JAPAN INTERNATIONAL COOPERATION AGENCY (JICA)

DIRECTORATE GENERAL OF WATER RESOURCES DEVELOPMENT
MINISTRY OF PUBLIC WORKS
THE REPUBLIC OF INDONESIA

THE STUDY ON KAMPAR-INDRAGIRI RIVER BASIN DEVELOPMENT PROJECT

VOLUME 3

SUPPORTING REPORT (1)

(FINAL REPORT)

DECEMBER 1995

CTI ENGINEERING CO., LTD.
IN ASSOCIATION WITH
NIPPON KOEI CO., LTD.

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CHAPTER 1 AVAILABILITY OF DATA

1.1 Meteorological and Rainfall Data

The inventory of meteorological and rainfall gaging stations and data available in and around the study area are given in Table I.1.1. The locations of these stations are as indicated in Fig. I.1.1. The following data were collected from these stations and the number of stations from which these data were collected are summarized in the table below.

Data Collected	Organization			Total
	DPU	BMG	DR-SB	
General MeteorologicalData	0	3	1	4
Daily Rainfall Data	22	3	1	26
Hourly Rainfall Data	4	2	1	7

DPU : Ministry of Public Works
BMG : Center of Meteorology and Geophysics
DR-SB : Ministry of Agriculture

1.2 Discharge Data

A total of 13 water level gaging stations are maintained by DPU in the Kampar and Indragiri river basins. The inventory and data available are as given in Table I.1.2. The locations of these stations are as indicated in Fig. I.1.1.

CHAPTER 2 METEOROLOGICAL AND HYDOROLOGICAL CONDITIONS

2.1 Climate

The study area is located in the provinces of Riau and West Sumatra in the central part of Sumatra Island which lies along the Malayan Peninsula. This region is in the southern part of Southeast Asia and lies in the Intertropical Zone. Since the climate of Southeast Asia is controlled by the Asian monsoon system, climate of study area is under the monsoon's influence.

Asian monsoons consist of two main seasons and two inter-monsoon periods; namely, the northeast monsoon season, the southwest monsoon season and the two inter-monsoon periods.

The northeast monsoon season, the rainy season, approximately lasts from November to March and winds are very constant in direction. Average rainfall and surface wind direction in January typical for the northeast monsoon in Southeast Asia are shown in Fig. I.2.1(B). The rainy season in the region in and around the study area continue during this period, because the northeast winds coming across the South China Sea bring humid air which causes rainfall on the eastern slopes of the Barisan Mountains, the backbone of Sumatra Island.

The southwest monsoon season, called the dry season, lasts from about June to September when the whole region is under the influence of southwesterly winds, as shown in Fig. I.2.1(A). Southwest monsoon winds are generally weaker than northeast monsoon winds. The southwest monsoon winds are humidified by the Indian Ocean and cause rainfall on the western slopes of the Barisan Mountains, but the weaker winds do not cause heavy rainfall.

2.2 Meteorology

Climatic conditions of the study area are sumarized in Table I.2.1 and monthly climatological data are shown in Table I.2.2 and Fig. I.2.2. The climatic characteristics are discussed below.

<u>Temperature</u>

The monthly mean temperature is almost constant throughout the year. The difference between months is small; much smaller than the fluctuation in daily temperature. Variation in altitude is more pronounced than the horizontal variation which is constant. The temperature in the plain area of the study area (lower than EL. 50 m above sea level) is approximately 26°C on an average, while it is 21°C in the mountainous area. The variation in maximum and minimum temperatures from the average temperature is from 4° and 9° in both mountainous and plain areas..

Wind

Wind direction is dominated by monsoons. In the rainy season from November to March, the northeast winds blow constantly and the humid air causes rainfall. The monthly mean velocity of wind is from 6.8 to 7.4 knots in Pekanbaru. In the dry season from June to September, the southwest or south winds blow with velocity lower than 7 knots.

Data on hourly wind velocity are available at only one station in the study area, the Pekanbaru Airport Station. The observation term is 21 years from 1973 to 1993. The monthly maximum hourly mean wind velocity is shown in Table I.2.3 and in this record the maximum data (64 knots) is selected as the representative value and 64 knots is equivalent to 33 m/s. This value is used to calculate wave height at dam reservoir surface.

Humidity

The monthly mean humidity is always high and almost constant throughout the year at approximately 83%. High humidity throughout the year is one of the factors causing rainfall in the dry season.

Evaporation

Annual evaporation is approximately 1,500 mm and monthly evaporation ranges between 130 mm in the dry season and 110 mm in the rainy season. The variation in monthly evaporation is from 99 to 143 mm/month in mountainous areas and from 105 to 137 mm/month in plain areas. Evaporation in the plain areas is constant and a little higher than in the mountainous areas.

2.3 Rainfall

Average annual rainfall is about 2,000 to 2,700 mm in almost all of the study area except east coastal areas. In the northeast monsoon season, humid winds come to cause rainfall in the northeast side of the Barisan Mountains. The isohyetal map of annual rainfall is presented in Fig. I.2.3. The northeast area of the Barisan Mountains inside of the Kampar river basin is the zone where rainfalls concentrate with about 3,000 mm/year.

December and January are months of typical climatic conditions of the northeast monsoon and the past large floods have occurred in these months. The eastern plains of Sumatra Island have rainfall of at least 100 mm/month even in dry seasons and this condition is very different from the condition in other islands of Indonesia.

The duration of rainfall is approximately 2 to 12 hours. In general, rainfall starts in the afternoon and stops before sunrise.

2.4 Discharge

River Discharge

Discharges observed by automatic water level recorders (AWLR) in the lower stretch of the Kampar Kanan, Kampar Kiri and Indragiri rivers are shown in Fig. I.2.4. The monthly average flow discharge of each gaging station is as given in the following table.

Unit: m^3/s

River/Station	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Ave.
Kampar Kanan/Danau Bingkuang	379	298	305	308	245	143	109	97	143	168	293	463	246
Kampar Kiri/Lipat Kain	301	259	264	327	274	132	85	73	111	148	283	389	221
Indragiri/ Pulau Berhala	467	447	462	491	463	201	148	180	214	173	288	568	342

Records of Major Floods

The five major floods that occurred at the three stations in recent years are summarized in the following table.

River	Kampar Kanan		Kampar Kiri		Indragiri	
Station	Danau Bingkuang		Lipat Kain		Pulau Berhala	
Catchment Area (km^2)	4,000		3,431		8,526	
No.	Date of Flood	Peak Q (m^3/s)	Date of Flood	Peak Q (m^3/s)	Date of Flood	Peak Q (m^3/s)
1	Dec. 5, 1978	2,516	Jan. 17, 1989	1,254	Jan. 8, 1986	3,850
2	Jan. 20, 1989	1,845	Dec. 3, 1990	1,197	Dec. 29, 1991	2,250
3	Dec.17, 1991	1,710	Dec. 27, 1991	1,114	Jan. 1, 1992	1,401
4	Jan. 19, 1991	1,539	Jan. 7, 1986	1,103	Jan. 24, 1989	1,342
5	Jan. 7, 1986	1,516	Jan. 4, 1991	1,085	Jan. 27, 1988	1,342

CHAPTER 3 RAINFALL ANALYSIS

3.1 Procedure of Rainfall Analysis

Rainfall analysis has been conducted in line with the following procedures, each one of which are described in the following section.

- Identification of necessary rainfall data
- Selection of rainfall gaging stations
- Supplementation of lacking rainfall data
- Calculation of average basin rainfall
- Calculation of probable rainfall
- Establishment of rainfall depth-duration curve of short duration
- Analysis of rainfall patterns during past notable floods
- Determination of design rainfall
- Calculation of probable maximum precipitation (PMP)

3.2 Identification of Necessary Rainfall Data

In this study, daily and hourly rainfall data are identified to be necessary for flood runoff and low flow analyses, as mentioned below.

Daily rainfall data are necessary:

- for the calculation of probable rainfall for the flood runoff analysis;
- for the estimation of temporal and spatial distribution of design rainfall for the flood runoff analysis; and
- for the low flow analysis.

Hourly rainfall data are necessary:

- for the estimation of temporal and spatial distribution of design rainfall for the flood runoff analysis; and
- for the preparation of rainfall depth and duration curve in short duration.

3.3 Selection of Rainfall Gaging Stations

There are 26 rainfall gaging stations in and around the study area. Of these 26 stations, 19 stations for the daily rainfall data and 4 stations for hourly rainfall data are selected considering the location of station and the completeness of data. The selected stations are listed together with their annual rainfall values in Table I.3.1.

3.4 Supplementation of Lacking Rainfall Data

The collected data were found to be lacking in rainfall data, which were supplemented for both long-term and short-term rainfalls by procedures mentioned below. Long-term rainfall is defined as the daily rainfall for one year and employed for the low flow analysis, while short-term rainfall is defined as the daily rainfall for several days during storms which cause flooding.

Long Term Rainfall

Supplementation for lacking periods was made by correlation with the neighboring stations for 12 years from 1981 to 1992.

Correlation of rainfall between each station has been obtained by linear equations for 1-day, 3-day, 7-day and monthly rainfalls (refer to Table I.3.2). As shown in this table, correlation coefficients for 1-day, 3-day and 7-day rainfalls are very low. This is because rainfalls in the basin are predominantly spot showers.

Accordingly, monthly rainfall correlation equations are applied to daily rainfall. The equations applied are given in Table I.3.3. Annual rainfalls after supplementation for each station are shown in Table I.3.4.

Short Term Rainfall

The procedure for long term rainfall was applied for short term rainfall during storms. The results are given in Table I.3.5

In the past floods, the daily rainfall correlation of stations located in the Barisan Mountains gave higher values of around 0.7. This means that rain which will cause a large flood fall in an area of more than 100 km in diameter.

3.5 Calculation of Average Basin Rainfall

Rainfalls observed or supplemented at each station are called point rainfall. The point rainfall at each station was converted to average basin rainfall by the Thiessen Polygon Method and the Area Reduction Factor. The average basin rainfall for low flow analysis was converted by the Thiessen Polygon Method for all sub-basins, while the average basin rainfall for the flood runoff analysis was converted by the Area Reduction Factro for sub-basins with the area of less than 500 km^2 and the Thiessen Polygon Method for sub-basins with the area of more than 500 km^2.

Thiessen Polygon Method

Mountains 1,000 to 2,000 m high (Mt. Gadang, Mt. Ngalautinggi, Mt. Solokjanjaang, Mt. Paninjauannanelok) separate the upper reaches of the Kampar and Indragiri river basins. The Indragiri river valley, mainly in West Sumatra Province, receives less rainfall due to the effect of the surrounding mountains; while, the Kampar River side, namely the northern to eastern side of the mountains, has a higher rainfall especially during the northeast monsoon season. Accordingly, the catchment boundary in this area was considered in formulating the Thiessen polygon.

Thiessen polygons considering the catchment boundary of the Kampar and Indragiri rivers are presented in Fig. I.3.1 and Thiessen coefficients by sub-basin are given in Table I.3.6.

Daily rainfalls for low flow and flood runoff analyses have been calculated by sub-basin from those observed at stations applying the Thiessen coefficients. The results are summarized in annual rainfall and monthly average rainfall, as presented in CHAPTER 5, LOW FLOW ANALYSIS.

Area Reduction Factor

Rainfall depth-duration curves developed from point rainfall at each station have been converted to average basin rainfall for the flood runoff analysis for sub-basins with the area of less than 500 km^2 by the Area Reduction Factor. (Refer to Fig. I.3.2.)

Relation between Rainfall and Altitude

It is generally said that the higher the altitude the more rain falls. Annual rainfalls in the basin have been plotted against altitudes as shown in Fig. I.3.3, but no significant relation is shown between the two.

Meteorologically, the valley of the Upper Indragiri River in West Sumatra Province has lower rainfall due to the effect of surrounding mountains, although altitudes are comparatively higher in this area. Accordingly, no effect of altitude to rainfall is considered.

3.6 Calculation of Probable Rainfall

Taking collected and supplemented rainfall data into consideration, probable values for both point rainfall and average basin rainfall have been calculated by the following procedure:

- Calculate the probable 1-day rainfall of each rainfall station;
- Pick up the annual maximum mean rainfall of basin with 1-, 2-, 3-, 5-, 7- and 10-day duration [refer to Table I.3.8(A)]; and
- Conduct probability analysis for the annual maximum rainfalls of the same duration.

The results of calculation of probable rainfall for the point rainfall are shown in Table I.3.7 and the results for the average basin rainfall are shown in Table I.3.8(B).

3.7 Establishment of Rainfall Depth-Duration Curve of Short Duration

Rainfall depth-duration curves of short duration have been compared with several methods and the Log Normal Method was employed for the flood runoff analysis based on the collected hourly rainfall data for catchment areas smaller than 500 km^2 (refer to Table I.3.9). The study was done and compared with four methods, and the equation of Talbot was selected by reason of least difference with the observed data.

3. 8 Analysis of Rainfall Patterns during Past Notable Floods

Rainfall patterns during the past notable floods have been examined in terms of temporal and spatial distribution to determine the patterns of design rainfall for sub-basins with the area of more than 500 km^2.

3. 9 Determination of Design Rainfall

The design rainfalls employed for the flood runoff analysis have been determined under the following three factors at each reference point.

(1) Total Volume of Rainfall

By using the probable rainfall calculated above, the total volume of rainfall is calculated for each area.

(2) Temporal Distribution of Rainfall

For sub-basins with areas larger than 500 km^2, the temporal distribution was determined by the distribution of past notable floods as mentioned above. For sub-basins with areas smaller than 500 km^2, the model hyetograph of the central concentration type with 12 hours of rainfall duration was adopted (refer to Table I.3.10 and Fig. I.3.4).

(3) Spatial Distribution of Rainfall

For areas larger than 500 km^2, the areal distribution is determined by the distribution of past notable floods as mentioned above. For areas smaller than 500 km^2, areal rainfall is obtained by applying the area reduction factor.

3.10 Calculation of Probable Maximum Precipitation (PMP)

Six damsites are proposed in the Overall Development Plan. The design discharge at each damsite has been calculated to determine the dimensions of spillways. Criteria to select rainfall for the calculation of discharge is different by dam type. The design discharge for designing a spillway of a rockfill dam is the probable maximum flood (PMF) discharge.

The PMF has been calculated from the PMP. There are two methods to calculate PMP; namely, (a) calculation based on dewpoint and wind velocity in and/or around the dam catchment areas, and (b) statistical Hershfield Method. In this Study, the PMP was estimated by the statistical Hershfield Method using a series of annual maximum daily precipitations at major rainfall stations of the basin. The Hershfield Method is recommended by the World Meteorological Organization (WMO) for areas where rainfall records are available but other climatic records are hardly obtained.

PMP is estimated by the following equation and the statistical coefficient (K_m) in the equation which is also estimated from the relationship of K_m and X_n values developed by Hershfield (refer to Fig. I. 3.5). The calculation procedure of the Hershfield Method is shown in Table I.3.11.

$$PMP = X_n + K_m \cdot S_n \quad (3.2.1)$$

where;

PMP	:	probable maximum precipitation
X_n	:	unadjusted average of a series of annual maximum precipitation
K_m	:	unadjusted average of a series of annual maximum precipitation excluding the highest value
S_n	:	unadjusted standard deviation of a series of annual maximum precipitation excluding the highest value

The calculation was made for the Kampar Kiri No. 1 and Upper Sinamar dams which are planned to be rockfill dams, and the results are summarized below.

Probable Maximum Precipitation (PMP)

Unit: mm

Name of Dam	1-day Rainfall	2-day Rainfall	3-day Rainfall
Kampar Kiri No. 1 Dam	224.0	453.8	475.3
Upper Sinamar Dam	294.0	428.9	596.8

3.11 Rainfall Analysis for Rengat Area Interior Drainage Analysis

Rainfall analysis for the interior drainage analysis in Rengat Area has been conducted. Annual maximum 10-day rainfall data for 13 years from 1981-1993 at Japura-Rengat Station (Serial 05109) were picked up (refer to Table I.3.12). Probable values for 2-, 5-, and 10-year return periods have been accordingly obtained by Hazen Plot as shown in Fig. I.3.6 and summarized below.

Unit: mm

Return Period	10-day Rainfall
2- year	205
5-year	230
10-year	245

CHAPTER 4 FLOOD RUNOFF ANALYSIS

4.1 Procedure of Flood Runoff Analysis

Flood runoff analysis has been conducted in accordance with the following procedures, each of which are described in the following section:

- Selection of method of flood runoff analysis;
- Selection of reference points and division of basin;
- Determination of constants for runoff model; and
- Calculation of probable flood hydrograph and probable maximum flood.

4.2 Flood Runoff Analysis

4.2.1 Selection of Method of Flood Runoff Analysis

In order to determine the basic project flood for each river, it is necessary to estimate the flood hydrograph from the design hyetograph using a flood runoff model. In this study, the Storage Function Model is employed. Various experimental values for constants have been developed for this model, and the model is suitable for river basins with a few discharge data for verification.

(1) Storage Function Model for River Basin

The storage function model has been developed to express non-linear characteristics of runoff phenomena introducing the following function between the storage volume (S_l) of a basin or a river channel and the discharge (Q_l) from the same.

$$S_l = KQ_l^p \tag{4.2.1}$$

where;

K and p are constants.

This equation is used as the equation of motion which expresses runoff as proportional to the exponent of storage volume. In this equation, the runoff phenomena is considered to be similar to the runoff from the notch of a container filled up with water.

Runoff calculation is performed in combination with the following equation of continuity for a basin.

$$\frac{dS_l}{dt} = \frac{1}{3.6} f \cdot r_{ave} \cdot A - Q_l \tag{4.2.2}$$

where;

S_l	:	apparent storage volume in the basin ($m^3/s/hr$)
f	:	inflow coefficient
r_{ave}	:	basin's average rainfall (mm/hr)
A	:	area of the basin (km^2)
$Q_l=Q(t+T_l)$	:	direct runoff height with lag time (m^3/s)
T_l	:	lag time (hr)

Runoff calculations for a basin are generally made by dividing the above two basic equations (4.2.1) and (4.2.2) by the area of a basin in order to express them by the storage height s (mm) and runoff height q (mm/hr). Accordingly, the basic equations are expressed as follows:

$$s_l = K \cdot q_l^P \tag{4.2.3}$$

$$\frac{ds_l}{dt} = f \cdot r_{ave} - q_l \tag{4.2.4}$$

The constant f in the above equation is to estimate the effective rainfall. In the storage function model, the coefficient f is not related to rainfall but to the drainage area A; namely, it is assumed that in the early stages of rainfall, f is f_1 (termed the primary runoff rate) and that runoff occurs only from the area f_1A (called the runoff area). When accumulated rainfall exceeds Rsa (saturation rainfall), then f=1 (this is termed the saturated runoff rate), and the runoff occurs also from the remaining part $(1-f_1)A$ (infiltration area) due to the rainfall after Rsa.

In this model, runoff from both the runoff area and the infiltration area is calculated separately by equations (4.2.3) and (4.2.4) until the end of the flood. The total runoff from the whole basin is given by the sum of runoff from both areas plus the base flow, as shown in the following equation.

$$Q = \frac{1}{3.6} f_1 A \cdot q_l + \frac{1}{3.6}(1 - f_1) A \cdot q_{sat} + Q_i \tag{4.2.5}$$

where;

Q	:	discharge (m^3/s)
f_1	:	primary runoff rate
q_l	:	runoff height caused by total rainfall (mm/hr)
q_{sat}	:	runoff height caused by total rainfall after saturation (mm/hr)
Qi	:	base flow (m^3/s)
A	:	total drainage basin (km^2)

(2) Storage Function Model for River Channel

The storage function of a river channel is expressed as follows:

$$S = K \cdot Q_l^P \quad (4.2.6)$$

$$\frac{dS_l}{dt} = Q_i - Q_l \quad (4.2.7)$$

where;

S	:	storage volume of river channel (m^3/s/hr)
Q_i	:	inflow discharge of channel (m^3/s)
$Q_l(t)=Q_o(t+T_l)$	:	outflow discharge of channel (m^3/s)
K, p	:	constants
T_l	:	lag time (hr)

4.2.2 Selection of Reference Points and Division of Basin

Reference Point

Reference points are generally selected on points of major flood control facilities such as dams, and confluences of major tributaries which become base points for hydraulic and hydrological analysis. For this Study, the following reference points have been selected.

Kampar River Basin		
Reference Point		C.A.
Name	No.	(km^2)
Kapoernan	6	699
Kotapanjang	12	3,337
Kiri No.1	20	1,187
Kiri No.2	28	552
Lipat Kain	37	3,284
Kiri+Kanan	54	12,284
Kerinci	58+61	16,768
Telukmeraniti	56+70	21,497

Indragiri River Basin		
Reference Point		C.A.
Name	No.	(km^2)
Sinamar 1	3	828
Agam	6	450
Sinamar 2	3+6	1,278
Low.Sinamar	12	1,779
Sukarami	24	534
Sukam	44	485
Kuantan	53	7,453
Peranap	61+64	10,885
Japura	67	12,320
Kualacenake	72+75	15,100

Division of Basin

In this study, the Kampar river basin is divided into 18 sub-basins related to 19 river channels and the Indragiri river basin is divided into 19 sub-basins related to 21 river channels, in consideration of topographical features and location of structures, as shown in Table I.4.1 and Fig. I. 4.1. Areas of sub-basins range between 400 and 3,000 km^2. Schematic illustrations of river basins and river channesl for the Kampar and Indragiri rivers are presented in Fig. I.4.2.

4.2.3 Determination of Constants for Runoff Model

Storage function constants for river basins and river channels are determined by the following formulae.

(1) Storage Function Constants for River Basin

The basin's storage function is as presented in Subsection 4.2.1 and constants K and p and lag time T_l for river basins are determined by empirical formula as shown below.

$$K = \frac{119}{I^{0.3}} \tag{4.2.8}$$

where;

I : basin slope gradient
p : fixed as 0.6

$$T_l = 0.047L - 0.56 \tag{4.2.9}$$

where;

T_l : lag time (hr)
L : channel length (km)

Primary runoff rate (f_1) and saturation rainfall (Rsa) are determined as $f_1 = 1$ and $Rsa = 0$, because the basin is usually saturated with the preceding rainfall when larger rainfalls occur.

(2) Storage Function Constants for River Channel

The constants K and p and lag time T_l for river channel are determined by empirical formula as shown below.

$$K = LB^{0.4}(n/I^{0.5})^{0.6}/3.6 \tag{4.2.10}$$

where,

L : channel length (km)
B : average channel width (m)
n : Manning's roughness coefficient
I : channel slope gradient
T_l : lag time (hr)

Constant p takes the value given below from Manning's Formula for steady flow.

Channel Width (B)	Constant p
300 m < B	0.60
50 m < B ≤ 300 m	0.61
50 m > B	0.62

Lag time T_l is expressed by the following equation:

$$T_l = 7.36 \times 10^{-4} \times L\, I^{-0.5} \tag{4.2.11}$$

where,

L : channel length (km)
I : channel slope gradient

(3) Calculation and Verification of Hydrograph

The constants were decided by using the three major floods in 1986, 1989 and 1991 after some trial calculations, as shown in Table I.4.2. The calculated hydrographs were verified by the observed hydrographs and it was confirmed that the calculated and observed hydrographs at each gaging station closely correlate, as shown in Fig. I. 4.3.

4.2.4 Calculation of Probable Flood Hydrograph and Probable Maximum Flood (PMF)

Probable flood hydrographs were obtained from probable rainfalls studied in CHAPTER 3, RAINFALL ANALYSIS, using the runoff models and constants given in the preceding subsection.

Design Hyetograph

From the three major floods simulated in the preceding Subsection, one hyetograph which recorded the maximum flood was selected as the actual rainfall distribution for the preparation of a model hyetograph; namely, the rainfall during the 1991 flood in the Kampar River and the rainfall during the 1986 flood in the Indragiri River. The model hyetograph was employed for catchment areas smaller than 500 km^2 considering that there are no sufficient hourly rainfall records during storms (refer to Table I.3.10).

Flood Concentration Time

The peak discharge volume at each reference point is governed by the total rainfall which has fallen in flood concentration time which is defined as the period of time required until rainfall which has fallen in the farthest area of basin reaches the reference point. In considering the scale of the catchment area, the flood concentration time has been decided as follows:

Catchment Area	Flood Concentration Time
A < 500 km^2	1-day
500 km^2 < A < 2,000 km^2	3-day
2,000 km^2 < A < 20,000 km^2	5-day
A > 20,000 km^2	7-day

Rainfall Extension Ratio

The actual rainfall amount was extended to probable rainfall. The extension rate at each reference point was calculated by dividing probable rainfall by the total of actually observed rainfall of each basin at flood concentration time, as shown in Table I.4.3.

Probable Flood Hydrograph for Flood Control

Peak discharges of the probable flood runoff hydrograph of 2-, 5-, 10-, 25-, and 50-year return period for each reference point are shown in Table I.4.4. The peak discharges of 50-year return period are plotted on the Maximum Record Floods in Indonesia published by DPU in 1984, as shown in Fig. I.4.4. The calculated values of the peak discharge are distributed in an appropriate range compared with the maximum records of other Indonesian rivers. The peak discharges at reference points of 50-year return period are summarized in the table below.

Kampar River Basin			Peak	Specific	Indragiri River Basin			Peak	Specific
Reference Point		C.A.	Discharge *	Discharge	Reference Point		C.A.	Discharge *	Discharge
Name	No.	(km^2)	(m^3/s)	($m^3/s/km^2$)	Name	No.	(km^2)	(m^3/s)	($m^3/s/km^2$)
Kapoeman	6	699	1,365	2.0	Sinamar	3+6	1,278	2,094	1.6
Kotapanjang	12	3,337	3,997	1.2	Low.Sinamar	12	1,779	2,421	1.4
Kiri No. 1	20	1,187	1,630	1.4	Kuantan	53	7,453	6,545	0.9
Kiri No. 2	28	552	1,399	2.5	Peranap	61+64	10,885	6,777	0.6
Lipat Kain	37	3,284	3,101	0.9	Japura	67	12,320	6,998	0.6
Kiri + Kanan	54	12,284	6,790	0.6	Kualacenake	72+75	15,100	7,659	0.5
Kerinci	58+61	16,768	7,034	0.4	Sinamar	3	828	1,840	2.2
Telukmeraniti	56+70	21,497	7,951	0.4	Agam	6	450	1,107	2.5
					Sukarami	24	534	1,482	2.8
					Sukam	44	485	1,414	2.9

* Peak Discharge = 50-year return period

Probable Flood Hydrograph for Dam Designing

In the Overall Development Plan, six sites have been selected as possible for dams; namely, the Kapoernan, the Kampar Kiri No. 1, the Kampar Kiri No. 2, the Sukam, the Upper Sinamar and the Lower Kuantan. Four of the six dams are planned as concrete gravity dams and the others, Kampar Kiri No. 1 and Upper Sinamar, are rockfill type dams.

The design criteria for a spillway is different by dam type and the design flood discharge at a damsite is necessary to be determined for further study. The design discharges for spillways of concrete gravity dams have been calculated with a flood of 1,000-year return period, and for rockfill dams, with the probable maximum flood

(PMF), as summarized below. PMF was calculated with the same models and constants with 1,000-year return period using the probable maximum precipitation (PMP).

The probable peak discharge and model hydrograph of each return period at damsites are shown in Table I.4.4 and Fig. I. 4.5. The probable peak discharge of 1,000-year return period for a concrete dam and probable maximum flow (PMF) for a rockfill dam are summarized as follows:

Dam Features			Peak Discharge (m^3/s)		Specific Discharge ($m^3/s/km^2$)
Location of Dam Site	Type of Dam	C.A. (km^2)	1,000-year Return Period	PMF	
Kapoeman	Concrete Gravity	699	2,181		3.1
Kiri No. 1	Rockfill	1,187		7,274	6.1
Kiri No. 2	Concrete Gravity	552	1,992		3.6
Sukam	Concrete Gravity	360	1,755		4.9
Upper Sinamar	Rockfill	1,580		8,383	5.3
Kuantan	Concrete Gravity	7,453	10,047		1.3

On the figure of Maximum Record Floods in Indonesia, the peak discharge and PMF of each damsite are distributed in an appropriate range compared with the maximum records of other Indonesian rivers, as shown in Fig. I. 4.6.

4.3 Design Discharge for Diversion Tunnels at Damsites

Design discharges of temporary river diversion tunnels are determined in consideration of dam type, flood characteristics, frequency of floods during construction and damage in case of overflow. As design discharges of diversion tunnels, 20-year return period discharge is employed for the rockfill type dams, and 2-year return period discharge for the concrete gravity type dams. The probable discharges were calculated with actually observed discharge data.

There are three automatic water level gaging stations (AWL) with observation records of nine years or more in the study area; namely, Danau Bingkuang in the lower stretch of the Kampar Kanan River, Lipat Kain in the Kampar Kiri River and Pulau Berhala in the Indragiri River. Annual maximum discharges are shown in Table I.4.5.

Design discharge is calculated, firstly, by calculating the probable discharge of the three AWL for 2- and 20-year return periods (refer to Fig. I.4.7), secondly, by determining the Creager number which is the coefficient of the specific discharge formula shown below for each return period, and finally, by selecting the largest number and calculating the design discharge at each dam site by Creager number and catchment area.

$q = C \times A^{(A^{-0.05}-1)}$ or $Q = C \times A^{(A^{-0.05})}$

where;

q : specific discharge ($m^3/s/km^2$)
Q : discharge (m^3/s)
C : Creager number
A : catchment area (km^2)

Discharges of 2-year return period have been applied for the design discharges of diversion channels of concrete gravity dams and discharges of 20-year return period for rockfill dams. The results of determination of design discharges at each damsite are summarized below.

Unit: m^3/s

Return Period (year)	Creager Number	Damsite (Catchment Area)					
		Kapoernan (699 km^2)	Kiri No. 1 (1,187 km^2)	Kiri No. 2 (552 km^2)	Sukam (360 km^2)	Upper Sinamar (1,580 km^2)	Kuantan (7,453 km^2)
2	C=5	560	-	500	400	-	1,510
20	C=8	-	1,150	-	-	1,310	-

CHAPTER 5 LOW FLOW ANALYSIS

The analysis on low flow, i.e., daily discharge, is explained in this chapter. The results of analysis have been used as the basic data for water resources development planning which need more than ten years of discharge data. Since the observed discharge data are not sufficient both in number and recording period, daily discharges have been simulated from daily rainfall.

Rainfall analysis is presented first. Models that convert daily rainfall to daily discharge are accordingly established after calibration. Simulated discharges are presented in Section 5.4.

5.1 Basic Conditions for Analysis

Calculation and Calibration Points

The points where daily discharge data are needed for water resources development planning are the proposed damsites, major tributaries confluence points, etc. Daily discharges have been calculated for each of the sub-basin shown in Fig. I.4.1 and accordingly summarized in SECTOR VIII, WATER RESOURCES DEVELOPMENT PLAN.

Actually observed data can be used directly. As Table I.1.2 shows, available discharge data in the basin are not sufficient both in number and recording period; hence, daily discharges have been estimated appropriately. Simulation of daily discharges is commonly conducted by a model that converts daily rainfall to daily discharge. The Tank Model has been applied for this study. Calibration points for the establishment of low flow simulation models are selected from those shown in Table I.1.2.

Simulation Period

Data for the low flow analysis are the daily rainfalls. Daily rainfall for the 12-year period from 1981 to 1992 can be compiled completely for all stations in the basin by mutual supplementation (refer to available data and period in Table I.1.1). Since data are available only at two to three stations for the period before 1981, the daily discharge data for the 12-year period from 1981 to 1992 have been calculated.

The appropriateness of the simulation period of 12 years from 1981 to 1992 is presented below. A probability analysis has been conducted for dry season rainfalls of the 28-year period from 1961 to 1992 at Sijunjung and 24-year period from 1970 to 1993 at Pasar Kampar. The results show that 5 to 10-year return period droughts exist in the period from 1981 to 1992 (refer to Table I.5.1). Therefore, the planning based on the 12-year data from 1981 to 1992 is deemed appropriate.

5.2 Average Basin Rainfall

Average basin rainfall has been calculated by sub-basin from those observed at stations, applying the Tiesen Coefficients as discussed in CHAPTER 3. The average basin rainfalls are summarized by sub-basin in annual rainfall and monthly average rainfall for 12 years from 1981 to 1992, as shown in Tables I.5.2 and I.5.3, respectively.

5.3 Runoff Model

Tank Model

The Tank Model is one of the notable models for low flow simulation. In this model, the runoff structure of the catchment basin is expressed by a series of several (usually four) tanks with side outlets. Rainfall input on the top tank flows out into the river through side outlets or flows down into the lower tank, i.e., to the lower aquifer. The same procedure is repeated in the next tank. The discharge is then calculated as the total of the flow from the side holes. Constants of the tank structure are determined through trial and error comparing the calculated and actually observed discharges. These cannot be determined mathematically.

The simulation condition for the present study is determined as follows:

(1) Number of Tanks : 4

(2) Calculation Time Interval : 1 day

(3) Evaporation

Pan-evaporation data at Sukarami have been adopted for the upper basins and those at Rengat, for the lower basins. The average daily and annual total values are as given below.

Unit: mm

Station	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Year
Surakami	3.19	3.93	3.55	4.40	4.32	4.77	4.32	4.48	4.07	3.87	3.50	5.58	1,519
Rengat	3.61	4.71	4.35	4.07	4.32	4.53	4.19	4.42	4.17	4.03	4.20	3.39	1,517

(4) Evaporation Coefficient

The evaporation coefficient of 0.8 has been applied for days with more than 0.5 mm rainfall. No reduction factor for the first and second tanks were considered, and reduction factor of 0.60 and 0.30, respectively, for the third and fourth tanks were considered.

(5) Annual Evapotranspiration

Annual evapotranspiration is usually less than the pan-evaporation. Usually, it is around 1,000 mm. In the model, annual evapotranspiration, i.e., the difference between rainfall and discharge has been adjusted to about 1,200 mm.

<u>Model and Calibration Results</u>

The constants of the tanks for the Kampar and Indragiri rivers have been determined as presented below:

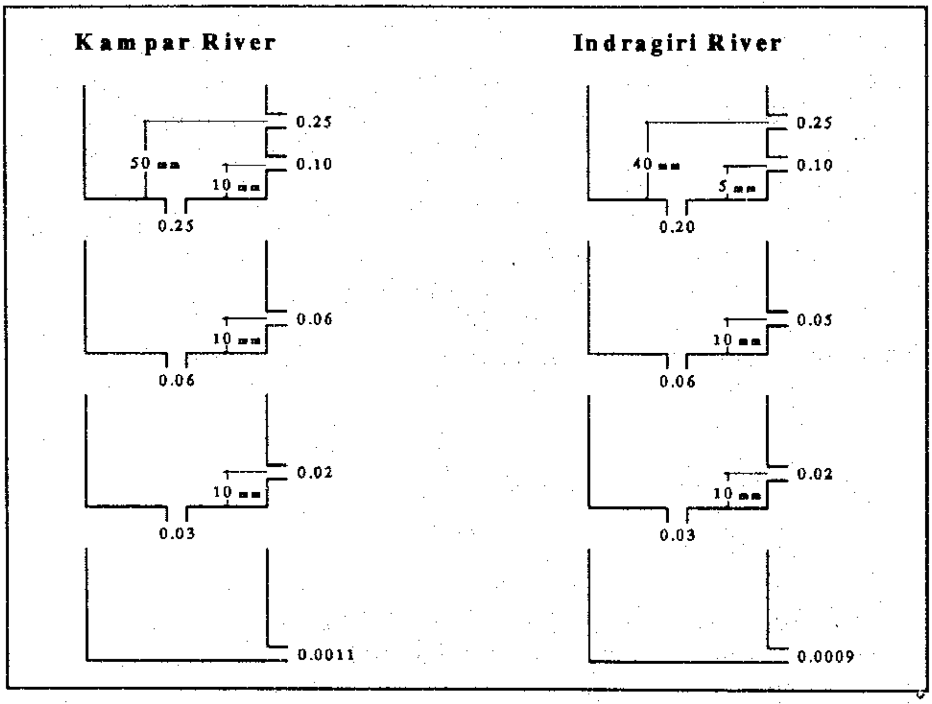


Fig. I.5.1 shows the observed and simulated hydrographs. The following considerations were taken for the determination of constants.

- Constants are determined to realize the tendency of upgrading and recession curves of hydrographs.
- Total loss, namely evapotranspiration, is adjusted to around 1,200 mm.

5.4 Simulated Discharge

The simulated discharges are summarized as annual discharges and monthly average discharges by sub-basin in Tables I.5.4 and I.5.5, respectively. Table I. 5.6 gives a summary of rainfall, average discharge, catchment area, specific discharge, runoff height and evapotranspiration by sub-basin. The average values for the 12-year period from 1981 to 1992 on the Kampar and Indragiri river basins are as tabulated below.

Particulars	Unit	Kampar River Basin	Indragiri River Basin
Rainfall	mm	2,513	2,338
Average Discharge at River Mouth	m^3/s	1,010	591
Catchment Area	km^2	24,548	16,268
Specific Discharge	$m^3/s/100km^2$	4.11	3.63
Runoff Height	mm	1,298	1,145
Loss (Evapotranspiration)	mm	1,215	1,193

CHAPTER 6 INUNDATION ANALYSIS

6.1 Inundation Condition in Study Area

Inundations regularly occur in five areas in the study area: the two areas along the middle and lower reaches of the Kampar and Indragiri rivers and the three areas of Payakumbuh, Solok and Sijunjun in the upper reaches of the Indragiri River. Large floods recently occurred in these areas in 1978, 1986 and 1991.

Inundation analysis for the five areas was carried out to estimate the flood damage area in several cases of flooding conditions, 2-year to 50-year return periods. Results of the inundation analysis are used for the flood damage calculation and economic evaluation of the flood control plan.

For the inundation analysis, the computer simulation model was established based on the field survey. As for the field survey, flood mark survey of the five areas was carried out by a local consultant to confirm the inundation area as well as the water depth of three big floods.

From the results of the field survey, the features of flooding in the inundation area have been estimated as follows:

- Overbank flow spreads from poor flow capacity points.
- The flood area is the alluvium plain with a slight slope towards the downstream, and overbanking water flows along river courses.

Considering the above features, flooding in the area shows the storage type and the flow/diffusion type. Therefore, the Two-Dimensional Unsteady Flow Model was employed.

6.2 Two-Dimensional Unsteady Flow Model

The basic equations applied to the model are derived from the following equations:

(1) Euler's Equation of Motion

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} + w\frac{\partial u}{\partial z} = X - \frac{1}{\rho}\frac{\partial P}{\partial X}$$

$$\frac{\partial v}{\partial t} + u\frac{\partial v}{\partial x} + v\frac{\partial v}{\partial y} + w\frac{\partial v}{\partial z} = Y - \frac{1}{\rho}\frac{\partial P}{\partial Y}$$

$$\frac{\partial w}{\partial t} + u\frac{\partial w}{\partial x} + v\frac{\partial w}{\partial y} + w\frac{\partial w}{\partial z} = Z - \frac{1}{\rho}\frac{\partial P}{\partial Z}$$

where;

u, v, w : velocity of x, y and z directions
X, Y, Z : gravity of x, y and z directions

ρ	:	water density (=1.0)
P	:	pressure

(2) Equation of Continuity

$$\frac{\partial u}{\partial x}+\frac{\partial v}{\partial y}+\frac{\partial w}{\partial z}=0$$

For actual application to the two-dimensional model, the above expressions are expressed as follows:

(1) Equation of Motion

$$\frac{1}{gA_x}\frac{\partial Q_x}{\partial t}-\frac{Q_x B_x}{gA_x^2}\frac{\partial H}{\partial t}+\frac{\partial H}{\partial x}+\frac{|Q_x|Q_x}{F_x^2}=0$$

$$\frac{1}{gA_y}\frac{\partial Q_y}{\partial t}-\frac{Q_y B_y}{gA_y^2}\frac{\partial H}{\partial t}+\frac{\partial H}{\partial y}+\frac{|Q_y|Q_y}{F_y^2}=0$$

$$F_x=\frac{1}{n}R_x^{2/3}A_x$$

$$F_y=\frac{1}{n}R_y^{2/3}A_y$$

(2) Equation of Continuity

$$\frac{\partial (Bh)}{\partial t}+\frac{\partial Q_x}{\partial x}+\frac{\partial Q_y}{\partial y}=0$$

where;

Q_x, Q_y	:	discharge of x and y directions
A_x, A_y	:	current area of x and y directions
B_x, B_y	:	width of x and y directions
R_x, R_y	:	hydraulic depth of x and y directions
g	:	gravity acceleration (9.8 m/s^2)
n	:	Manning's roughness coefficient
H	:	water level
h	:	water depth

The above equations are finally transformed into finite difference for numerical computation, as follows:

(1) Finite Difference Form of Equation of Motion

$$\frac{1}{gA_{I,J}^{n-1/2}}\frac{Q_{I,J}^{n}-Q_{I,J}^{n-1}}{\Delta t}-\frac{\left(\dfrac{Q_{I,J}^{n}-Q_{I,J}^{n-1}}{2}\right)\cdot\Delta y}{g\cdot\left(A_{I,J}^{n-1/2}\right)^{2}}\frac{H_{I,J}^{n-1/2}-H_{I,J}^{n-3/2}}{\Delta t}+$$

$$\frac{H_{I+1/2,J}^{n-1/2}-H_{I-1/2,J}^{n-1/2}}{\Delta x}+\frac{\left|Q_{I,J}^{n-1}\right|Q_{I,J}^{n}}{\left\{\dfrac{1}{n}\cdot\left(\dfrac{A_{I,J}^{n-1/2}}{\Delta y}\right)^{2/3}\cdot A_{I,J}^{n-1/2}\right\}^{2}}=0$$

(2) Finite Difference Form of Equation of Continuity

$$\frac{(Bh)_{I,J}^{n}-(Bh)_{I,J}^{n-1}}{\Delta t}+\frac{Q_{I+1/2,J}^{n-1/2}-Q_{I-1/2,J}^{n-1/2}}{\Delta x}+\frac{Q_{I,J+1/2}^{n-1/2}-Q_{I,J-1/2}^{n-1/2}}{\Delta y}=0$$

where;

suffix I, J	:	mesh number of x and y directions
suffix n	:	computation time step number

6.3 Inundation Analysis for Overall Development Plan

6.3.1 Establishment of Inundation Model

Flood inundation models have been prepared for five inundation areas under the following conditions:

- Inundation areas are to be divided into mesh blocks of 1,860 m by 1,860 m which is equivalent to one minute of longitude and latitude in the middle reaches and 465 m by 465 m which is equivalent to a quarter of one minute in the upper reaches of the Indragiri River, as shown in Fig. I. 6.1.
- The average ground height of each mesh is to be obtained using the topographic map with a scale of 1/50,000.

As the initial condition for computation, it is necessary to give overflow discharge to the inundation area and the overflow section. The following initial conditions were taken into account:

- The overflow sections selected are of poor flow capacity estimated by the non-uniform calculation method as marked with arrows on the mesh map shown in Fig. I.6.1.

- It is assumed that in probable flood hydrographs, the surplus discharge over the flow capacity overflows at the overflow section. The overflow discharge *(dQ)* at the overflow section is given by the surplus discharge *(Q)* minus the flow capacity *(q)* at the overflow section in the hydrographs, as follows:

$$dQ = Q - q$$

6.3.2 Adaptability of Model and Computation Case

Adaptability of the simulation model was verified by comparing the actually inundated area with the simulated area using the middle reaches of the Indragiri River in the 1986 flood as the verification model. The simulation model is adequate to the inundation records, as shown in Fig. I. 6.2.

The maximum inundation depth and the inundation area have been examined under five cases of probable flood discharge of 2- to 50-year return periods.

6.3.3 Calculation Results

The probable inundation area and water depth which correspond to the probable flood of the five areas are shown in Fig. I.6.1, and the inundation area occasioned at each return period is shown in the following table.

Unit: km^2

Return Period (Year)	Inundation Area				
	Kampar River	Indragiri River			
	Middle Reaches	Middle Reaches	Paya-kumbuh	Solok	Sijunjung
50	2,151	1,460	284	106	60
25	2,136	1,406	273	102	59
10	2,025	1,356	254	95	56
5	1,902	1,314	234	83	54
2	1,752	1,237	131	60	47

6.4 Inundation Analysis for Priority Projects

6.4.1 Establishment of Inundation Model

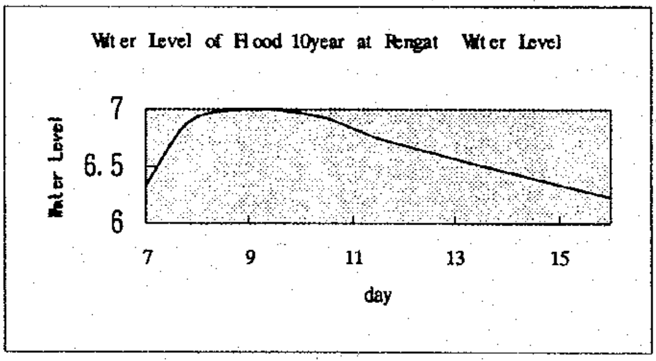
Detailed flood inundation models for the two areas selected as priority for flood control projects, namely Bangkinang and Rengat, have been established under the following conditions (refer to Fig. I.6.3).

(1) Bangkinang Area along Kampar River

- The whole inundation area is divided into 930 m by 930 m mesh blocks which are equivalent to half minute of longitude and latitude.
- The average ground height of each mesh is obtained using the topographic map with a scale of 1/50,000.
- The overflow discharge is given by the discharge volume as the case of the Overall Plan.
- Two cases, 2- and 5-year return period, are calculated considering the design scale of the flood control project.

(2) Rengat Area along Indragiri River

- The whole inundation area is divided into 250 m by 250 m mesh blocks.
- The average ground height of each mesh is obtained using the topographic map with a scale of 1/5,000.
- The overflow discharge is given by the water level resulting from the inundation analysis, the case of the middle reaches of Indragiri River in the Overall Plan. In the case of a 10-year return period flood, the water level at Rengat is as shown in the following figure.



- Three cases, 2-, 5- and 10-year return period, were calculated considering the design scale of the flood control project.

6.4.2 Calculation Results

The probable inundation area and water depth corresponding to the probable flood of the two areas are shown in Fig. I.6.3, and the inundation area occasioned at each return period is shown in the following table.

Unit: km^2

Return Period (Year)	Inundation Area	
	Kampar River	Indragiri River
	Bangkinang	Rengat
10	-	27
5	99	25
2	92	23

CHAPTER 7 TIDAL ANALYSIS

7.1 Analysis on Tidal Level

Data on tidal level were obtained from the office of DINAS HIDRO-OCEANOGRAFI TNI-AL. Tables of tidal level from 1993 to 1994 at two stations, Blandong in the Kampar River and Kuala Lajau in the Indragiri River, were collected (refer to Fig. I.7.1). The collected mean tidal level and mean high water spring are as below.

Item	Blandong/ Kampar River	Kuala Lajau/ Indragiri River
Mean Tidal Level	2.10 m	2.50 m
Mean High Water Spring	4.05 m	3.90 m

7.2 Saltwater Intrusion Analysis

Saltwater intrudes into river freshwater flowing to the sea. This phenemenon results from the density difference between pure river water and ocean water due to salinity variations. The density currents in rivers are categorized into three types in terms of degree of mixing, as follows:

(1) Mixed Type

Both river and sea water mix so well at the estuary that a clear stratification between these bodies of water does not occur.

(2) Partially Mixed Type

Mixing between river and sea water partially takes place. Consequently, both horizontal and vertical salinity variations exist in the currents.

(3) Stratified Type

This type has a horizontal interface between river and sea water. The saltwater enters under a body of fresh river water, and it is generally called a salt wedge.

In the Kampar and Indragiri rivers, the longest length of salt wedge occurs under the minimum discharge flow and the high water spring. Under this quiet condition, the stratified type is formed. The stratified type is as explained below.

7.2.1 Surface of Saltwater Wedge

Assuming a saltwater wedge in a prismatic, horizontal, rectangular channel discharging into an infinite and non-tidal sea, Schjib and Schonfeld (1953) solved the equations of motion and continuity for the upper and lower layers and expressed the surface of this wedge in the following formula:

$$\eta\left(-\frac{1}{5}Fd_o^{-2}\eta^4+\frac{1}{4}Fd_o^{-2}\eta^3+\frac{1}{2}\eta-1\right)+3Fd_o^{2/3}\left(\frac{1}{4}-\frac{1}{10}Fd_o^{2/3}\right)=\frac{\overline{f_i}}{2}\frac{x}{H} \qquad (7.2.1)$$

with,

$$Fd_o=U_o/\sqrt{\varepsilon\cdot g\cdot H} \qquad (7.2.2)$$

$$\eta=h_1/H \qquad (7.2.3)$$

$$\varepsilon=(\rho_2-\rho_1)/\rho_2 \qquad (7.2.4)$$

where,

Fd_o	:	Froude number in river at upstream end of wedge
U_o	:	velocity in river at upstream end of wedge
g	:	acceleration of gravity
H	:	water depth in river at upstream end of wedge
h_1	:	water depth at x point of river
x	:	coordinate along river, positive upstream from river mouth
ρ_1, ρ_2	:	mass density of river water and sea water
$\overline{f_i}$	:	average coefficient of resistance along interface between two water masses and can be calculated by the following empirical equation (Kaneko, 1966):

$$f_i=0.2\psi^{-1/2} \qquad (7.2.5)$$

and ψ *is* the Iwasaki's number (Iwasaki, 1962):

$$\psi=\mathrm{Re}\cdot F_{dm}^2 \qquad (7.2.6)$$

where;

Re and F_{dm} are the Reynolds and Froude numbers, considering the influence of density differences between the two bodies of water.

7.2.2 Length of Saltwater Wedge

Schijband and Schonfeld (1953) also derived an expression for the length of a saltwater wedge based on the same assumption as the length of a saltwater wedge described above; that is, river flow discharging into the sea has a critical depth at the mouth (x = 0). Hence, the equation for the length of intrusion is shown in the following formula:

$$L = \frac{H}{2f_i}\left(\frac{1}{5}F_{do}^{-2} - 2 + 3F_{do}^{2/3} - \frac{6}{5}F_{do}^{4/3}\right) \qquad (7.2.7)$$

7.2.3 Relationship between Discharge and Length of Saltwater Wedge

Based on the preceding equations, the length of saltwater wedge in the Kampar and Indragiri rivers can be calculated with the minimum river flow discharge as the stratified type. The present case of monthly minimum flow ($Q = 275\ m^3/s$) at the Kampar river mouth is calculated as follows:

$$U_o = \frac{Q}{bH} = \frac{275}{1570 \times 7.45} = 0.0234(m/s)$$

where;

b : average river width (m)

H : water depth, assumed to be 7.45 m from high water spring level, in river upstream from wedge [riverbed elevation EL -5.5m + (mean high water spring minus mean tidal level calculated EL +1.95m)]

Taking $\varepsilon = 0.025$ for Eq. (7.2.4), $g = 9.8\ m/s^2$, and from Eq. (7.2.2),

$$Fd_o = U_o/\sqrt{\varepsilon \cdot g \cdot H} = 0.0234/\sqrt{0.025 \times 9.8 \times 7.45} = 0.0174$$

Thus,

$$\eta_c = h_{1c}/H = Fd_o^{2/3} = (0.0174)^{2/3} = 0.067$$

Therefore, the critical depth at the mouth h_{1c} is:

$$h_{1c} = \eta_c \cdot H = 0.0174 \times 7.45 = 0.50m$$

The average coefficient of resistance of f_i is calculated in the average water depth h_{1m} between H and h_{1c}:

$$h_{1m} = (H + h_{1c})/2 = 3.98m$$

From Eq. (7.2.6) for kinematic viscosity, $\nu = 0.01 \times 10^{-4}$ m^2/s:

$$\psi = \mathrm{Re} \cdot F_{dm}^2 = \left(\frac{U_o \cdot H}{\nu}\right)\left(\frac{Q / b \cdot h_{1m}}{\sqrt{\varepsilon \cdot g \cdot h_{1m}}}\right)^2$$

$$= \left(\frac{0.0235 \times 7.45}{0.01 \times 10^{-4}}\right)\left(\frac{275 / 1570 \times 3.98}{\sqrt{0.025 \times 9.8 \times 3.98}}\right)^2$$

$$= 349.2$$

Hence, from Eq. (7.2.5):

$$\overline{f_i} = f_{im} = 0.2 \times (349.2)^{-1/2} = 0.0107$$

Finally, the length of the saltwater wedge is determined by Eq. (7.2.7):

$$L = \frac{7.45}{2 \times 0.0107}\left(\frac{1}{5} \times 0.0174^{-2} - 2 + 3 \times 0.0174^{2/3} - \frac{6}{5} \times 0.0174^{4/3}\right)$$

$$= 229{,}218m \cong 229km$$

7.2.4 Summary of Analysis Results

Other cases of water demand in 2019; namely, the case without project and the case with project which provide maintenance flow, as well as the cases for Indragiri River, are calculated in the same way. The results of calculation are shown in Table I.7.1 and summarized as follows:

River	Project	$Q(m^3/s)$	B (m)	H (m)	L (km)
Kampar	At present	275	1,570	7.45	229
	Future without project	300	1,630	7.45	222
	Future with project	352	1,730	7.45	210
Indragiri	At present	170	745	6.90	160
	Future without project	108	660	6.90	195
	Future with project	187	785	6.90	156

The relationship between discharge and length of saltwater wedge is shown in Fig. I.7.2. The saltwater wedge actually intrudes until the vicinity of Kerinci in the Kampar River and around Rengat in the Indragiri River; however, in accordance with the increase of minimum flow discharge by maintenance flow with the project, the length of saltwater wedge intrusion will be kept at less than the present condition. The length of saltwater wedge will be shorter, until 210 km in the Kampar River and 156 km in the Indragiri River.