.000	0.000	0.044	0.000	0.194	0.073	0.125	0.450	0.000	0.438	0.057
5	0.000	0.000	0.000	0.313	0.000	0.156	0.436	1.041	0.399	0.635	0.080	0.274	2.212
6	0.206	0.075	1.302	0.196	3.290	0.000	0.048	0.097	1.473	0.000	0.321	0.055	0.000
7	0.019	0.019	0.000	0.587	0.022	0.735	0.411	0.702	0.175	0.053	0.455	0.329	0.000
8	0.037	2.762	1.358	0.176	0.000	0.000	0.944	0.000	0.424	0.000	0.027	0.055	0.510
9	1.666	0.000	0.000	1.956	0.022	0.000	0.315	1.307	0.000	1.377	0.000	0.137	0.000
10	0.000	0.395	0.415	0.000	0.221	1.649	1.331	0.169	1.223	1.165	3.025	0.137	1.106
Total	4.792	4.792	4.792	4.792	4.792	4.792	4.792	4.792	4.792	4.792	4.792	4.792	4.792

5-year Return Period

Unit: $10^6 m^3$

Day	Date and Year												
	1993	1988	1991	1992	1985	1986	1982	1983	1984	1989	1987	1990	1981
	4/28- 5/7	1/17- 1/26	3/7- 3/16	10/25- 11/3	11/27- 12/6	4/29- 5/8	12/15- 12/24	11/26- 12/5	4/7- 4/16	11/29- 12/8	4/22- 4/3	11/19- 11/28	4/29- 5/8
1	0.693	1.096	0.085	0.966	0.669	2.125	0.516	0.625	0.644	1.010	0.481	1.444	0.286
2	2.121	0.148	1.841	0.768	0.000	0.000	0.272	0.625	0.056	0.089	0.150	0.061	0.477
3	0.000	0.232	0.000	0.022	0.669	0.400	0.462	0.326	0.392	0.149	0.360	2.273	0.254
4	0.399	0.253	0.000	0.000	0.050	0.000	0.217	0.081	0.140	0.505	0.000	0.492	0.064
5	0.000	0.000	0.000	0.351	0.000	0.175	0.489	1.168	0.448	0.713	0.090	0.307	2.481
6	0.231	0.084	1.460	0.219	3.692	0.000	0.054	0.109	1.652	0.000	0.360	0.061	0.000
7	0.021	0.021	0.000	0.658	0.025	0.825	0.462	0.787	0.196	0.059	0.511	0.369	0.000
8	0.042	3.099	1.524	0.197	0.000	0.000	1.059	0.000	0.476	0.000	0.030	0.061	0.57[illegible]
9	1.869	0.000	0.000	2.194	0.025	0.000	0.353	1.466	0.000	1.545	0.000	0.154	0.000
10	0.000	0.443	0.466	0.000	0.248	1.850	1.493	0.190	1.372	1.307	3.394	0.154	1.241
Total	5.376	5.376	5.376	5.376	5.376	5.376	5.376	5.376	5.376	5.376	5.376	5.376	5.376

Table VI.6.2 (1/5) SIMULATION RESULTS OF STORED RUNOFF VOLUME FOR DIFFERENT PUMP CAPACITY AT EACH RETURN PERIOD

Pump Capacity of 1 m^3/s at 2-year Return Period

Unit: $10^6 m^3$

Day	Simulated Date and Year												
	1993	1988	1991	1992	1985	1986	1982	1983	1984	1989	1987	1990	1981
	4/28-	1/17-	3/7-	10/25-	11/27-	4/29-	12/15-	11/26-	4/7-	11/29-	4/22-	11/19-	4/29-
	5/7	1/26	3/16	11/3	12/6	5/8	12/24	12/5	4/16	12/8	4/3	11/28	5/8
1	0.531	0.891	0.000	0.774	0.510	1.808	0.373	0.470	0.488	0.814	0.342	1.201	0.169
2	2.335	0.936	1.555	1.372	0.423	1.722	0.529	0.940	0.451	0.807	0.389	1.169	0.508
3	2.249	1.056	1.469	1.305	0.933	1.992	0.854	1.144	0.714	0.853	0.624	3.109	0.648
4	2.518	1.195	1.382	1.219	0.891	1.905	0.961	1.131	0.753	1.216	0.538	3.461	0.618
5	2.432	1.109	1.296	1.446	0.805	1.975	1.311	2.085	1.065	1.765	0.532	3.648	2.744
6	2.551	1.098	2.511	1.555	4.008	1.889	1.273	2.095	2.452	1.679	0.767	3.616	2.657
7	2.484	1.030	2.425	2.055	3.944	2.538	1.598	2.711	2.540	1.646	1.135	3.858	2.571
8	2.435	3.706	3.697	2.145	3.858	2.451	2.455	2.624	2.878	1.559	1.076	3.827	2.995
9	4.014	3.620	3.610	4.014	3.793	2.365	2.683	3.845	2.791	2.849	0.989	3.877	2.908
10	3.928	3.928	3.939	3.928	3.928	3.928	3.928	3.928	3.928	3.928	3.928	3.928	3.928
Total	25.478	18.568	21.882	19.814	23.094	22.573	15.964	20.974	18.059	17.116	10.320	31.694	19.746

Pump Capacity of 3 m^3/s at 2-year Return Period

Unit: $10^6 m^3$

Day	Simulated Date and Year												
	1993	1988	1991	1992	1985	1986	1982	1983	1984	1989	1987	1990	1981
	4/28-	1/17-	3/7-	10/25-	11/27-	4/29-	12/15-	11/26-	4/7-	11/29-	4/22-	11/19-	4/29-
	5/7	1/26	3/16	11/3	12/6	5/8	12/24	12/5	4/16	12/8	4/3	11/28	5/8
1	0.359	0.718	0.000	0.601	0.337	1.635	0.201	0.297	0.315	0.641	0.169	1.028	0.000
2	1.990	0.590	1.382	1.027	0.078	1.376	0.183	0.595	0.106	0.461	0.044	0.823	0.166
3	1.731	0.538	1.123	0.787	0.415	1.473	0.336	0.626	0.196	0.334	0.106	2.590	0.134
4	1.827	0.504	0.864	0.528	0.200	1.214	0.270	0.439	0.061	0.525	0.000	2.769	0.000
5	1.568	0.245	0.605	0.582	0.000	1.111	0.447	1.221	0.201	0.901	0.000	2.784	1.952
6	1.515	0.061	1.647	0.518	3.031	0.852	0.236	1.059	1.415	0.642	0.062	2.580	1.693
7	1.274	0.000	1.388	0.846	2.794	1.328	0.388	1.501	1.330	0.436	0.258	2.649	1.434
8	1.052	2.503	2.487	0.762	2.535	1.069	1.073	1.242	1.495	0.177	0.026	2.444	1.685
9	2.459	2.244	2.228	2.459	2.298	0.810	1.128	2.290	1.236	1.294	0.000	2.322	1.426
10	2.200	2.379	2.384	2.200	2.259	2.200	2.200	2.200	2.200	2.200	2.766	2.200	2.273
Total	15.974	9.783	14.106	10.310	13.946	13.069	6.460	11.470	8.555	7.612	3.430	22.190	10.763

Pump Capacity of 5 m^3/s at 2-year Return Period

Unit: $10^6 m^3$

Day	Simulated Date and Year												
	1993	1988	1991	1992	1985	1986	1982	1983	1984	1989	1987	1990	1981
	4/28-	1/17-	3/7-	10/25-	11/27-	4/29-	12/15-	11/26-	4/7-	11/29-	4/22-	11/19-	4/29-
	5/7	1/26	3/16	11/3	12/6	5/8	12/24	12/5	4/16	12/8	4/3	11/28	5/8
1	0.186	0.545	0.000	0.429	0.164	1.462	0.028	0.125	0.142	0.468	0.000	0.855	0.000
2	1.644	0.245	1.209	0.681	0.000	1.030	0.000	0.249	0.000	0.116	0.000	0.478	0.000
3	1.212	0.019	0.777	0.269	0.164	0.955	0.000	0.108	0.000	0.000	0.000	2.072	0.000
4	1.136	0.000	0.345	0.000	0.000	0.523	0.000	0.000	0.000	0.018	0.000	2.078	0.000
5	0.704	0.000	0.000	0.000	0.000	0.247	0.004	0.609	0.000	0.221	0.000	1.920	1.780
6	0.478	0.000	0.870	0.000	2.858	0.000	0.000	0.273	1.041	0.000	0.000	1.543	1.348
7	0.065	0.000	0.438	0.155	2.448	0.303	0.000	0.543	0.783	0.000	0.023	1.439	0.916
8	0.000	2.330	1.364	0.000	2.016	0.000	0.512	0.111	0.775	0.000	0.000	1.062	0.994
9	1.234	1.898	0.932	1.524	1.606	0.000	0.394	0.986	0.343	0.945	0.000	0.767	0.562
10	0.802	1.861	0.915	1.092	1.395	1.217	1.294	0.724	1.134	1.678	2.593	0.472	1.236
Total	7.460	6.899	6.851	4.149	10.653	5.739	2.231	3.728	4.219	3.445	2.616	12.686	6.835

Table VI.6.2 (2/5) SIMULATION RESULTS OF STORED RUNOFF VOLUME FOR DIFFERENT PUMP CAPACITY AT EACH RETURN PERIOD

Pump Capacity of 8 m^3/s at 2-year Return Period

Unit: $10^6 m^3$

Day	Simulated Date and Year												
	1993	1988	1991	1992	1985	1986	1982	1983	1984	1989	1987	1990	1981
	4/28-5/7	1/17-1/26	3/7-3/16	10/25-11/3	11/27-12/6	4/29-5/8	12/15-12/24	11/26-12/5	4/7-4/16	11/29-12/8	4/22-4/3	11/19-11/28	4/29-5/8
1	0.000	0.286	0.000	0.169	0.000	1.203	0.000	0.000	0.000	0.209	0.000	0.596	0.000
2	1.199	0.000	0.950	0.163	0.000	0.512	0.000	0.000	0.000	0.000	0.000	0.000	0.000
3	0.508	0.000	0.259	0.000	0.000	0.177	0.000	0.000	0.000	0.000	0.000	1.335	0.000
4	0.173	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	1.082	0.000
5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.349	0.000	0.000	0.000	0.665	1.520
6	0.000	0.000	0.611	0.000	2.599	0.000	0.000	0.000	0.781	0.000	0.000	0.028	0.829
7	0.000	0.000	0.000	0.000	1.930	0.044	0.000	0.011	0.265	0.000	0.000	0.000	0.138
8	0.000	2.071	0.667	0.000	1.239	0.000	0.253	0.000	0.000	0.000	0.000	0.000	0.000
9	0.975	1.380	0.000	1.265	0.570	0.000	0.000	0.616	0.000	0.685	0.000	0.000	0.000
10	0.284	1.083	0.000	0.000	0.000	0.958	0.640	0.094	0.532	1.159	2.334	0.000	0.415
Total	3.138	4.821	2.487	1.597	6.337	2.895	0.893	1.070	1.578	2.054	2.334	3.706	2.902

Pump Capacity of 10 m^3/s at 2-year Return Period

Unit: $10^6 m^3$

Day	Simulated Date and Year												
	1993	1988	1991	1992	1985	1986	1982	1983	1984	1989	1987	1990	1981
	4/28-5/7	1/17-1/26	3/7-3/16	10/25-11/3	11/27-12/6	4/29-5/8	12/15-12/24	11/26-12/5	4/7-4/16	11/29-12/8	4/22-4/3	11/19-11/28	4/29-5/8
1	0.000	0.113	0.000	0.000	0.000	1.030	0.000	0.000	0.000	0.036	0.000	0.423	0.000
2	1.027	0.000	0.777	0.000	0.000	0.166	0.000	0.000	0.000	0.000	0.000	0.000	0.000
3	0.163	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	1.162	0.000
4	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.736	0.000
5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.177	0.000	0.000	0.000	0.146	1.348
6	0.000	0.000	0.438	0.000	2.426	0.000	0.000	0.000	0.609	0.000	0.000	0.000	0.484
7	0.000	0.000	0.000	0.000	1.584	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	1.898	0.494	0.000	0.720	0.000	0.080	0.000	0.000	0.000	0.000	0.000	0.000
9	0.802	1.034	0.000	1.092	0.000	0.000	0.000	0.443	0.000	0.513	0.000	0.000	0.000
10	0.000	0.565	0.000	0.228	0.000	0.785	0.467	0.000	0.359	0.814	2.161	0.000	0.242
Total	1.991	3.611	1.709	1.320	4.731	1.982	0.547	0.620	0.967	1.362	2.161	2.468	2.073

Pump Capacity of 20 m^3/s at 2-year Return Period

Unit: $10^6 m^3$

Day	Simulated Date and Year												
	1993	1988	1991	1992	1985	1986	1982	1983	1984	1989	1987	1990	1981
	4/28-5/7	1/17-1/26	3/7-3/16	10/25-11/3	11/27-12/6	4/29-5/8	12/15-12/24	11/26-12/5	4/7-4/16	11/29-12/8	4/22-4/3	11/19-11/28	4/29-5/8
1	0.000	0.000	0.000	0.000	0.000	0.166	0.000	0.000	0.000	0.000	0.000	0.000	0.000
2	0.163	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.298	0.000
4	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.484
6	0.000	0.000	0.000	0.000	1.562	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	1.034	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	0.000	0.000	0.228	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
10	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	1.297	0.000	0.000
Total	0.163	1.034	0.000	0.228	1.562	0.166	0.000	0.000	0.000	0.000	1.297	0.298	0.484

Table VI.6.2 (3/5) SIMULATION RESULTS OF STORED RUNOFF VOLUME FOR DIFFERENT PUMP CAPACITY AT EACH RETURN PERIOD

Pump Capacity of 30 m^3/s at 2-year Return Period — Unit: $10^6 m^3$

Day	Simulated Date and Year												
	1993	1988	1991	1992	1985	1986	1982	1983	1984	1989	1987	1990	1981
	4/28-5/7	1/17-1/26	3/7-3/16	10/25-11/3	11/27-12/6	4/29-5/8	12/15-12/24	11/26-12/5	4/7-4/16	11/29-12/8	4/22-4/3	11/19-11/28	4/29-5/8
1	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
4	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
6	0.000	0.000	0.000	0.000	0.698	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	0.170	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
10	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.433	0.000	0.000
Total	0.000	0.170	0.000	0.000	0.698	0.000	0.000	0.000	0.000	0.000	0.433	0.000	0.000

Pump Capacity of 1 m^3/s at 5-year Return Period — Unit: $10^6 m^3$

Day	Simulated Date and Year												
	1993	1988	1991	1992	1985	1986	1982	1983	1984	1989	1987	1990	1981
	4/28-5/7	1/17-1/26	3/7-3/16	10/25-11/3	11/27-12/6	4/29-5/8	12/15-12/24	11/26-12/5	4/7-4/16	11/29-12/8	4/22-4/3	11/19-11/28	4/29-5/8
1	0.607	1.010	0.000	0.879	0.583	2.039	0.430	0.538	0.558	0.924	0.394	1.358	0.200
2	2.641	1.071	1.755	1.561	0.496	1.953	0.615	1.076	0.527	0.926	0.458	1.333	0.591
3	2.555	1.217	1.669	1.496	1.079	2.266	0.990	1.316	0.833	0.988	0.732	3.520	0.759
4	2.868	1.383	1.582	1.410	1.042	2.180	1.121	1.311	0.886	1.407	0.646	3.925	0.736
5	2.781	1.297	1.496	1.675	0.955	2.269	1.523	2.392	1.248	2.033	0.649	4.145	3.131
6	2.926	1.295	2.870	1.808	4.561	2.182	1.491	2.414	2.814	1.947	0.923	4.121	3.045
7	2.860	1.229	2.784	2.380	4.499	2.921	1.866	3.115	2.923	1.920	1.347	4.403	2.958
8	2.816	4.242	4.221	2.491	4.413	2.835	2.839	3.029	3.313	1.834	1.291	4.378	3.444
9	4.599	4.156	4.135	4.599	4.351	2.748	3.105	4.409	3.227	3.292	1.205	4.445	3.358
10	4.512	4.512	4.514	4.512	4.512	4.512	4.512	4.512	4.512	4.512	4.512	4.512	4.512
Total	29.165	21.412	25.025	22.810	26.490	25.905	18.491	24.111	20.841	19.783	12.158	36.138	22.734

Pump Capacity of 3 m^3/s at 5-year Return Period — Unit: $10^6 m^3$

Day	Simulated Date and Year												
	1993	1988	1991	1992	1985	1986	1982	1983	1984	1989	1987	1990	1981
	4/28-5/7	1/17-1/26	3/7-3/16	10/25-11/3	11/27-12/6	4/29-5/8	12/15-12/24	11/26-12/5	4/7-4/16	11/29-12/8	4/22-4/3	11/19-11/28	4/29-5/8
1	0.434	0.837	0.000	0.706	0.410	1.866	0.257	0.365	0.385	0.751	0.221	1.185	0.027
2	2.296	0.726	1.582	1.215	0.151	1.607	0.269	0.731	0.182	0.581	0.112	0.987	0.245
3	2.037	0.698	1.323	0.978	0.560	1.748	0.471	0.797	0.314	0.470	0.214	3.001	0.240
4	2.176	0.692	1.064	0.719	0.351	1.489	0.429	0.620	0.195	0.716	0.000	3.233	0.045
5	1.917	0.433	0.805	0.811	0.091	1.405	0.659	1.528	0.384	1.169	0.000	3.281	2.267
6	1.889	0.258	2.006	0.771	3.524	1.145	0.454	1.377	1.777	0.910	0.101	3.084	2.008
7	1.651	0.020	1.747	1.170	3.289	1.711	0.657	1.906	1.714	0.710	0.353	3.193	1.749
8	1.434	2.860	3.012	1.108	3.030	1.452	1.456	1.646	1.931	0.451	0.123	2.995	2.062
9	3.043	2.601	2.752	3.043	2.796	1.193	1.550	2.853	1.671	1.737	0.000	2.890	1.803
10	2.784	2.784	2.959	2.784	2.784	2.784	2.784	2.784	2.784	2.784	3.135	2.784	2.784
Total	19.661	11.908	17.249	13.306	16.986	16.401	8.987	14.607	11.337	10.279	4.259	26.634	13.230

Table VI.6.2 (4/5) SIMULATION RESULTS OF STORED RUNOFF VOLUME FOR DIFFERENT PUMP CAPACITY AT EACH RETURN PERIOD

Pump Capacity of 5 m^3/s at 5-year Return Period

Unit: $10^6 m^3$

Day	Simulated Date and Year												
	1993	1988	1991	1992	1985	1986	1982	1983	1984	1989	1987	1990	1981
	4/28- 5/7	1/17- 1/26	3/7- 3/16	10/25- 11/3	11/27- 12/6	4/29- 5/8	12/15- 12/24	11/26- 12/5	4/7- 4/16	11/29- 12/8	4/22- 4/3	11/19- 11/28	4/29- 5/8
1	0.261	0.664	0.000	0.534	0.237	1.693	0.084	0.193	0.212	0.578	0.049	1.012	0.000
2	1.950	0.380	1.409	0.870	0.000	1.261	0.000	0.385	0.000	0.235	0.000	0.641	0.04[illegible]
3	1.518	0.180	0.977	0.460	0.237	1.230	0.030	0.279	0.000	0.000	0.000	2.483	0.000
4	1.485	0.001	0.545	0.028	0.000	0.798	0.000	0.000	0.000	0.073	0.000	2.542	0.000
5	1.053	0.000	0.113	0.000	0.000	0.541	0.057	0.736	0.016	0.354	0.000	2.417	2.049
6	0.852	0.000	1.142	0.000	3.260	0.109	0.000	0.412	1.236	0.000	0.000	2.047	1.617
7	0.441	0.000	0.710	0.226	2.852	0.502	0.030	0.768	1.000	0.000	0.079	1.984	1.185
8	0.051	2.667	1.802	0.000	2.420	0.070	0.657	0.336	1.044	0.000	0.000	1.613	1.326
9	1.488	2.235	1.370	1.762	2.013	0.000	0.578	1.370	0.612	1.113	0.000	1.335	0.894
10	1.056	2.246	1.404	1.330	1.829	1.418	1.639	1.128	1.552	1.987	2.962	1.056	1.703
Total	10.157	8.373	9.473	5.209	12.848	7.621	3.073	5.605	5.673	4.340	3.089	17.130	8.820

Pump Capacity of 8 m^3/s at 5-year Return Period

Unit: $10^6 m^3$

Day	Simulated Date and Year												
	1993	1988	1991	1992	1985	1986	1982	1983	1984	1989	1987	1990	1981
	4/28- 5/7	1/17- 1/26	3/7- 3/16	10/25- 11/3	11/27- 12/6	4/29- 5/8	12/15- 12/24	11/26- 12/5	4/7- 4/16	11/29- 12/8	4/22- 4/3	11/19- 11/28	4/29- 5/8
1	0.002	0.405	0.000	0.274	0.000	1.434	0.000	0.000	0.000	0.319	0.000	0.753	0.000
2	1.432	0.000	1.150	0.351	0.000	0.743	0.000	0.000	0.000	0.000	0.000	0.123	0.00[illegible]
3	0.741	0.000	0.459	0.000	0.000	0.452	0.000	0.000	0.000	0.000	0.000	1.705	0.000
4	0.448	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	1.505	0.000
5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.476	0.000	0.022	0.000	1.121	1.790
6	0.000	0.000	0.769	0.000	3.000	0.000	0.000	0.000	0.961	0.000	0.000	0.492	1.099
7	0.000	0.000	0.078	0.000	2.334	0.134	0.000	0.096	0.466	0.000	0.000	0.169	0.408
8	0.000	2.408	0.911	0.000	1.643	0.000	0.368	0.000	0.251	0.000	0.000	0.000	0.289
9	1.178	1.717	0.220	1.503	0.976	0.000	0.030	0.775	0.000	0.853	0.000	0.000	0.000
10	0.487	1.468	0.000	0.812	0.533	1.159	0.832	0.274	0.681	1.469	2.703	0.000	0.549
Total	4.287	5.998	3.587	2.941	8.486	3.923	1.229	1.622	2.358	2.663	2.703	5.869	4.135

Pump Capacity of 10 m^3/s at 5-year Return Period

Unit: $10^6 m^3$

Day	Simulated Date and Year												
	1993	1988	1991	1992	1985	1986	1982	1983	1984	1989	1987	1990	1981
	4/28- 5/7	1/17- 1/26	3/7- 3/16	10/25- 11/3	11/27- 12/6	4/29- 5/8	12/15- 12/24	11/26- 12/5	4/7- 4/16	11/29- 12/8	4/22- 4/3	11/19- 11/28	4/29- 5/8
1	0.000	0.232	0.000	0.102	0.000	1.261	0.000	0.000	0.000	0.146	0.000	0.580	0.000
2	1.257	0.000	0.977	0.006	0.000	0.397	0.000	0.000	0.000	0.000	0.000	0.000	0.000
3	0.393	0.000	0.113	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	1.409	0.00[illegible]
4	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	1.037	0.000
5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.304	0.000	0.000	0.000	0.480	1.617
6	0.000	0.000	0.596	0.000	2.828	0.000	0.000	0.000	0.788	0.000	0.000	0.000	0.753
7	0.000	0.000	0.000	0.000	1.988	0.000	0.000	0.000	0.120	0.000	0.000	0.000	0.000
8	0.000	2.235	0.660	0.000	1.124	0.000	0.195	0.000	0.000	0.000	0.000	0.000	0.000
9	1.005	1.371	0.000	1.330	0.285	0.000	0.000	0.602	0.000	0.681	0.000	0.000	0.000
10	0.141	0.950	0.000	0.466	0.000	0.986	0.629	0.000	0.508	1.123	2.530	0.000	0.377
Total	2.796	4.789	2.347	1.904	6.225	2.645	0.824	0.906	1.416	1.950	2.530	3.506	2.747

Table VI.6.2 (5/5) SIMULATION RESULTS OF STORED RUNOFF VOLUME FOR DIFFERENT PUMP CAPACITY AT EACH RETURN PERIOD

Pump Capacity of 20 m^3/s at 5-year Return Period

Unit: $10^6 m^3$

Day	Simulated Date and Year												
	1993	1988	1991	1992	1985	1986	1982	1983	1984	1989	1987	1990	1981
	4/28-5/7	1/17-1/26	3/7-3/16	10/25-11/3	11/27-12/6	4/29-5/8	12/15-12/24	11/26-12/5	4/7-4/16	11/29-12/8	4/22-4/3	11/19-11/28	4/29-5/8
1	0.000	0.000	0.000	0.000	0.000	0.397	0.000	0.000	0.000	0.000	0.000	0.000	0.000
2	0.393	0.000	0.113	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.545	0.000
4	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.753
6	0.000	0.000	0.000	0.000	1.964	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	0.000	0.000	0.000	0.260	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	1.371	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.141	0.000	0.000	0.466	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
10	0.000	0.000	0.000	0.000	0.000	0.122	0.000	0.000	0.000	0.000	1.666	0.000	0.000
Total	0.534	1.371	0.113	0.466	2.224	0.520	0.000	0.000	0.000	0.000	1.666	0.545	0.753

Pump Capacity of 30 m^3/s at 5-year Return Period

Unit: $10^6 m^3$

Day	Simulated Date and Year												
	1993	1988	1991	1992	1985	1986	1982	1983	1984	1989	1987	1990	1981
	4/28-5/7	1/17-1/26	3/7-3/16	10/25-11/3	11/27-12/6	4/29-5/8	12/15-12/24	11/26-12/5	4/7-4/16	11/29-12/8	4/22-4/3	11/19-11/28	4/29-5/8
1	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
4	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
6	0.000	0.000	0.000	0.000	1.100	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
7	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	0.000	0.507	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
10	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.802	0.000	0.000
Total	0.000	0.507	0.000	0.000	1.100	0.000	0.000	0.000	0.000	0.000	0.802	0.000	0.000

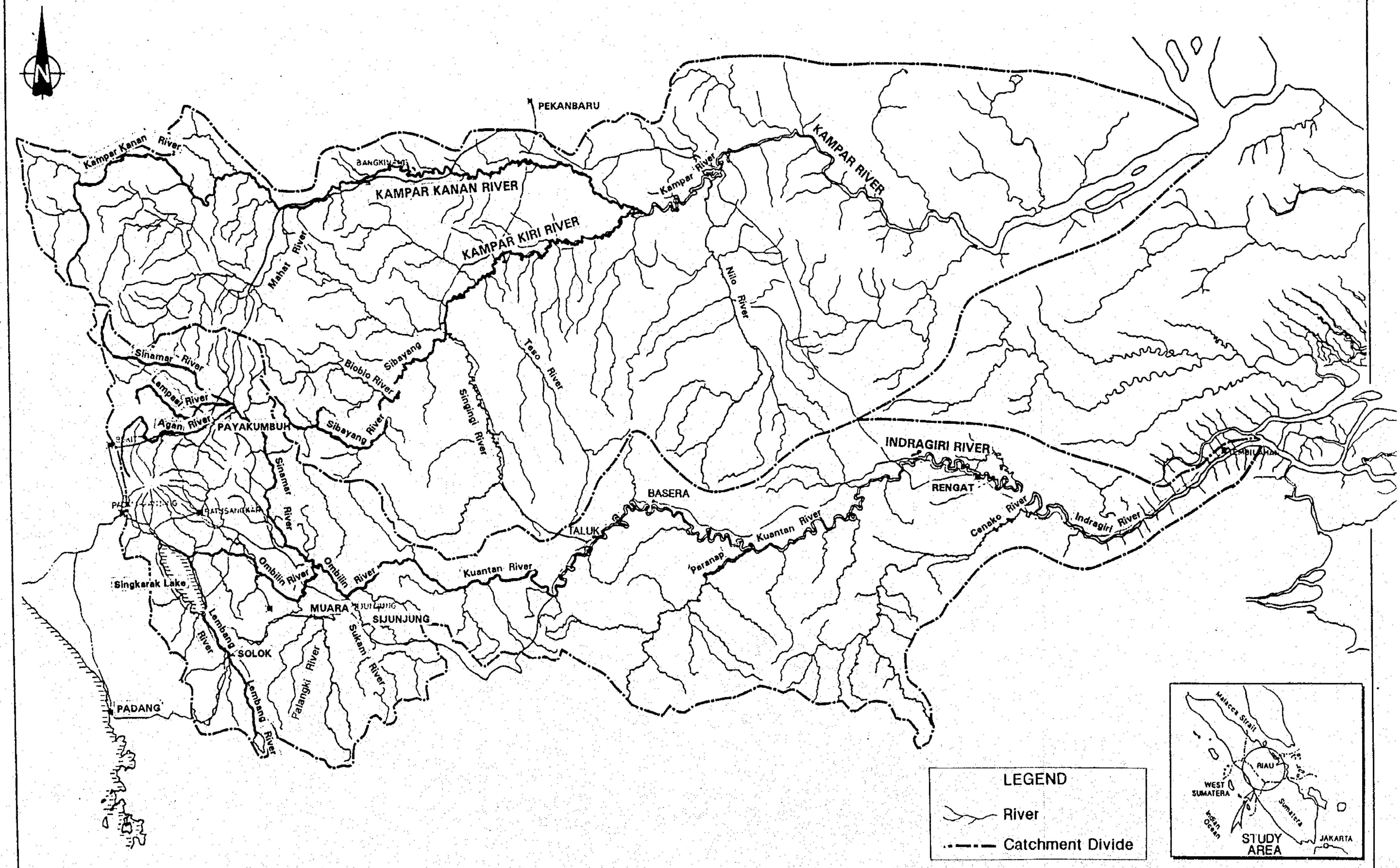
Table VI.6.3 AVERAGE ANNUAL FLOOD DAMAGE FOR DIFFERENT PUMP CAPACITY AT EACH RETURN PERIOD

Unit in Damage: Rp. 10^6

Pump Capacity	Return Period (T)	Probability of Rainfall Being Equaled or Exceeded in Any Year (N = 1/T)	Expected Damage (L_n)	Average Damage ($L_{AV} = 1/2[L_n + L_{n+1}]$)	Probability of Rainfall in Interval ΔN	Average Annual Flood Damage $L_{AV} * \Delta N$
No Pump	-	1.0	0	-	-	-
	2	0.5	5,210	2,605	0.5	1,302
	5	0.2	5,700	5,455	0.3	1,636
	Total					2,939
1 m^3/s	-	1.0	0	-	-	-
	2	0.5	4,819	2,410	0.5	1,205
	5	0.2	5,154	4,986	0.3	1,496
	Total					2,701
3 m^3/s	-	1.0	0	-	-	-
	2	0.5	3,447	1,723	0.5	862
	5	0.2	4,111	3,779	0.3	1,134
	Total					1,995
5 m^3/s	-	1.0	0	-	-	-
	2	0.5	3,445	1,722	0.5	861
	5	0.2	4,111	3,778	0.3	1,133
	Total					1,994
8 m^3/s	-	1.0	0	-	-	-
	2	0.5	3,127	1,563	0.5	782
	5	0.2	3,447	3,287	0.3	986
	Total					1,768
10 m^3/s	-	1.0	0	-	-	-
	2	0.5	3,127	1,563	0.5	782
	5	0.2	3,447	3,287	0.3	986
	Total					1,768
20 m^3/s	-	1.0	0	-	-	-
	2	0.5	2,471	1,235	0.5	618
	5	0.2	2,700	2,585	0.3	776
	Total					1,393
30 m^3/s	-	1.0	0	-	-	-
	2	0.5	1,262	631	0.5	316
	5	0.2	1,859	1,561	0.3	468
	Total					784

FIGURES

VI FLOOD CONTROL PLAN



SCALE: 1/1,000,000

0 10 20 30 40 50 Km

THE STUDY ON
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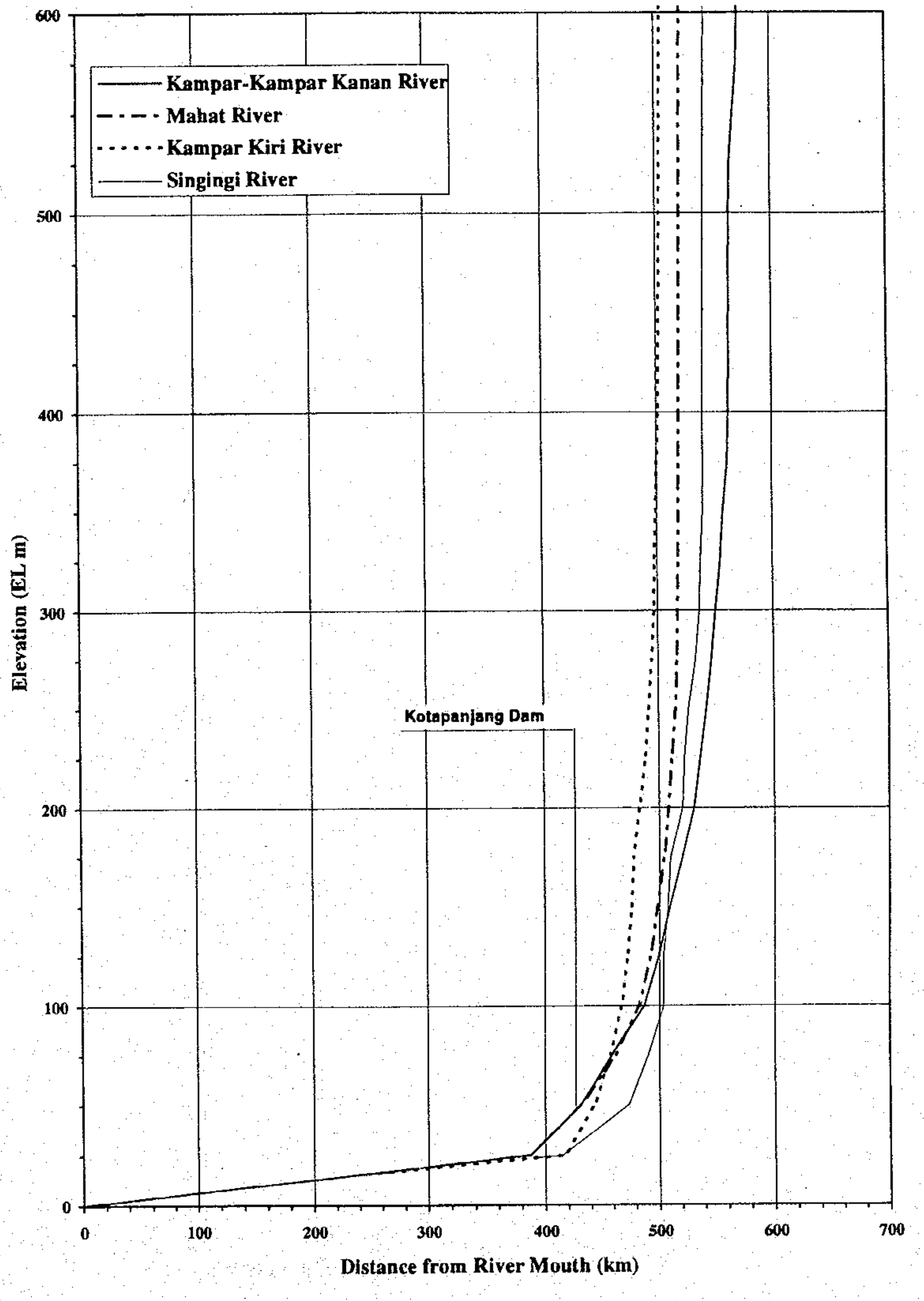
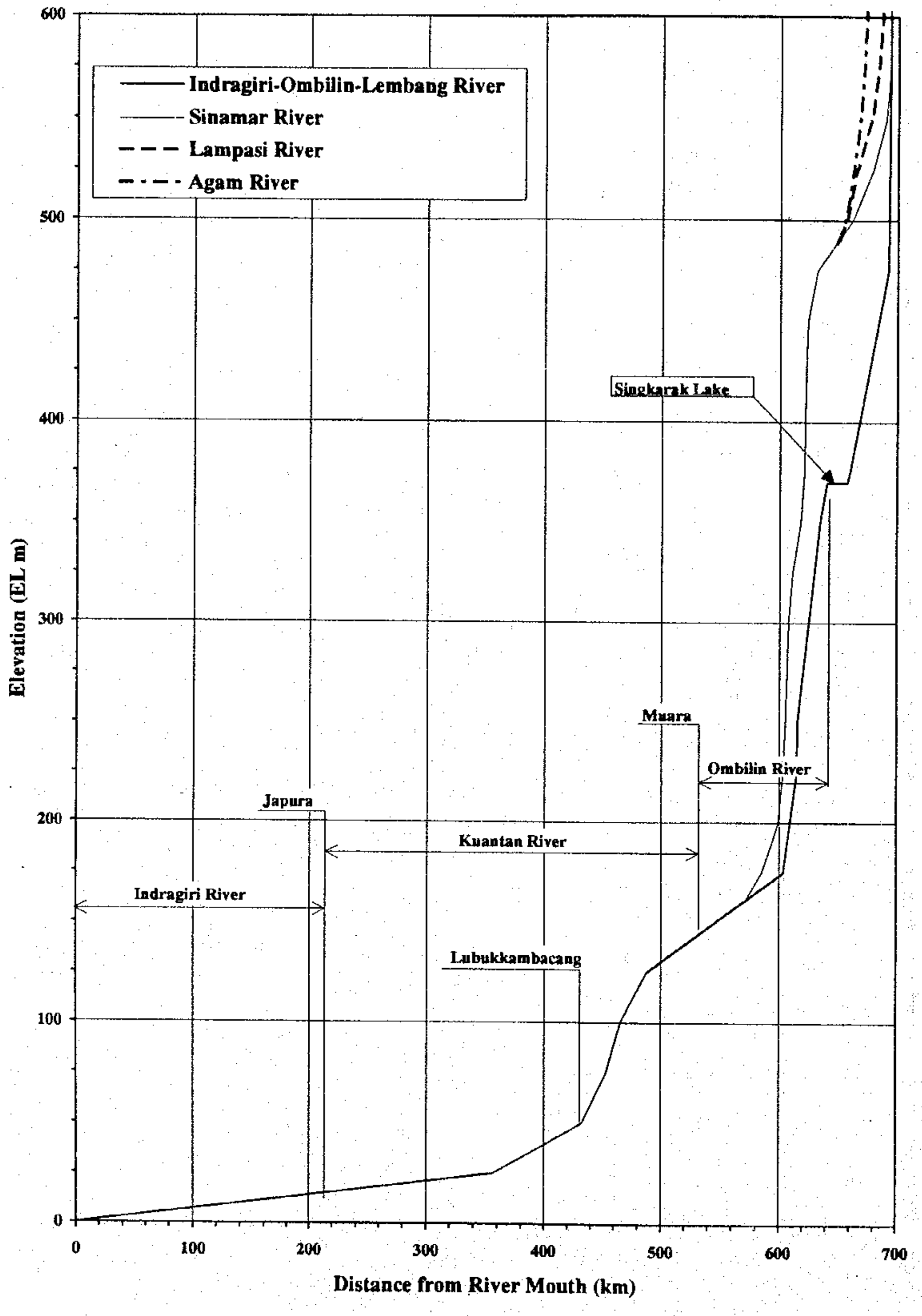
Fig. VI.1.1 KAMPAR AND INDRAGIRI RIVER SYSTEM

Kampar River System



THE STUDY ON KAMPAR-INDRAGIRI RIVER BASIN DEVELOPMENT PROJECT IN THE REPUBLIC OF INDONESIA	Fig. VI.1.2 (1/2) LONGITUDINAL PROFILE OF RIVERS (PRESENT CONDITION)
JAPAN INTERNATIONAL COOPERATION AGENCY	

Indragiri River System



THE STUDY ON KAMPAR-INDRAGIRI RIVER BASIN DEVELOPMENT PROJECT IN THE REPUBLIC OF INDONESIA	Fig. VI.1.2 (2/2)	LONGITUDINAL PROFILE OF RIVERS (PRESENT CONDITION)
JAPAN INTERNATIONAL COOPERATION AGENCY		

Kampar Kanan River

NO	D-NAME	Q(m^3/s)
1	0-K	1393.9
2	1-K	1540.0
3	2-K	1292.2
4	3-K	1231.9
5	4-K	625.9
6	5-K	702.1
7	6-K	740.0
8	7-K	760.0
9	8-K	707.6
10	9-K	639.0
11	10-K	730.5
12	11-K	664.7
13	12-K	644.1
14	13-K	679.2
15	14-K	724.1
16	15-K	1747.5
17	16-K	643.3
18	17-K	563.3
19	18-K	599.8
20	19-K	631.1
21	20-K	714.9
22	21-K	739.2
23	22-K	821.8
24	23-K	911.8
25	24-K	930.2
26	25-K	930.8
27	26-K	1139.7
28	27-K	779.7
29	28-K	885.5
30	29-K	815.3
31	30-K	668.3
32	31-K	580.0
33	32-K	568.0
34	33-K	838.4
35	34-K	5717.0
36	35-K	958.0
37	36-K	1084.0
38	37-K	919.9
39	38-K	1097.3
40	39-K	1171.6
41	40-K	973.3
42	41-K	1197.2
43	42-K	947.1
44	43-K	961.0
45	44-K	1020.8
46	45-K	986.2
47	46-K	1322.6
48	47-K	931.5
49	48-K	1254.5
50	49-K	1041.4
51	50-K	4141.7
52	51-K	1126.9
53	52-K	1389.2
54	53-K	677.0
55	54-K	816.6
56	55-K	850.5
57	56-K	662.3
58	57-K	1001.8
59	58-K	1470.9
60	59-K	19878.5
61	60-K	7033.7
62	61-K	7151.6
63	62-K	5156.2

Kampar Kiri River

NO	D-NAME	Q(m^3/s)
1	0-K	3452.2
2	1-K	3629.3
3	2-K	139.1
4	3-K	132.2
5	4-K	201.9
6	5-K	186.1
7	6-K	135.5
8	7-K	165.4
9	8-K	221.3
10	9-K	160.5
11	10-K	186.3
12	11-K	200.3
13	12-K	271.3
14	13-K	238.6
15	14-K	290.4
16	15-K	286.1
17	16-K	315.9
18	17-K	326.1
19	18-K	347.0
20	19-K	359.8
21	20-K	285.1
22	21-K	594.7
23	22-K	559.2
24	23-K	562.3
25	24-K	741.8
26	25-K	764.5
27	26-K	929.8
28	27-K	716.6
29	28-K	697.3
30	29-K	793.6
31	30-K	1271.8
32	31-K	3337.0
33	32-K	550.2
34	33-K	746.6
35	34-K	686.0
36	35-K	444.7
37	36-K	550.7
38	37-K	569.7
39	38-K	278.5
40	39-K	514.9
41	40-K	1021.9
42	41-K	613.2
43	42-K	473.6
44	43-K	605.9
45	44-K	1905.4
46	45-K	2149.1
47	46-K	2464.9

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Fig. VI.1.3 (1/4) FLOW CAPACITY OF PRESENT RIVER CHANNEL

Middle and Lower Reaches of Indragiri River

NO	D-NAME	Q(m^3/s)
1	0-K	896.1
2	1-K	1228.9
3	2-K	924.4
4	3-K	1154.9
5	4-K	1238.6
6	5-K	1298.2
7	6-K	1487.0
8	7-K	1354.1
9	8-K	1174.2
10	9-K	954.2
11	10-K	1246.7
12	11-K	959.4
13	12-K	1867.6
14	13-K	1192.8
15	14-K	8634.4
16	15-K	1254.7
17	16-K	1412.4
18	17-K	1073.7
19	18-K	1197.0
20	19-K	1519.0
21	20-K	755.0
22	21-K	1275.3
23	22-K	1548.0
24	23-K	1357.2
25	24-K	1638.8
26	25-K	1598.7
27	26-K	1082.4
28	27-K	2061.2
29	28-K	1842.4
30	29-K	1983.0
31	30-K	2133.9
32	31-K	1056.4
33	32-K	1165.3
34	33-K	1334.6
35	34-K	1905.8
36	35-K	1824.6
37	36-K	1330.1
38	37-K	1651.1
39	38-K	1415.8
40	39-K	1683.0
41	40-K	1185.4
42	41-K	1543.6
43	42-K	1196.2
44	43-K	1870.8
45	44-K	977.9
46	45-K	1279.4
47	46-K	1015.5
48	47-K	1415.0
49	48-K	1267.0
50	49-K	1107.1
51	50-K	1018.0
52	51-K	1175.6
53	52-K	1093.8
54	53-K	930.8
55	54-K	1082.4
56	55-K	1438.1
57	56-K	1711.7
58	57-K	1654.2
59	58-K	637.2
60	59-K	936.0
61	60-K	1173.1
62	61-K	1284.6
63	62-K	1113.5
64	63-K	786.5
65	64-K	1252.6
66	65-K	1481.4
67	66-K	1471.1
68	67-K	1420.1
69	68-K	10844.6
70	69-K	2126.9
71	70-K	1915.6
72	71-K	863.7
73	72-K	1061.3
74	73-K	915.0
75	74-K	1879.2
76	75-K	1672.8
77	76-K	781.8
78	77-K	1093.0
79	78-K	4717.2
80	79-K	1875.9
81	80-K	749.5
82	81-K	1918.9
83	82-K	1605.5
84	83-K	1470.2
85	84-K	723.9
86	85-K	1459.3
87	86-K	7849.8
88	87-K	1221.9
89	88-K	1272.0
90	89-K	1067.1
91	90-K	1189.3
92	91-K	967.8
93	92-K	1732.1
94	93-K	3293.1
95	94-K	2002.0
96	95-K	1340.4
97	96-K	1814.6

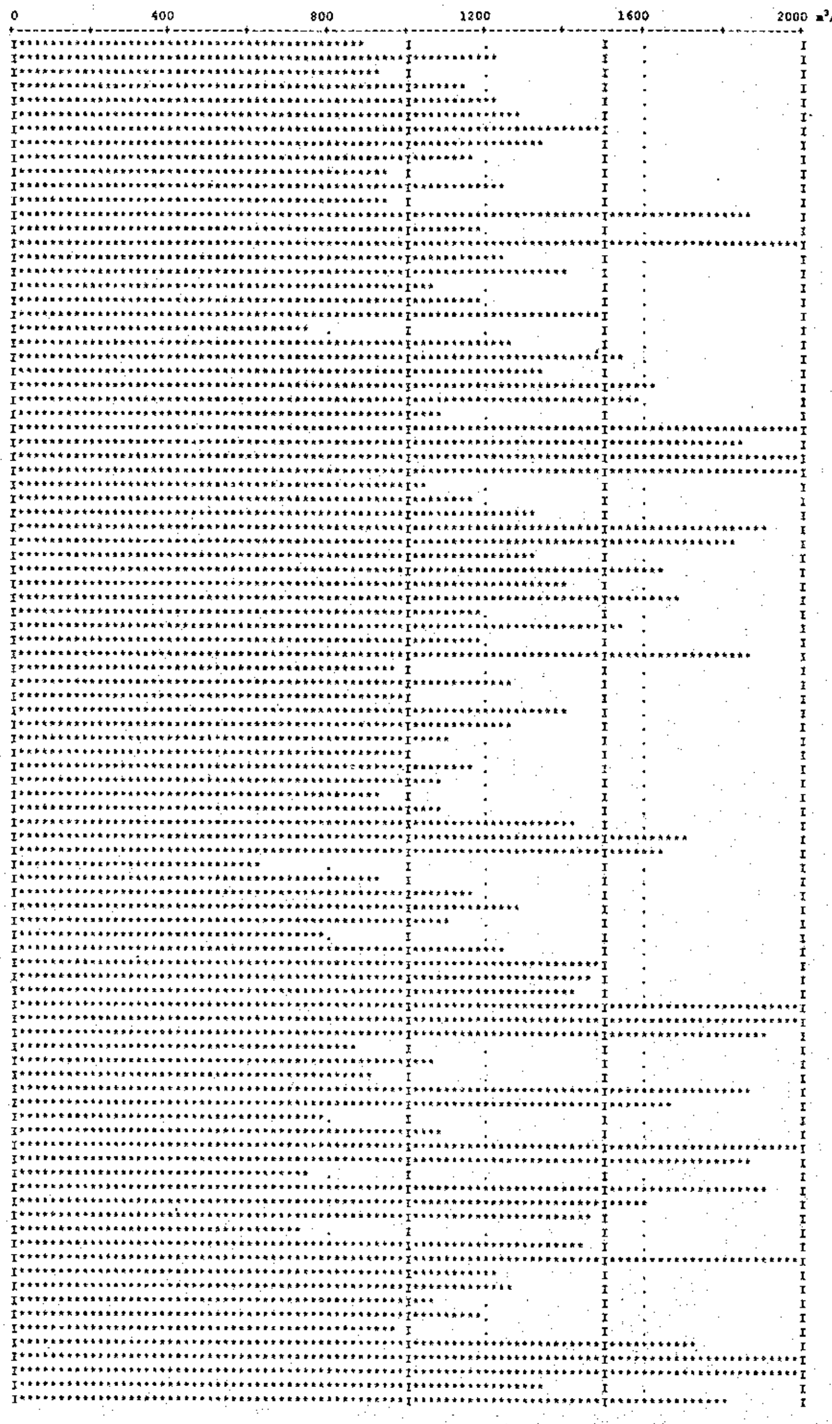

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Fig. VI.1.3 (2/4) FLOW CAPACITY OF PRESENT RIVER CHANNEL

Sinamar River

NO	D-NAME	Q(m^3/s)
1	0-K	291.5
2	1-K	583.8
3	2-K	201.0
4	3-K	464.1
5	4-K	411.6
6	5-K	936.6
7	6-K	437.6
8	7-K	347.4
9	8-K	512.0
10	9-K	437.6
11	10-K	359.5
12	11-K	435.7
13	12-K	430.5
14	13-K	331.2
15	14-K	281.4
16	15-K	170.3
17	16-K	1194.4
18	17-K	252.0
19	18-K	396.7
20	19-K	227.1
21	20-K	1192.8
22	21-K	446.8
23	22-K	539.5
24	23-K	1310.4
25	24-K	363.2
26	25-K	1273.4
27	26-K	283.5
28	27-K	210.8
29	28-K	190.0
30	29-K	237.8
31	30-K	448.0
32	31-K	175.2
33	32-K	172.4
34	33-K	184.4
35	34-K	372.1
36	35-K	249.2
37	36-K	172.6
38	37-K	313.0
39	38-K	625.2
40	39-K	318.3
41	40-K	.0
42	41-K	284.6
43	42-K	712.7
44	43-K	129.9
45	44-K	135.3
46	45-K	586.4

Agam River

NO	D-NAME	Q(m^3/s)
1	0-K	384.6
2	1-K	566.9
3	2-K	386.3
4	3-K	537.1
5	4-K	338.3
6	5-K	806.8
7	6-K	407.7
8	7-K	303.7
9	8-K	1128.0
10	9-K	215.3
11	10-K	1750.9
12	11-K	322.4
13	12-K	379.8
14	13-K	653.6
15	14-K	423.1
16	15-K	127.5
17	16-K	803.1
18	17-K	764.5

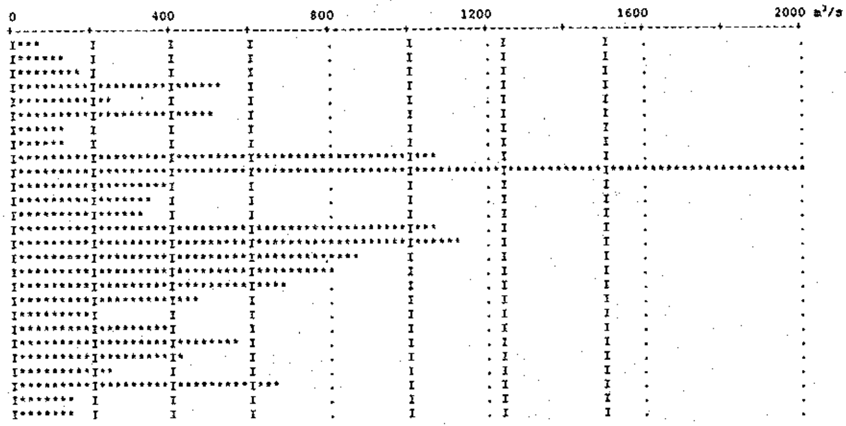
Lampasi River

NO	D-NAME	Q(m^3/s)
1	0-K	1904.3
2	1-K	240.0
3	2-K	815.4
4	3-K	115.6
5	4-K	265.3
6	5-K	624.2
7	6-K	26.3
8	7-K	73.4
9	8-K	384.4
10	9-K	46.1
11	10-K	87.5
12	11-K	83.5
13	12-K	78.1
14	13-K	70.3
15	14-K	57.5
16	15-K	73.6
17	16-K	140.2
18	17-K	306.6
19	18-K	692.5

THE STUDY ON KAMPAR-INDRAGIRI RIVER BASIN DEVELOPMENT PROJECT IN THE REPUBLIC OF INDONESIA	Fig. VI.1.3 (3/4) FLOW CAPACITY OF PRESENT RIVER CHANNEL
JAPAN INTERNATIONAL COOPERATION AGENCY	

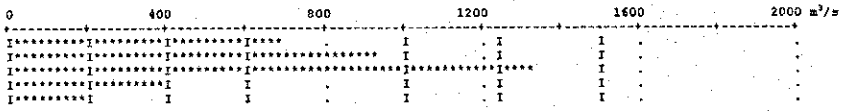
Lembang River

NO	D-NAME	Q(m^3/s)
1	0-K	76.8
2	1-K	127.5
3	2-K	164.7
4	3-K	539.1
5	4-K	243.5
6	5-K	509.0
7	6-K	122.1
8	7-K	131.1
9	8-K	1069.4
10	9-K	3194.1
11	10-K	412.7
12	11-K	349.2
13	12-K	324.1
14	13-K	1071.0
15	14-K	1132.4
16	15-K	862.7
17	16-K	817.8
18	17-K	690.3
19	18-K	464.8
20	19-K	214.2
21	20-K	387.1
22	21-K	568.2
23	22-K	435.3
24	23-K	243.3
25	24-K	664.1
26	25-K	157.8
27	26-K	152.7



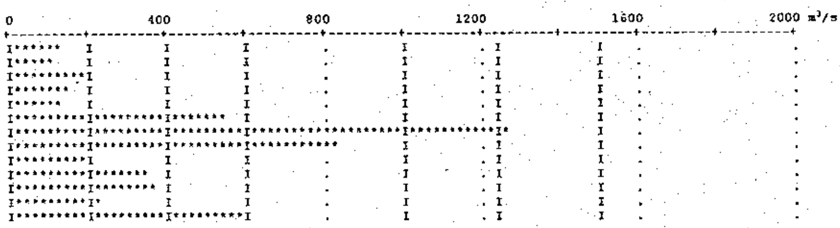
Sumani River

NO	D-NAME	Q(m^3/s)
1	0-K	683.5
2	1-K	922.0
3	2-K	1335.1
4	3-K	409.8
5	4-K	203.8



Sukam River

NO	D-NAME	Q(m^3/s)
1	0-K	138.6
2	1-K	117.2
3	2-K	190.1
4	3-K	147.3
5	4-K	128.8
6	5-K	550.4
7	6-K	1279.1
8	7-K	831.3
9	8-K	207.9
10	9-K	355.4
11	10-K	377.2
12	11-K	236.3
13	12-K	588.7



THE STUDY ON KAMPAR-INDRAGIRI RIVER BASIN DEVELOPMENT PROJECT IN THE REPUBLIC OF INDONESIA	Fig. VI.1.3 (4/4) FLOW CAPACITY OF PRESENT RIVER CHANNEL
JAPAN INTERNATIONAL COOPERATION AGENCY	

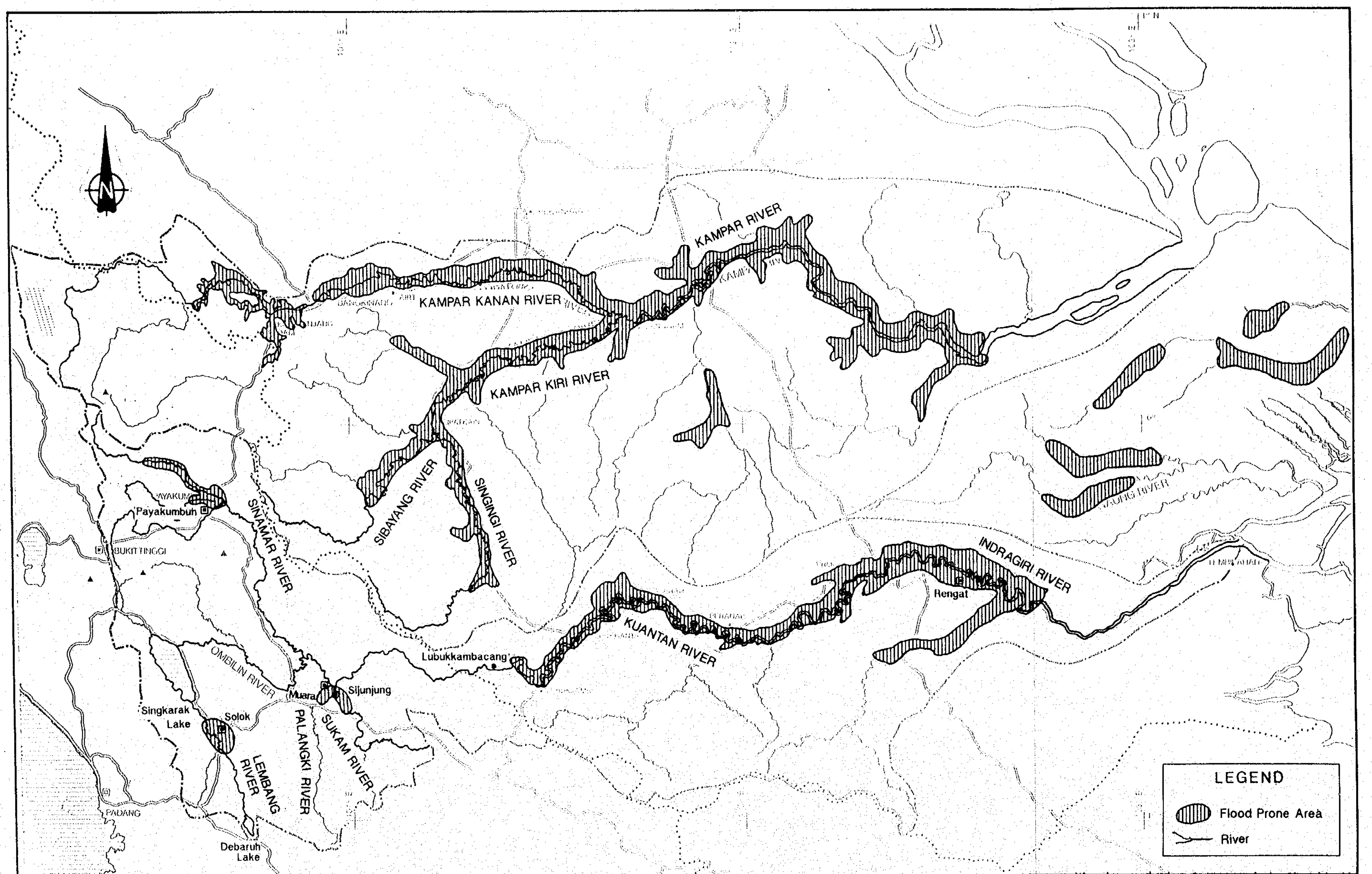


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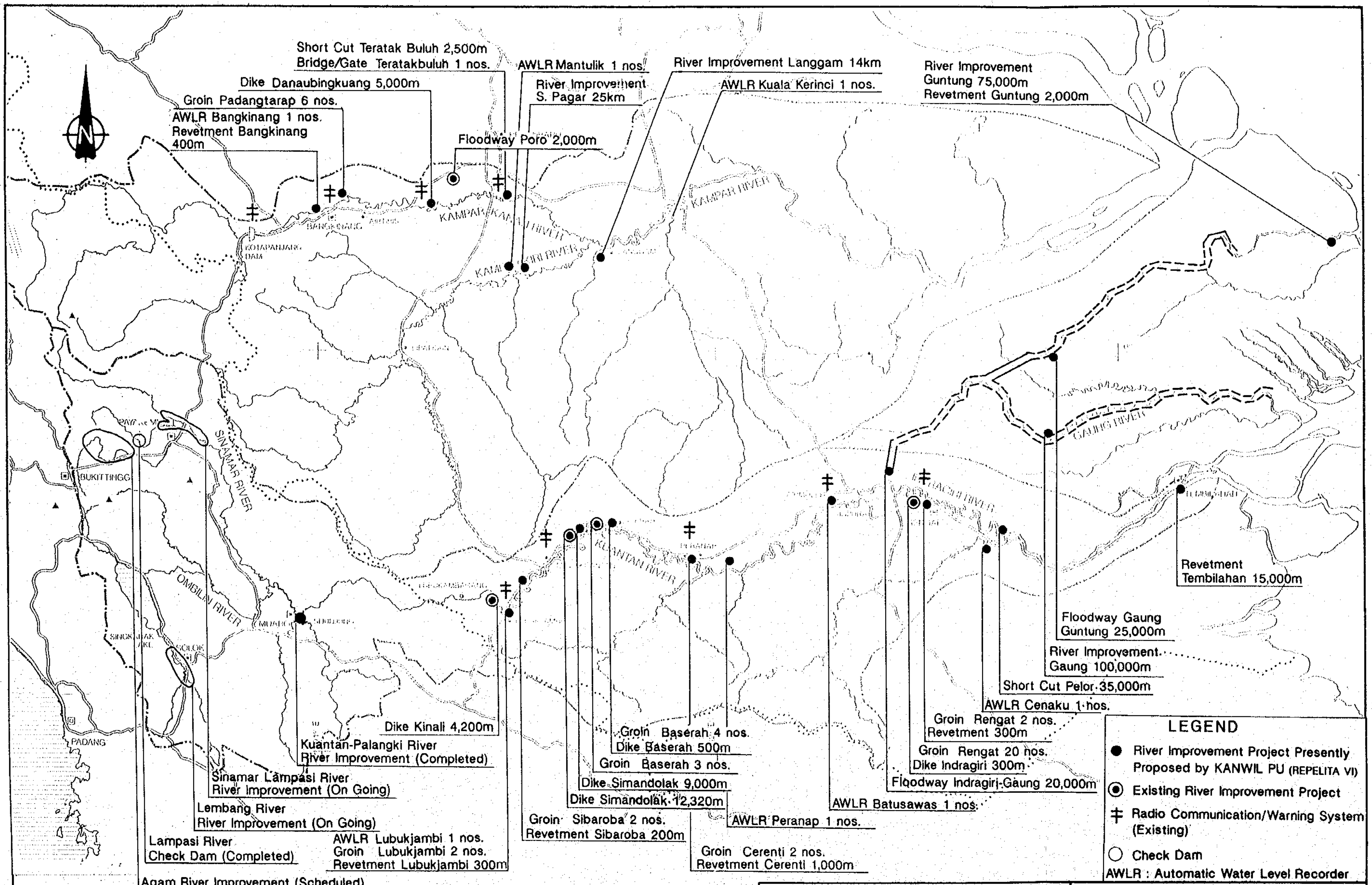
Fig. VI.2.1 FLOOD PRONE AREAS OF KAMPAR AND INDRAGIRI RIVER BASINS



SCALE: 1/1,000,000

0 10 20 30 40 50km

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Fig. VI.3.1 EXISTING AND PROPOSED RIVER IMPROVEMENT PROJECTS

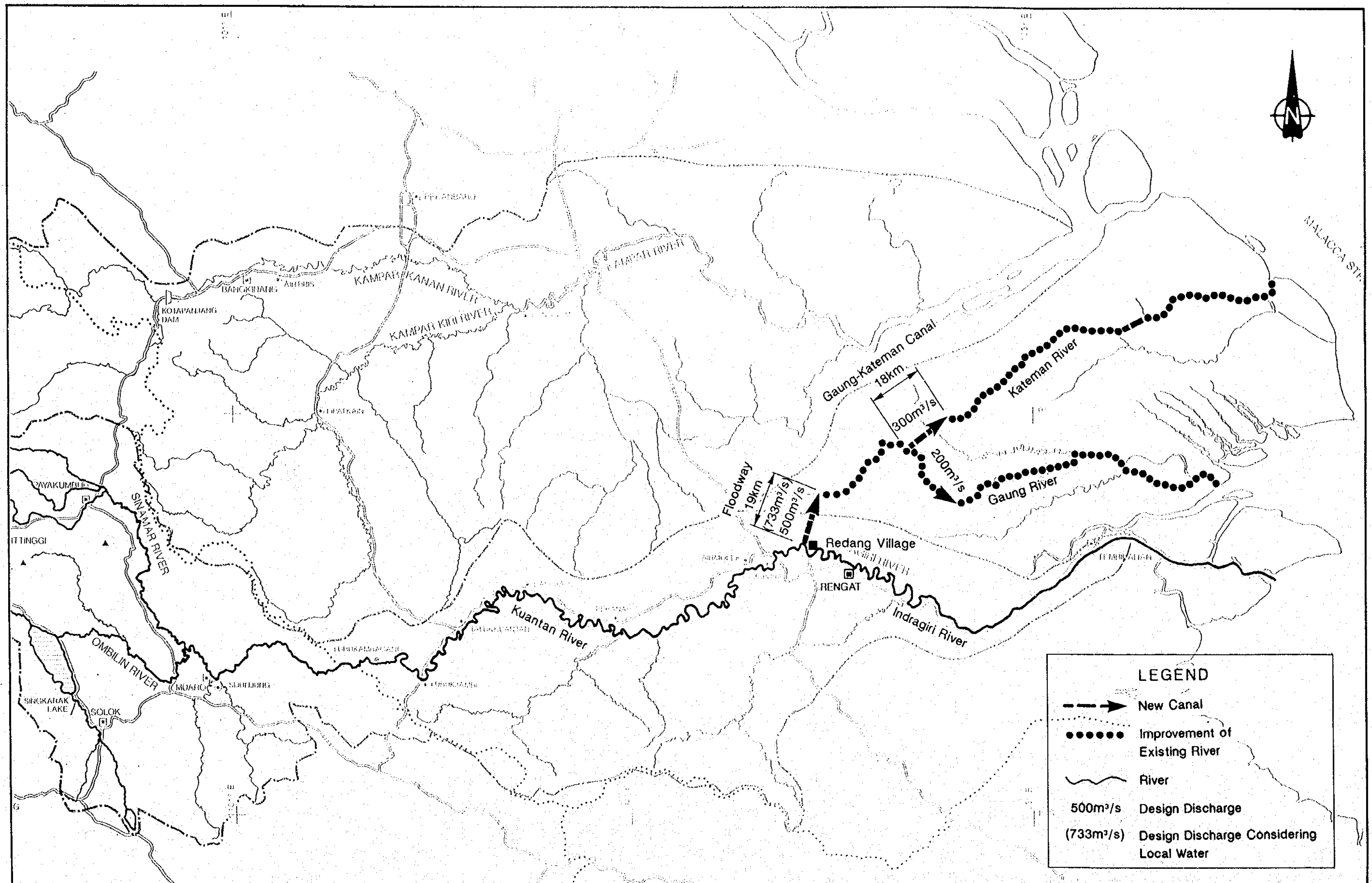

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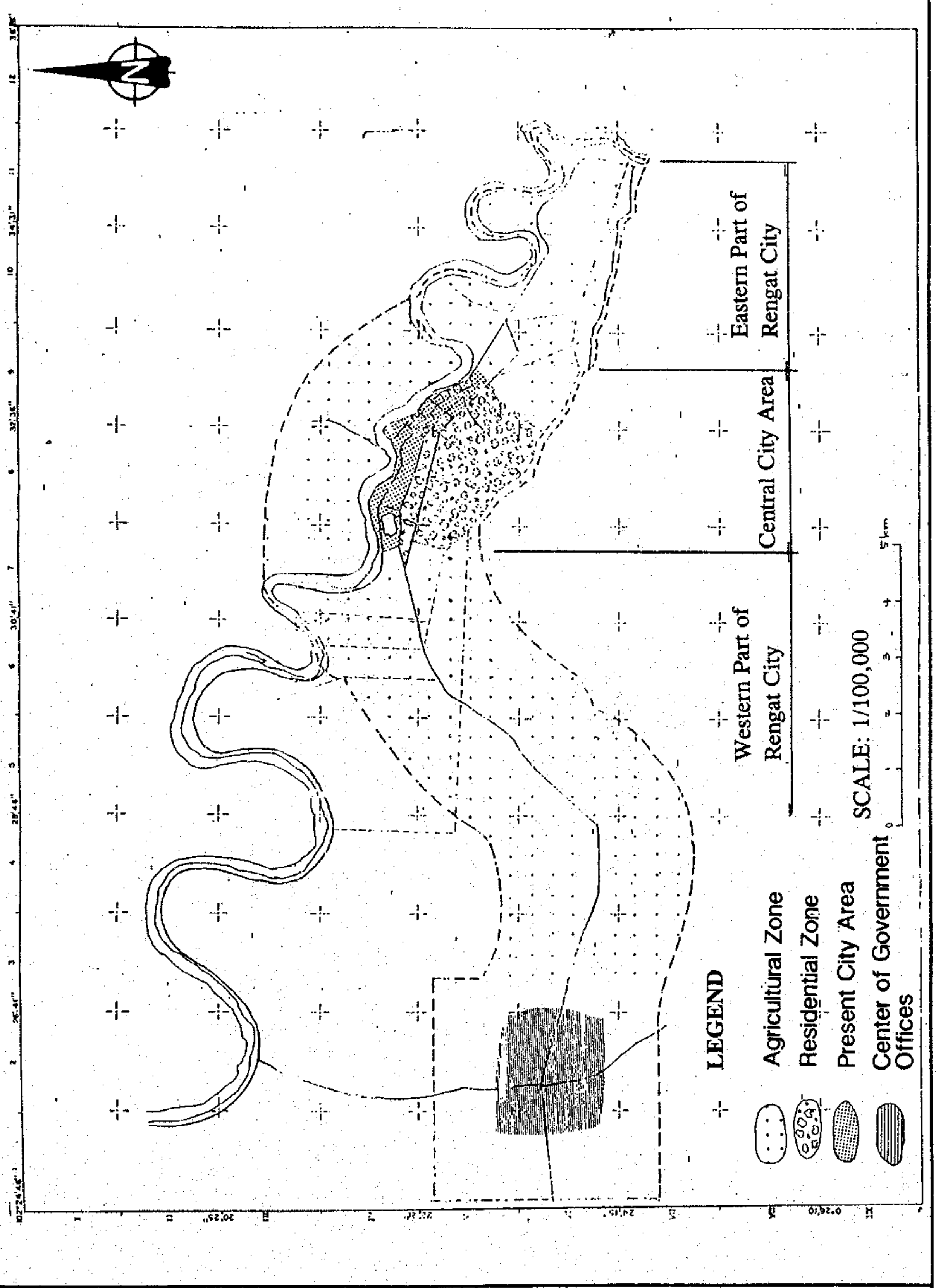
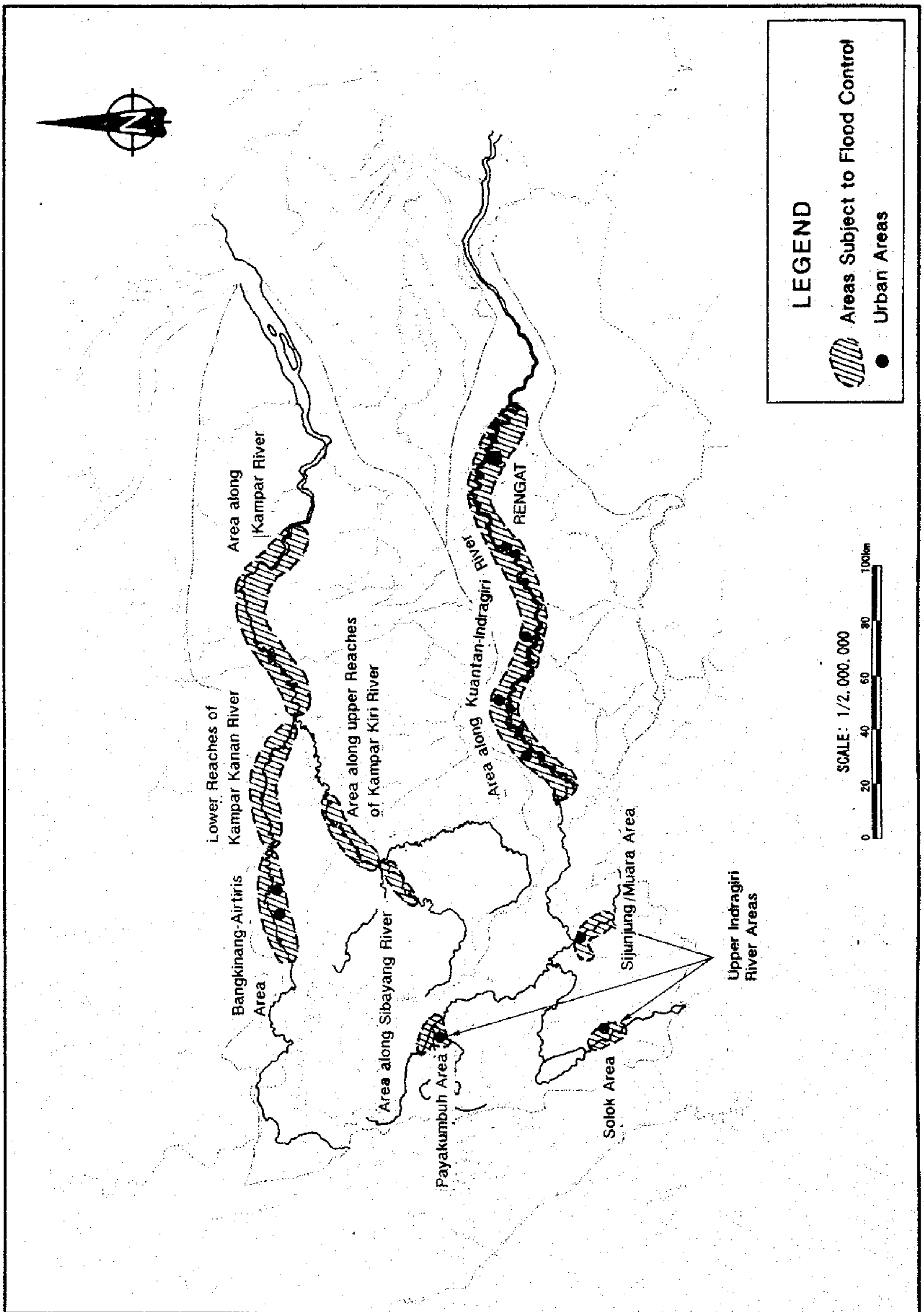
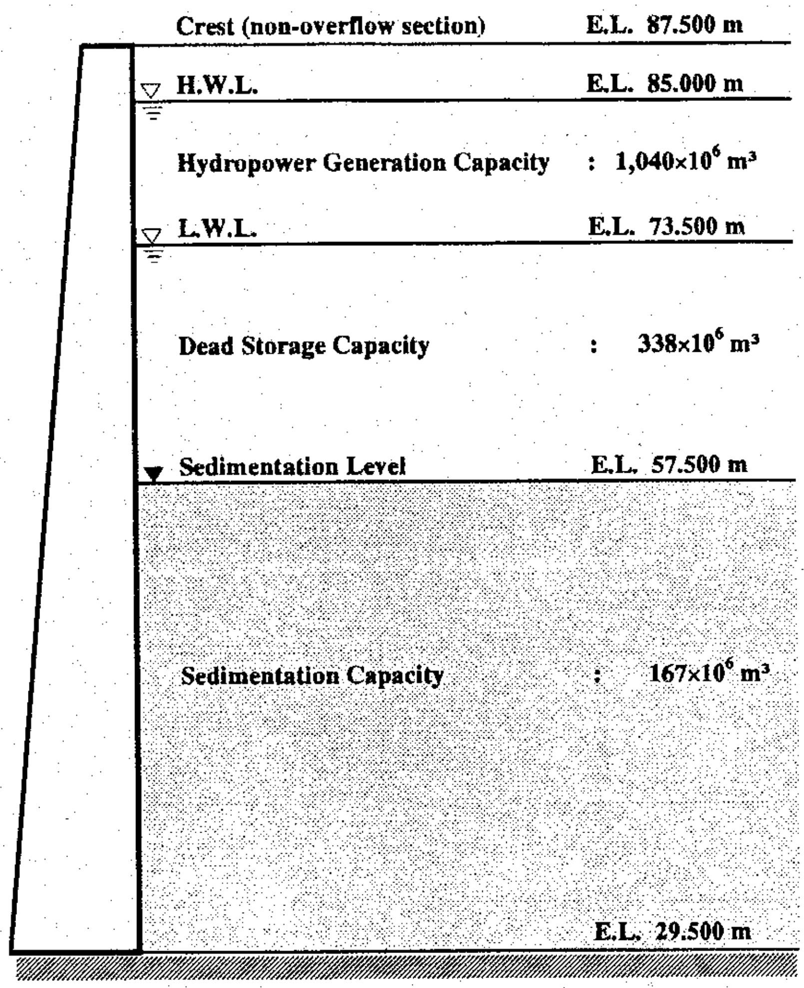
Fig. VI.3.2 GENERAL LAYOUT OF PROPOSED FLOODWAY AND RELATED RIVER IMPROVEMENT IN KAMPAR-INDRAGIRI DELTA AREA



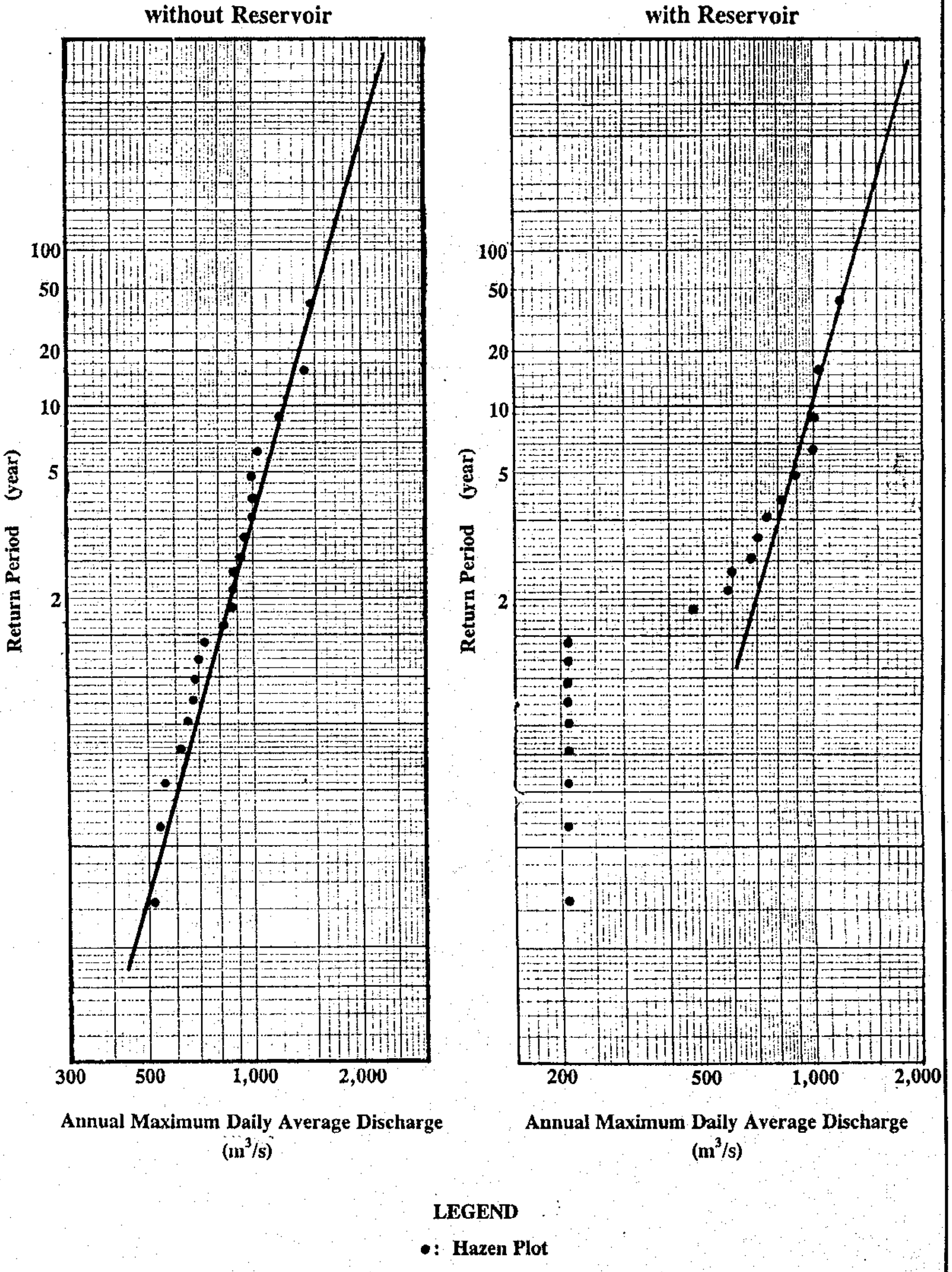
THE STUDY ON KAMPAR-INDRAGIRI RIVER BASIN DEVELOPMENT PROJECT IN THE REPUBLIC OF INDONESIA	Fig. VI.3.3 CITY LAYOUT PLAN OF RENGAT CITY
JAPAN INTERNATIONAL COOPERATION AGENCY	



THE STUDY ON KAMPAR-INDRAGIRI RIVER BASIN DEVELOPMENT PROJECT IN THE REPUBLIC OF INDONESIA	Fig. VI.4.1 AREAS SUBJECT TO FLOOD CONTROL
JAPAN INTERNATIONAL COOPERATION AGENCY	



THE STUDY ON KAMPAR-INDRAGIRI RIVER BASIN DEVELOPMENT PROJECT IN THE REPUBLIC OF INDONESIA	Fig. VI.5.1 RESERVOIR CAPACITY ALLOCATION OF KOTAPANJANG DAM
JAPAN INTERNATIONAL COOPERATION AGENCY	



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IN THE REPUBLIC OF INDONESIA

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. VI.5.2 PROBABILITY PLOT FOR ANNUAL MAXIMUM DAILY DISCHARGES WITH/WITHOUT KOTAPANJANG RESERVOIR

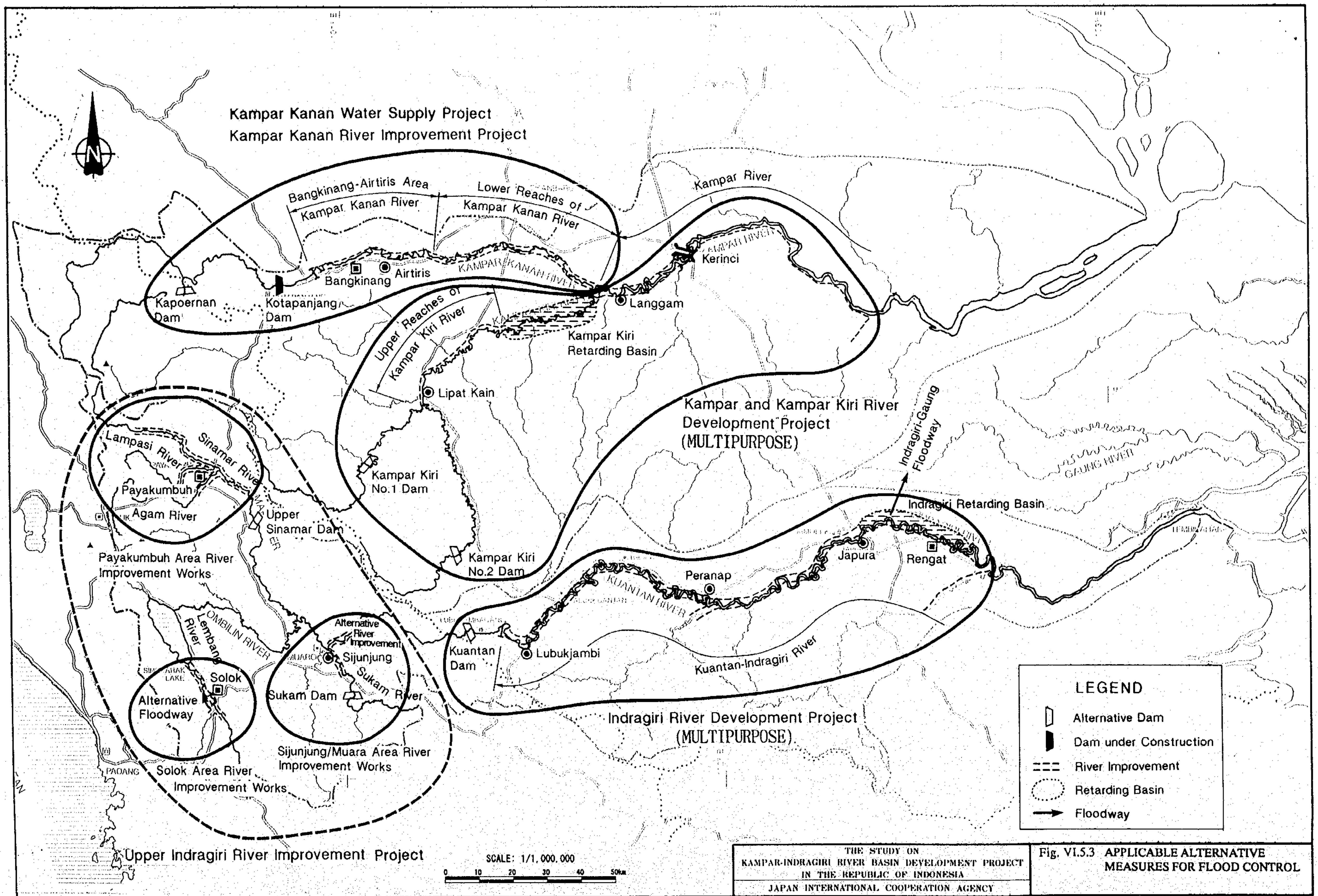

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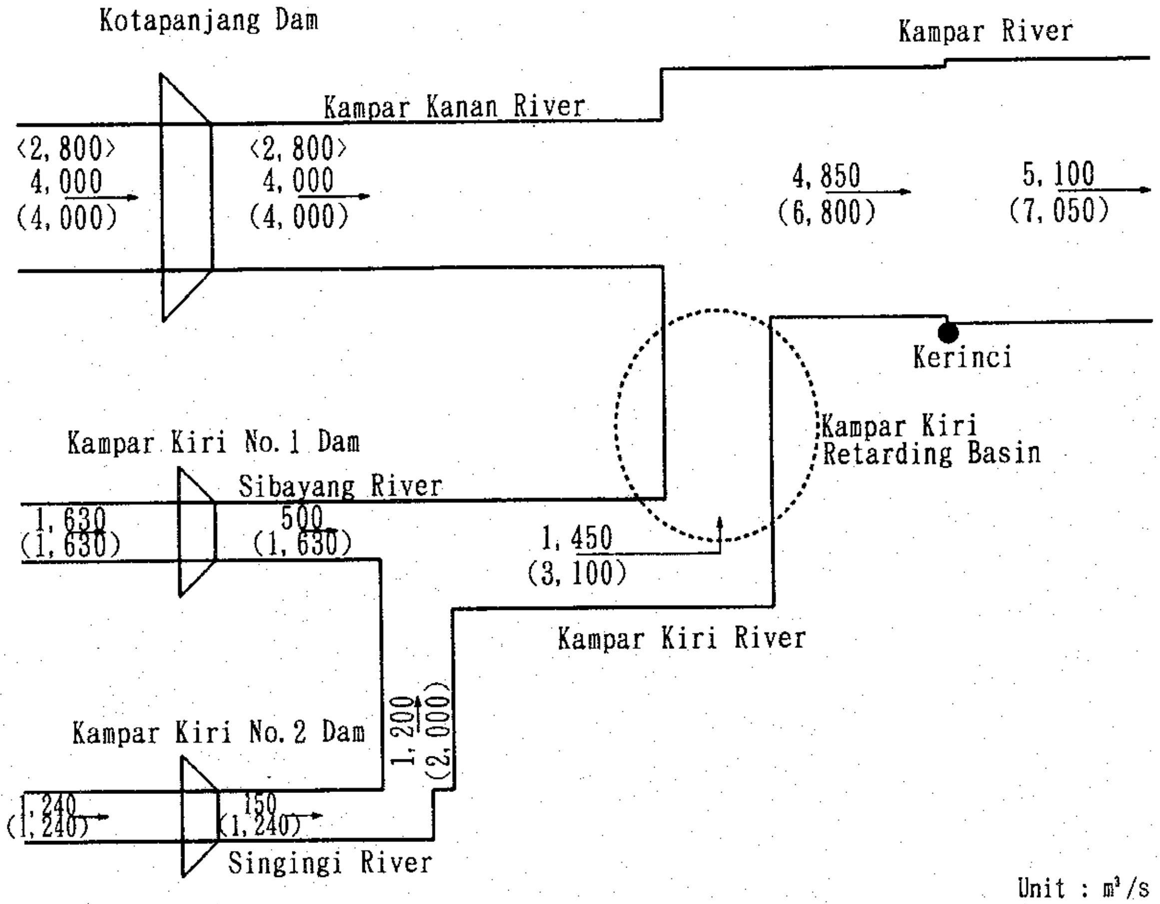
Fig. VI.5.3 APPLICABLE ALTERNATIVE MEASURES FOR FLOOD CONTROL

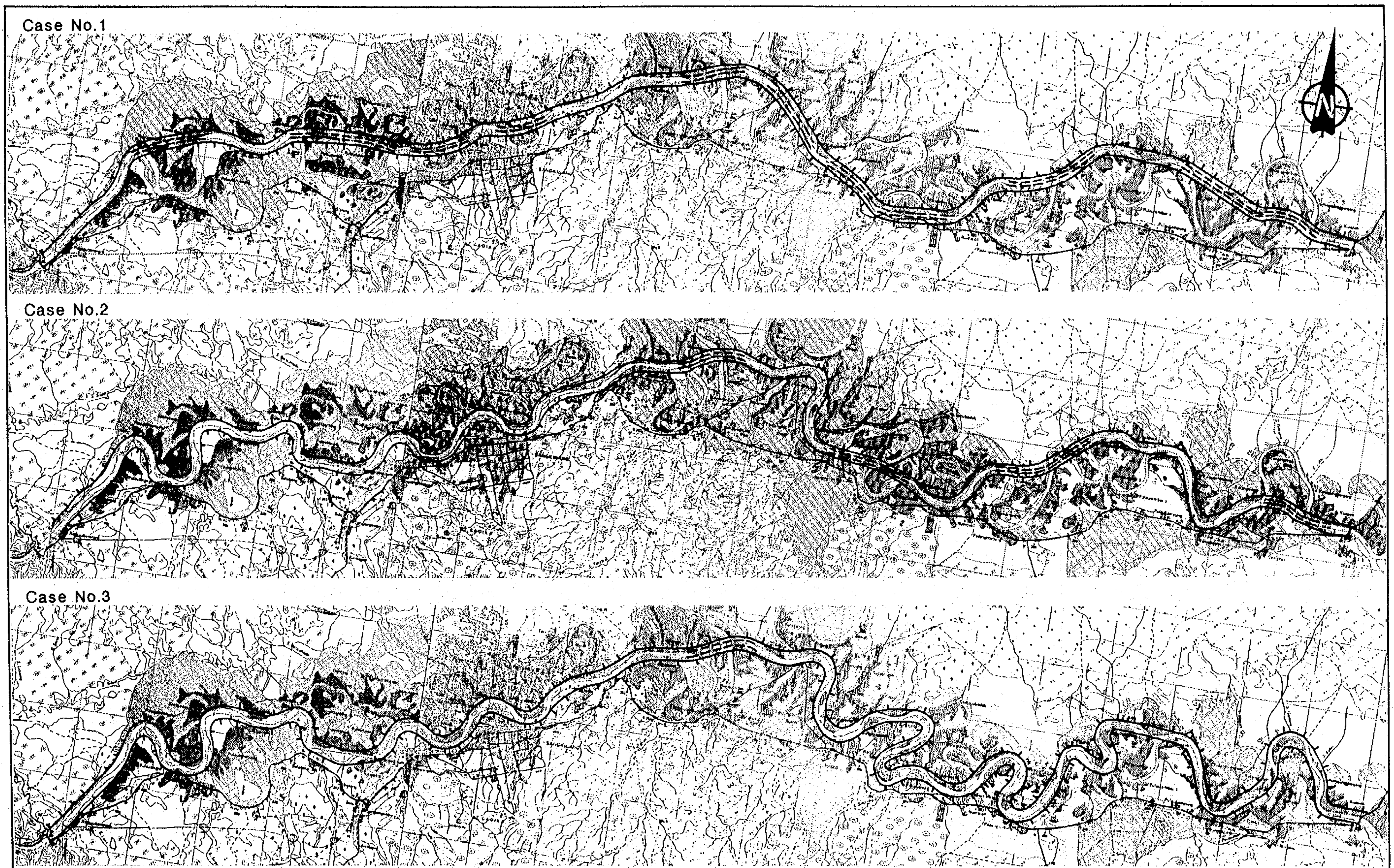
Kampar River



Unit : m³/s

Note : Design Scale: -No Parentheses: 50-year Return Period
-< >: 10-year Return Period
Figures in Parentheses are standard design discharges.

THE STUDY ON KAMPAR-INDRAGIRI RIVER BASIN DEVELOPMENT PROJECT IN THE REPUBLIC OF INDONESIA	Fig. VI.5.4 DISTRIBUTION OF DESIGN DISCHARGES FOR KAMPAR RIVER SYSTEM
JAPAN INTERNATIONAL COOPERATION AGENCY	



LEGEND

Embankment

Shortcut

SCALE: 1/100,000

0 1 2 3 4 5km

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Fig. VI.5.5 ALTERNATIVE ALIGNMENT FOR KAMPAR KANAN RIVER IMPROVEMENT PLAN

	Case 1		Case 2		Case 3	
Alignment	Create shortest channels with many shortcuts		Sortcuts at heavily meandering parts		Shortcuts only at extremely meandering parts	
Particulars	Quantity	Cost	Quantity	Cost	Quantity	Cost
Unit	km	$Rp.\times10^9$	km	$Rp.\times10^9$	km	$Rp.\times10^9$
Embankment	100.0	340.0	113.0	384.2	142.0	482.8
Shortcut	48.5	310.4	24.8	158.7	10.3	65.9
Total		650.4		542.9		548.7

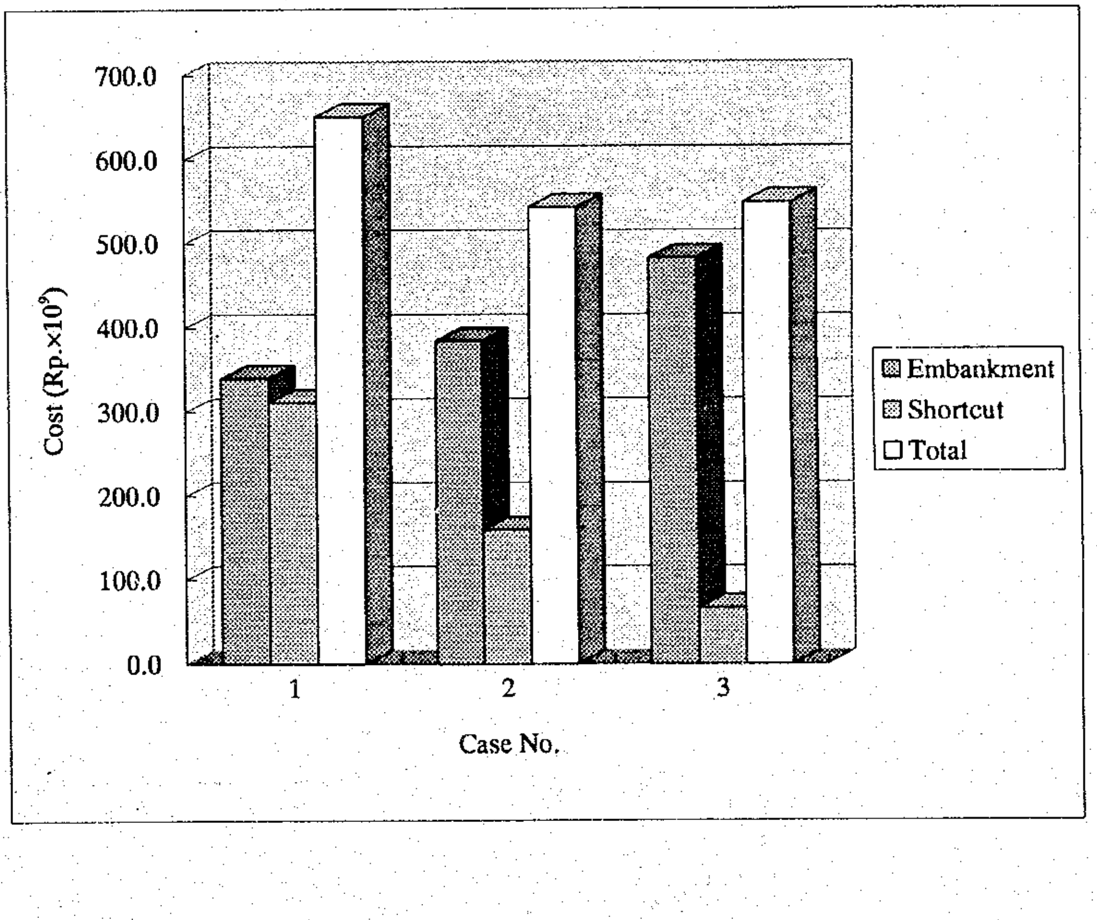
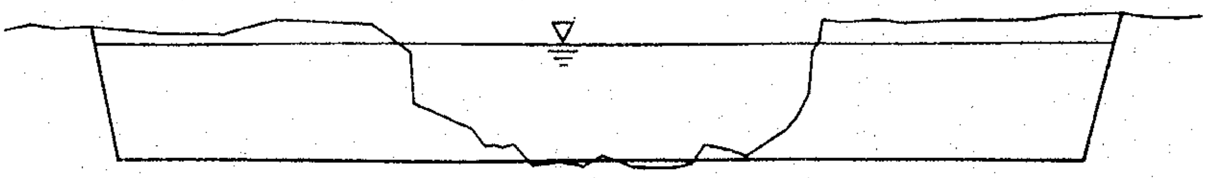
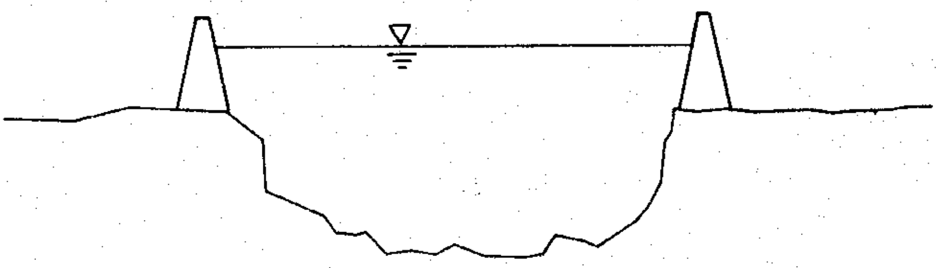


Fig. VI.5.6 COST COMPARISON FOR ALTERNATIVE ALIGNMENT OF KAMPAR KANAN RIVER IMPROVEMENT PLAN

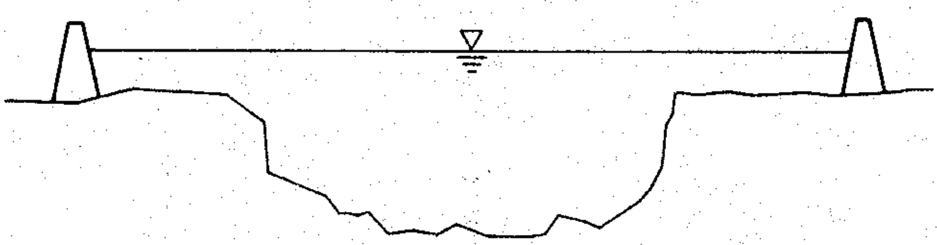
Case 1 Excavated Channel



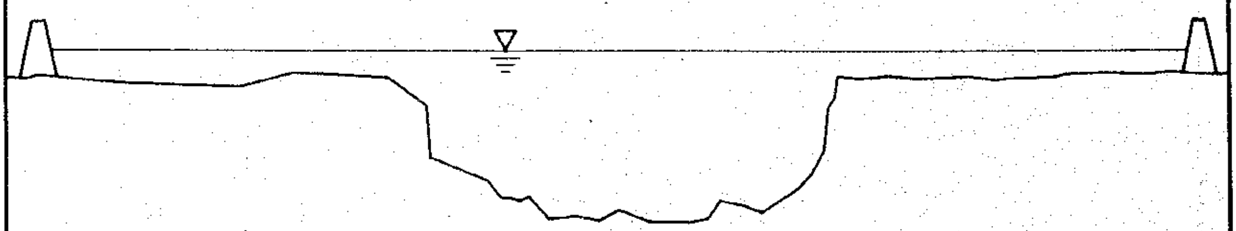
Case 2 Narrow and High Embankment Channel



Case 3 Intermediate of Case 2 and Case 4



Case 4 Wide and Low Embankment Channel



THE STUDY ON KAMPAR-INDRAGIRI RIVER BASIN DEVELOPMENT PROJECT IN THE REPUBLIC OF INDONESIA	Fig. VI.5.7 ALTERNATIVE CROSS SECTIONS FOR KAMPAR KANAN RIVER IMPROVEMENT PLAN
JAPAN INTERNATIONAL COOPERATION AGENCY	

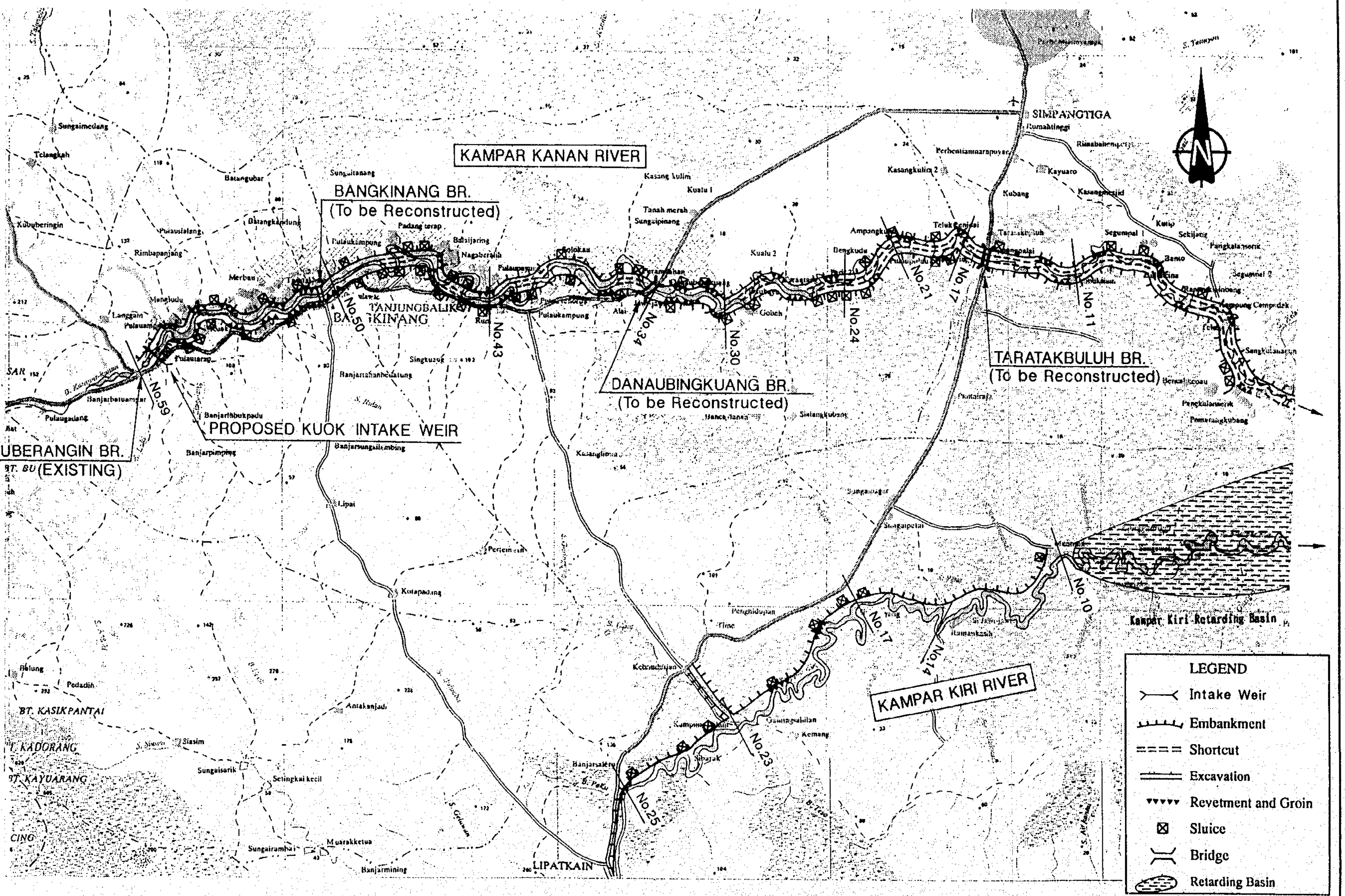


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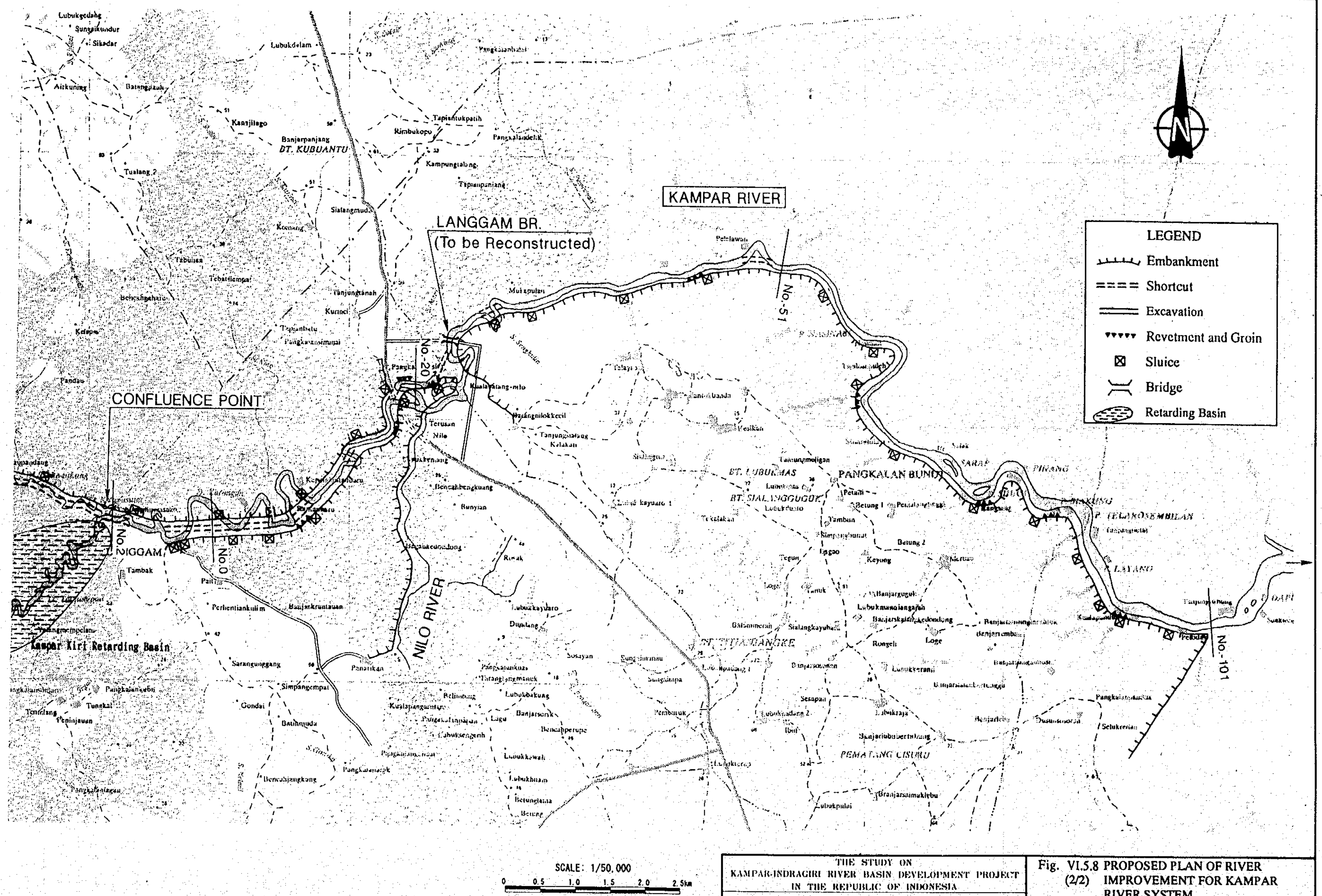
Fig. VI.5.8 (1/2) PROPOSED PLAN OF RIVER IMPROVEMENT FOR KAMPAR RIVER SYSTEM



THE STUDY ON KAMPAR-INDRAGIRI RIVER BASIN DEVELOPMENT PROJECT IN THE REPUBLIC OF INDONESIA	Fig. VI.5.8 (2/2) PROPOSED PLAN OF RIVER IMPROVEMENT FOR KAMPAR RIVER SYSTEM
JAPAN INTERNATIONAL COOPERATION AGENCY	