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LOWER JENEBERANG RIVER FLOOD CONTROL PROJECT

SUPPORTING REPORT

HYDROLOGY
GEOLOGY
SABO AND SOIL CONSERVATION
RIVER IMPROVEMENT
DAM AND RESERVOIR
DRAINAGE SYSTEM IMPROVEMENT
CONSTRUCTION PLANNING
PROJECT ECONOMY

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I HYDROLOGY

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1. GENERAL

The following hydrological studies were conducted in order to clarify hydrological condition for the formulation of the overall and urgent flood control plan.

These studies were based on the data collected during this study period.

The meteorological analysis was conducted to know the required characteristics such as wind, sunshine, humidity, temperature and evaporation. This study result is useful for formulation of a construction schedule and also for the study of irrigation works.

The rainfall analysis was conducted to estimate probable rainfall and areal mean rainfall and to draw the rainfall intensity curve. This study result is used for the estimation of the design discharge and utilizable discharge.

The run-off analysis was based on the above mentioned result and discharge data. The purpose of the study is to estimate the design discharge, design hydrograph and utilizable discharge.

The tidal stage is considered to be the outlet water stage of the proposed drainage channels. Therefore, a study is required to formulate a new drainage system.

2. PRESENT CONDITIONS OF HYDROLOGY

2.1 Existing Data

Meteorology

At present six meteorological stations exist in the project area; namely, Hasanuddin Airport, Panakkukang, Maros, Bontobill, Bontosunggu and Malino. Table 2-1 shows the measuring items and the starting year of recording. The location of these stations is presented in Fig. 2-1.

Rainfall

1) Daily rainfall

Only 4 stations have been measuring daily rainfall in and around the Jeneberang river basin for a long time; these are Malino, Bontobili, Sungguminasa and Ujung Pandang. In addition to the 4 stations, a lot of stations were newly installed in the left side area of the Jeneberang lower reaches especially in 1975. Table 2-2 and Fig. 2-2 show the existence of the rainfall data over a period of about 70 years at the various stations. And the location of these stations is presented in Fig. 2-3.

2) Hourly rainfall

Only 3 automatic rainfall stations exist around this area as shown in Table 2-3 and Fig. 2-4.

The Hasanuddin station has been recording since 1963. Therefore, the rainfall data of this station is useful to grasp the accurate hourly rainfall characteristics.

Water Stage and Discharge

There exist five and three water stage gauging stations along the Jeneberang river and the Tallo river respectively. Of the above 8 stations, 2 stations along the Jeneberang river have automatic recorders. The low water discharge was also measured at one of these 2 stations 18 times. (Refer to Table 2-4.) However, these stations have started recording recently, except for the Kampili station which started recording in 1966. Therefore, the water stage data are not sufficient to analyze the run-off. Table 2-6 shows the existence of the run-off data, and Fig 2-5 shows the location of the water stage gauging station. Flood marks were observed at the Kampili weir and the Sugguminasa bridge to know the high water stage of the past major floods (refer to Table 2-5).

Tidal Stage

The tidal stage has been measured at the Makassar port since 1976, although the tidal data are sometimes lacking.

Fig. 2-5 shows the location of the tidal stage station.

2.2 Meteorology

Temperature

The monthly mean temperature is about 26°C, fluctuating slightly throughout the year. The maximum and minimum monthly temperatures are 30°C and 22°C.

Fig. 2-6 and Table 2-7 show the monthly mean temperature of each station.

Humidity

The monthly mean relative humidity is about 85% in the rainy season and 70% in the dry season (refer to 2.3). Fig. 2-7 and Table 2-8 show the monthly relative humidity recorded at each station.

Sunshine

The sunshine ratio fluctuates in accordance with the change of seasons. The rate of sunshine per month is between

40% and 50% in the rainy season and 80% in the dry season. Table 2-9 shows the monthly mean sunshine ratio of each station.

Wind

The mean wind velocity in this area is about 4 m/s throughout the year. The maximum wind velocity is 20 m/s in the rainy season and 17 m/s in the dry season. (Refer to Table 2-10). As regard to the direction of the wind, the easterly wind dominates in the rainy season and it turns westerly during the dry season.

Evaporation

The evaporation was measured by a pan evaporimeter at both Maros and Bontosunggu for about 2 years. According to these data, the annual mean evaporation ranges from 1600mm to 1800mm. (Refer to Table 2-11 and Fig. 2-8).

2.3 Rainfall

Annual Rainfall

As mentioned before, only 4 stations have recorded rainfall for a long time. According to the data, though not sufficient for estimation, the annual mean rainfall is considered to be 4,000 mm and 2,800 mm in the mountainous area and the lowlying land respectively (refer to Fig. 2-9). These figures are relatively high in comparison with those in Jawa Island ranging from 1,500 mm to 2,000 mm.

Monthly Rainfall

According to the monthly rainfall distribution throughout the year, the climate in this area is divided into two pronounced seasons; a rainy season, from November to April, and a dry season, from May to October as shown in Fig. 2-9 and Table 2-12. This condition is the same as that in Jawa Island and is opposite to that in the Central Slawesi.

The maximum monthly rainfall of the year is usually recorded in January; that is 900 mm in the mountainous area and 600 mm in the lowlying area. The number of the rainy days is 28 in the former area and 20 in the latter area.

Daily Rainfall

Judging from Fig. 2-10 which shows the daily rainfall during the major floods of the last 5 years, a hyetal region of daily rainfall spreads over the whole basin. The correlations of daily rainfall was studied by using the data at four stations; namely, Malino, Bontobili, Sungguminasa and Ujung Pandang. These correlations are not so clear as shown in Fig. 2-11. The maximum daily rainfall records at these stations are 235 mm in Malino, 222 mm in Bontobili, 259 mm in Sungguminasa and 200 mm in Ujung Pandang.

Hourly Rainfall

In this area there exist only 3 automatic rainfall measuring stations; Malino, Panakkukang and Hasanuddin. The heavy rainfalls of over 50 mm per day were selected from the records of these stations, in order to clarify the hourly rainfall distribution during flood. The hourly rainfall distribution is expressed by the ratio of hourly rainfall to daily rainfall as shown in Fig. 2-12.

2.4 Run-off and Flood

Run-off

The upper reaches of the Jeneberang river show topographically a feather-shaped mountainous land, in which flood run-off congregates. According to the water stage data at Bili-Bili, a high peak discharge of flood is observed periodically, and appears once a day. This hydrograph corresponds to the hourly rainfall distribution in the rainy season.

The lower reaches of Bili-Bili has an alluvial fan, and there exist a natural buffer. A flood discharge can be regulated in this area, so that the hydrograph becomes less-sharp.

Water stage of the Jeneberang river has been gauged at Kampili and Bili-Bili. According to the data well-arranged in 1978, the annual mean run-off at Bili-Bili with a catchment area of 384 km^2 is estimated at 30 m^3/s.

Flood

According to the data collected at Kampili station, the first, second and third biggest floods in the past were recorded at 3,350 m^3/s in 1967, 2,120 m^3/s in 1977 and 1,440 m^3/s in 1974 as tabulated in Table 4-7.

Water stage of the Tallo river has been gauged in the tidal compartment since April of 1979. Flood discharge of the Tallo river cannot be clarified due to insufficient data.

2.5 Tidal Stage

According to the tidal stage data at the Makassar port, the tidal stage is relatively steady. The amplitude at spring tide and low tide is 1.0 m and 0.6 m respectively, and the cycle is twelve hours.

The maximum high tide during the past 3 years was 0.74 m in September of 1977. Fig. 2-13 shows the tidal hydrograph at the Makassar port.

3. RAINFALL ANALYSIS

3.1 Daily Rainfall

Point Rainfall

The probable daily point rainfall was studied by means of Gumbel method and by using the daily rainfall records at 4 stations; Malino, Bontobili, Sungguminasa and Ujung Pandang. The probable daily point rainfall is shown in Table 3-1.

Areal Mean Rainfall

Areal mean rainfall is estimated by the arithmatic mean method in the following combinations of the daily rainfall data at various stations.

Table 3-2 COMBINATION OF THE DAILY RAINFALL DATA

	Milano	Bontobili	Sungguminasa	Ujung Pandang
Bibi-Bili upper basin	0	0		
Kampili, Sungguminasa upper basin	0	0	0	
Inner basin			0	0
Tallo river basin		0	0	0

The areal mean rainfall estimated above is not so accurate for the catchment area of 600 km^2.

The daily rainfall data at 8 stations in the period of 1975 to 1979 were referred to in order to modify the areal mean rainfall.

According to the correlation between the 2-station and 8-station areal mean rainfalls, the difference is about 15% for the daily rainfall under 100 mm. There is not a serious difference for the daily rainfall over 150 mm.

Fig. 3-1 shows the relation mentioned above. The areal mean rainfall is modified by using the dotted line shown in Fig. 3-1 and its result is tabulated in Table 3-3.

Areal Mean Probable Daily Rainfall

Areal mean probable daily rainfall is estimated by means of Gumbel method. The areal mean probable daily rainfall is shown in Fig. 3-2 and Table 3-4.

3.2 Rainfall Intensity Curve

Hourly rainfall data around this area are not sufficient to estimate the probable hourly rainfall directly because of the short period of recording.

Accordingly the rainfall intensity curve cannot be drawn from the probable hourly rainfall data. The rainfall intensity curve around this area is studied in the following procedure.

1) The heavy rainfalls of over 50 mm per day are picked up from the records of each heavy rainfall.

2) The hourly distribution of each heavy rainfall is expressed by the ratio of hourly rainfall to daily rainfall in percentage.

3) Maximum 1-, 2-, 3-, 6- and 12-hour rainfalls of each heavy rainfall are studied from the hourly distribution diagram.

4) The rainfall intensity curve is obtained by means of the least square method and by using the average value of the above-mentioned maximum rainfalls.

According to the result of the above study, the average maximum 1-hour rainfall is around 35% of the daily rainfall at any stations, and 90% of the daily rainfall is observed in a period of 6 hours (refer to Fig. 3-3).

3.3 Design Hyetograph

The design hyetograph at each station, shown in Fig. 3-4, is prepared by the following procedure.

1) The rainfall ratio of arbitrary one hour to daily rainfall, rt, is given by the Eq. (3.1) in a period from t=t to t=t+1; namely,

$$rt = R(t+1) \times (t+1) - R(t) \times (t) \quad \text{..............(3.1)}$$

$$R(t) = \frac{b}{t + a} \; ; \quad \text{rainfall intensity curve}$$

2) The value of rt (t=1) is set at center of the design hyetograph.

3) The value of rt (t; even number) are placed in the rising side and the value of rt (t; odd number) are placed in the recession side of the hyetograph in the order of largeness.

3.4 Probable Hourly Rainfall

Probable hourly rainfall is derived from the rainfall intensity curve and the probable daily rainfall as shown in the following Eq. (3.2).

$$r(t)_{100} = R_{d-100} \times \frac{b}{t + a} \quad \text{.................(3.2)}$$

where,

R_{d-100} : Probable daily rainfall of 100-year return period

$r(t)_{100}$: Probable t-hour rainfall of 100-year return period

3.5 Daily Rainfall Distribution

The study on the duration of daily rainfall is necessary to make the drainage plan of this area. Distribution of daily rainfall in a week is studied by the following procedure, in the same manner as the study of the rainfall intensity curve.

1) The 32-year annual maximum weekly rainfalls are selected from the rainfall records of the inner basin.

2) The distribution of daily rainfall is expressed by the ratio of daily rainfall to weekly rainfall in percentage. This distribution was studied for each of 32 annual maximum weekly rainfall.

3) The maximum 1-, 2-, 3-, 4-, 5- and 6-day rainfalls are picked up from the rainfall data for each annual maximum weekly rainfall.

4) A typical daily rainfall distribution in a week is given by the same procedure mentioned in 3.3.

According to the result shown in Fig. 3-5, one particular day has as much as 37% of the weekly rainfall.

4. RUN-OFF ANALYSIS

4.1 Purpose of Study

Standard project and design flood discharge, design flood hydrograph, and utilizable discharge were studied by using the existing run-off data and the results of rainfall analysis as mentioned in the foregoing. The purposes of study are mentioned below.

1) Standard porject and design flood discharge, and design flood hydrograph - To formulate a river improvement plan and to determine the flood control capacity of a reservoir

2) Utilizable discharge - To facilitate water resources development

4.2 Run-off Features

Time Lag, "T"

A time lag of flood discharge in a basin can be expressed by the propagation time of flood wave. A time lag of flood discharge is observed on the grounds that it takes flood water a long time to flow down along the basin slope and to continue through a river channel. The time of flood wave propagation through a river channel is calculated by the following equations when the propagation velocity is based on the Manning's mean velocity formula.

$$T = Ts + Tr \quad \text{......(4.1)}$$

$$Tr = Lr/Vr = Lr \cdot n/(Rr^{0.67} \cdot Ir^{0.5}) \quad \text{......(4.2)}$$

Where;

T : Time lag of flood
Ts,Tr : Time of flood wave propagation over slope and in river channel respectively
n : Roughness coefficent value in river channel
Rr : Hydraulic radius of flood in river channel
Ir : Mean gradient of river channel
Vr : Discharge velocity in river channel
Lr : River channel length

Table 4-1 shows the figure of each item mentioned above on the Jeneberang river and the Tallo river. One hour is applied to Ts, taking into account the topographical condition of the basin. Fig. 4-1 shows the riverbed elevation along the Jeneberang river course.

According to the relation between the rainfall at Malino and the water stage at Bili-Bili, the mean time lag at Bili-Bili is approximately 4 hours as shown in Fig. 4-2. And this time lag is nearly equal to the time lag calulated by the above equation.

Roughness Coefficient, "n"

Roughness coefficients which are used in the above formula are obtained from the following.

1) Roughness coefficient "n" of 0.045.

This is obtained by applying Manning's formula to discharge observation values at Bili-Bili (refer to Table 4-2).

2) Roughness coefficient "n" of 0.035

This is obtained from Eq. (4.3) by substituting average grain diameter, 0.4 mm of river bed materials around Sungguminasa.

$$n = 0.0417\ d^{1/6} \quad \dots\dots\dots\dots\dots\dots\dots\dots \quad (4.3)$$

3) Roughness coefficients of other rivers

The relation between roughness coefficients and the riverbed gradients applicable in Indonesia is shown in Fig. 4-3.

Based on the above study, specific roughness coefficients required for the study are given below.

Upper reach of Bili-Bili	:	0.045
Bili-Bili to Kampili	:	0.040
Kampili to the estuary	:	0.035

Run-off Coefficient

1) Jeneberang river

Run-off coefficient, "f", can be estimated by the following equation (4.4) converted from the rational formula.

$$f = \frac{3.6 \times Q}{r} \times A \quad \dots\dots\dots\dots\dots\dots\dots\dots (4.4)$$

where,

r : Rainfall intensity (mm/hr)
Q : Run-off discharge (m^3/s)
A : Catchment area (km^2)

In this equation, the flood discharge at Bili-Bili is applied to Q; namely, water stage at Bili-Bili can be converted to the flood discharge by using the rating curve. Likewise, hourly rainfall intensity, "r", is obtained by applying the flood propagation period to the duration curve at Malino. The run-off coefficient is calculted at 0.7. (Refer to Table 4-3.)

2) Inner basin

The run-off coefficient, "f", calculated by the above formula, is applicable for the mountainous area, but not for the inner basin. The run-off coefficient in the former area seems to be different from that in the latter area, because the latter area is already developed and furthermore will be rapidly urbanized in accordance with the regional development plan.

Herein, the run-off coefficient in the inner basin is calculated by the following formulas (4.5 and 4.6).

$$\bar{f} = \frac{Am \times fm + Au \times fu + Ap \times fp + Ai \times fi}{A} \quad ..(4.5)$$

$$A = Am + Au + Ap + Ai \quad ....(4.6)$$

Where;

fm : run-off coefficients in the mountainous area
fu : run-off coefficients in the urban area
fp : run-off coefficients in the paddy field area
fi : run-off coefficients in the inundation area

Am : mountainous area
Au : urban area
Ap : paddy field area
Ai : inundation area
$\bar{f}$: run-off coefficient applied to the estimation of the probable discharge

The values shown in Table 4-4 are usually employed in and also applicable to fm, fu, fp and fi in the above formula. Table 4-5 show the run-off coefficients used for estimation of the probable discharge in the first, second and third stage areas of the regional development.

4.3 Probable Discharge

Water Shed Schematic Diagram

The Jeneberang basin is divided into nine parts in order to estimate the probable discharge at points, namely Bili-Bili, Kampili and Sungguminasa. The division of the water shed is based on the field survey results, topographic maps and aerophotos. The schematic diagram of river systems of the divided basins are shown in Fig. 4-4.

Probable Discharge at Kampili

The probable discharge at Kampili is studied in the following procedure.

1) The annual maximum water stage

Daily water stage at the Kampili weir has been recorded since 1966 (refer to Table 2-6). The Annual maximum water stage is selected from the daily water stage data.

As the daily water stage of Kampili is recorded at the fixed time of a day, 7:00 and 17:00, the maximum water stage of a flood may not be detected.

According to the water stage record at Bili-Bili, there is a difference between the maximum daily water stage gauged at the fixed time and the maximum water stage of the flood (refer to Table 4-6). This table shows that the maximum water stage of the flood is about 10% higher than those of the fixed time.

The daily maximum water stages at Kampili could be modified by adding 10% to them in accordance with this relation. However, the water stage of the flood in 1967 is not modified because the maximum water stage recorded on that day coincides with the height of the existing flood mark.

2) Annual maximum discharge

The water stages can be converted to the dishcarge by using the following formula.

$$Q = C \cdot B \cdot H^{3/2} \quad \text{..............................} (4.7)$$

Where;

C: Discharge coefficient (= 2.0)
B: Width of weir (m)
H: Overflow water depth (m)

Table 4-7 shows the annual maximum discharge converted from the water stage estimated above.

3) Probable discharge

Probable discharge at Kampili is calculated by means of Gumbel method. Frequency curve of the flood discharge at Kampili is shown in Fig. 4-5. Probable discharge is shown in Table 4-10.

Probable Discharge at Bili-Bili

Water stage data at Bili-Bili is not sufficient to estimate accurate discharge. Therefore, the probable discharge at Bili-Bili is estimated by the following.

1) Study on the peak discharge based on the water stage and discharge data

Table 4-8 shows the correlation of water stage between Bili-Bili and Kampili for the major floods since 1974. The following equation is obtained by means of the least square method and by using these water stage data.

$$Hb = 0.42 \times Hk + 1.21 \quad \text{.........................(4.8)}$$

Where;

Hb: Water stage of Bili-Bili
Hk: Water stage of Kampili

The low water discharge is measured several times at Bili-Bili and the rating curve is calculated by means of the least square method and by using these discharge and water stage data. Fig. 4-6 shows the rating curve thus obtained.

According to the flood discharge data shown in Table 4-9, the peak discharge ratio of Bili-Bili to Kampili is approximately 0.75.

2) Study on the peak discharge considering the catchment area and the rainfall depth.

The peak discharge ratio can be obtained from the catchment area and rainfall depth and is expressed in the following equation (4.9).

$$C = \frac{Qb}{Qk} = \frac{Ab \times Rb}{Ak \times Rk} = \frac{384.4 \times 271}{623.7 \times 224} \quad \text{..............(4.9)}$$

Where;

C: Discharge ratio
Qb: Discharge at Bili-Bili
Qk: Discharge at Kamplili
Ab, Rb: Catchment area and areal mean rainfall of Bili-Bili upper basin
Ak, Rk: Chatchment area and areal mean rainfall of Kampili upper basin

When the areal mean rainfall is substituded by the probable rainfall in this equation, the peak discharge ratio will be 0.75, which is same as the result of 1).

Therefore, the probable discharge at Bili-Bili can be estimated by multiplying the discharge at Kampili by 0.75 as shown in Table 4-10.

Probable Discharge at Other Point

Run-off formulas are helpful to estimate the probable discharge where run-off data are insufficient. The rational formula which is generally used is applied to this study.

Rational formula;

$$Q = \frac{1}{3.6} \times f \times \bar{r} \times A \quad \text{................. (4.10)}$$

Where;

Q : Run-off discharge (m^3/s)
A : Catchment area (km^2)
r : Hourly rainfall intensity in the time lag (mm/hr)
f : Run-off coefficient

The hourly rainfall intensity, "$\bar{r}$", is given by the rainfall intensity curve (refer to Fig. 3-3).

$$\frac{\bar{r}}{R} = \frac{b}{t + a} \qquad \text{.............................} \quad (4.11)$$

Where;

$\bar{r}$: Rainfall intensity (mm/hr)
R : Daily rainfall (mm)
t : Time lag (hr)
a,b : Coefficient obtained from hourly rainfall distribution

Probable discharge is estimated under the following conditions.

1) The hourly rainfall intensity can be estimated by applying the time lag mentioned in 4.2 to the rainfall intensity curve.

2) As mentioned in 4.2, the value 0.7 is employed as run-off coefficient. However, as shown in Table 4-5, run-off coefficient of the inner basin may vary according to the progress of urbanization.

Table 4-10 shows the probable discharge at other points calculated by the rational formula. Comparison of the probable discharges given by the rational formula and by the records of water stage at Kampili verifies that rational formula is suitable for the estimation of probable discharge in this basin.

Fig. 4-7 and Table 4-11 show the specific discharge and the design discharge of rivers in Indonesia respectively.

According to Fig. 4-7 the probable discharge of the Jeneberang river corresponds to those of other rivers in Indonesia.

Design Hydrograph

Design hydrograph was estimated at three sites; Bili-Bili, Kampili and Sungguminasa as shown in Fig. 4-8. The following equations give the design hydrograph.

$$Q(tn) = \frac{1}{3.6} \times f \times \bar{r}(tn) \times A$$

$$\bar{r}\ (tn) = \frac{\bar{r}\ (tn - T) + \bar{r}\ (tn - T + 1)\ \ldots.\ \bar{r}\ (tn)}{T} \quad ..(4.12)$$

Where;

Q(tn) : Discharge at arbitrary time, "tn". (m^3/s)
f : Run-off coefficient
A : Catchment basin (km^2)
$\bar{r}(tn)$: Hourly rainfall intensity (mm/hr)
T : Time lag (hr)

4.4 Design Flood for Overall Plan

Design flood discharge of a 50-year return period is employed for the formulation of an overall flood control plan.

Flood Control Capacity and Flow Down Discharge

Relation between the flood control capacity and the flow down discharge is studied by using the design hydrograph as described in the foregoing.

Flood control capacity can be estimated by integrating the remaining discharge of the design hydrograph after elimination of the flow down discharge (refer to Fig. 4-9).

Table 4-12 shows the relation betweeen the flood control capacity and the flow down discharge.

Optimum Scale of Reservoir and River Improvement

The optimum scale of reservoir and river channel is decided after making a comparative study of optimum scale between dam construction and river improvement. The total cost varies according to the share ratio described in this Supporting Report,"River Improvement".

The design flood distribution is decided in parallel with the decision of optimum scale of reservoir and river improvement. Fig. 4-10 shows standard project flood distribution and design flood distribution.

4.5 Design Flood for Urgent Plan

Natural Buffer

Design flood discharge of a 10-year return period is applied to formulation of the urgent flood control plan.

There is a natural buffer between the Sungguminasa bridge and the Kampili weir which can regulate the flood discharge. (Refer to Fig. 4-11.) The probable discharge described in 4.2 is studied on the assumption that the regulation effect of the area is not expected any more as the result of the river improvement.

In the stage of urgent plan, the regulation effect of the natural buffer will still be existing. Therefore, in the study of the probable discharge, the effects may have to be taken into consideration.

In the study of regulation effects, the following conditions are considered.

1) For the run-off calculation method, storage function method is applied. The simulation model is shown in Fig. 4-12.

2) Relation between the storage and the discharge (S-Q relation) is used. The relation is based on the result of non-uniform flow calculation and with reference to the topographic maps in scales of 1/50,000 and 1/10,000. (Refer to Fig. 4-13.)

3) For the hydrograph to be used in computation, the design hydrograph obtained at Kampili is applied (refer to Fig. 4-15). Fig. 4-14 shows the regulation effects of the natural buffer.

Design Flood Distribution for Urgent Plan

Design flood distribution for urgent plan is determined under the conditions that the natural buffer is still functioning. Therefore, a 10-year return period flood of 2,500 m^3/s at Kampili is to be regulated down to 2,100 m^3/s at Sungguminasa. The storage capacity of the natural buffer is 13.0×10^6 m^3.

4.6 Flow Regime

Flow regime is studied in the following procedure.

Study of areal mean rainfall

Run-off analysis by Tank Model Method

Calculation of flow regime

The low water discharge and water stage data collected only at the Bili-Bili station were used for the study on the flow regime.

Areal Mean Rainfall

Areal mean rainfalls for ten years are estimated by arithmatic mean method of daily rainfall data at three stations. The catchment basins whose areal mean rainfalls are estimated are as follows; Bili-Bili upper basin and Kampili upper basin. The stations, the data of which were used for the estimation, are also given in the following table.

Table 4-13 ESTIMATION OF AREAL MEAN RAINFALL

Catchment basin	Station
Bili-Bili upper basin	Malino and Bontobili
Kampili upper basin	Malino, Bontobili and Sungguminasa

Run-off Analysis by Tank Model Method

In this study, tank model method is applied for the estimation of the flow regime in the Jeneberang river.

First, the number of tank stages is determined. Usually, a serial three-stage tank model is applied to the study.

Run-off is calculated by using daily rainfall as inflow and by multiplying outlet coefficients. These are determined so that the calculated values may coincide with the actual discharge. In this study, daily discharge data at Bili-Bili in 1978 were used as the actual discharge data and areal mean daily rainfall data in the Bili-Bili upper basin in 1978 were used as the daily rainfall. Daily discharge is obtained by using the water stage data and the rating curve.

Flow Regime

The flow regime at Kampili and Bili-Bili is calculated by applying tank model method to the last ten annual records of rainfall data in the period from 1953 to 1978. The flow regime is shown in Table 4-14.

4.7 Utilizable Discharge

The utilizable discharge at Kampili is estimated in accordance with the flow chart shown in Fig. 4-16. In this flow chart, 5-day mean flow regime at Kampili and Bili-Bili, "Qk and Qb", in section 4.6 is applied. A storage capacity of 238×10^6 m^3 is applied for "Vmax". The utilizable discharge was calculated by using the last ten annual records of rainfall data from 1953 to 1978. Table 4-15 shows the utilizable discharge of each year.

5. TIDAL STAGE

Tidal stage in Makassar Strait is recorded at the Makassar Port tide-gauge station as mentioned before. Based on the well-kept records in 1976, 1977 and 1978, high and low water springs are selected at 25 each as samples for estimation of mean high and low water springs. Table 5-1 shows the mean high and low water springs. The model tidal stage is given by a sinecurve. This sinecurve is composed of the amplitude which represents the difference between high and low water springs, and the cycle which is the interval of 12 hours. Fig. 5-1 shows the model tidal hydrograph thus obtained.

Table 2- 1 EXISTING METEOROLOGICAL DATA

Name of Station	Start of Record	Item of Observation							Adminis-tration
		Tempera-ture	Rainfall	Sunshine	Air-pressure	Humidity	Wind	Evapo-ration	
Makassar	1971	○	○	○	○	○	○		PMG
Panakukkang	1971	○	○	○		○			PMG
Maros	1975	○	○	○		○		○	LPPM
Bontobili	1975	○	○						LPPM
Bonto Sunggu.	1977	○	○	○		○	○	○	PMA
Malino	1975	○							DIPERTA

Note : PMG - Pusat Meteorologie & Geofisika
PMA - Penyelidikan Masalah Air
DIPERTA - Dinas Pertanian
LPPM - Lembaga Prnrlitian Pertanian Maros

Table 2-2 EXISTING RAINFALL DATA (ORDINARY)

Name of station	Adminis-tration	Setting year	River System	Remarks
1. Manipi (419)	PMG	1969	-	out of Basin
2. Malino (419b)	PMG	1951	Jeneberang	in Basin
3. U.Pandang	PMG	1952	-	"
4. Sungguminasa	PMG	1912	Jeneberang	"
5. Bontowada	PMG	1947	Jeneberang	"
6. Borong Rapoa	PMG	1925	Jeneberang	out of Basin
7. Bonto Bili	PMG	1952	Jeneberang	in Basin
8. Limbung		1913	Jeneberang	out of Basin
9. Bontomanai	E.P	1975		"
10. Tamalayang (BPL I)	E.P	1975		"
11. Borong Lo'E	E.P	1975	Jeneberang	"
12. Barembeng	E.P	1975		"
13 Ko'bang	E.P	1975	Jeneberang	in Basin
14. Sungguminasa (B.S.VIII)	E.P	1925	Jeneberang	"
15. Macini Baji (BL IIab)	E.P	1975	Jeneberang	"
16. Senre	PMA	1975	Jeneberang	"
17. Intake Bili Bili	PMA	1975	Jeneberang	"
18. Kampili	PMA	1971	Jeneberang	"
19. Mandalle	E.P	1925		out of Basin
20. Palleko	E.P	1975		"
21. Tete Batu	E.P	1925	Jeneberang	"
22. Julu Bori BL I	E.P	1925		"
23. Tinggi Mae (BK III)	E.P	1975		"
24. Kala Bajeng (BL IIIb)	E.P	1975	Jeneberang	"
25. Tete Batu I (BP II)	E.P	1975	Jeneberang	"
26. Malino	PMA	1977	Jeneberang	in Basin
27. Bonto Sallang BS I	E.P	1975	Jeneberang	out of Basin
28. Sanro Bone	E.P	1975		"
29. Bonto Kassi B.B IV	E.P	1975		"
30. Campagaya	E.P	1975		"
31. Salo Jirang	PMA	1970	Maros	"
32. Bajeng (BPLE)	E.P	1975		"
33. Pekelli	E.P	1975	Maros	"
34. Tamangapa	PMA	1975	Tallo	in Bain
35. Manrinisi	E.P	1975	Maros	out of Basin
36. Panyalingan	E.P	1975	Maros	"
37. Maroanging	E.P	1975	"	"
38. Batu Bassi	PMA	1970	Maros	"
39. Tanra Lili	PMA	1970		"
40. Bonti Bonti	PMA	1970	Maros	"

Note: PMA - Penyelidikan Masalah Air
PMG - Pusat Meteorologi & Geofisika
E.P - Exploitasi & Pemeliharaan

Table 2- 3 EXISTING RAINFALL DATA (AUTOMATIC)

Name of Station	Start of Record	Administration	Remarks
Malino	1975	PMA	
Panakukkang	1973	PMG	
Hasanuddin	1963	PMG	

Note: PMA - Penyelidikan Masalah Air
PMG - Pusat Meteorologi & Geofisika

Table 2-4 DISCHARGE OBSERVATION (LOW WATER DISCHARGE)

Date year mon. day	Water stage* (m)	Velocity (m/s)	Current area (m2)	Discharge (m3/s)
1976 2 14	1.02	0.80	61.63	49.14
3 6	1.06	0.79	65.79	51.97
1977 9 30	0.08	0.05	19.65	0.90
1978 3 9	0.44	0.73	66.17	48.60
3 29	0.23	0.94	28.65	26.97
4 18	0.15	0.20	71.07	14.10
6 27	0.39	0.62	82.42	51.41
7 19	0.05	0.17	62.75	10.96
9 14	0.0	0.13	46.69	5.96
14 7	0.14	0.17	66.69	11.43
11 4	-0.05	0.05	56.36	2.87
12 4	0.42	0.58	66.57	38.65
1979 2 3	0.48	0.66	86.78	56.82
2 20	0.35	0.60	66.60	39.47
3 18	0.34	0.48	80.85	38.36
3 26	0.28	0.42	79.89	33.29
4 24	0.21	0.38	66.41	24.88
5 24	0.01	0.2	62.73	12.54

* : Water Stage from Gauge Zero

Table 2- 5 FLOOD MARKS

Kampili Weir		Sungguminasa Bridge	
Date year month day	Water Stage* (m.)	Date year month day	Water Stage (M.S.L.m)
1967 - -	6.35	-	-
-	-	1967 2 15	7.92
1968 - -	3.70	1968 1 13	7.49
1969 - -	2.10	-	-
1970 - -	2.30	-	-
-	-	1971 3 -	7.24
-	-	1972 1 2	7.73
-	-	1972 1 10	7.55

* : Water Stage from Gauge Zero

Table 2-6 EXISTING WATER STAGE DATA

Name of Stations	River System	Administration	Start of Record	Remarks
Kampili Weir	Jeneberang	E.P.	1966	Ordinary
Bili-Bili	Jeneberang	PMA	1974	Automatic
Jenelata	Jenelata	PMA P3SA	1974 1979	Ordinaly Automatic
Patompo	Jeneberang	P3SA	1979	Ordinary
Parantambung		P3SA	1979	Ordinary
Tallo	Tallo	P3SA	1979	Ordinary
Jembatan Tello	Tello	P3SA	1979	Ordinary
Pampang	Pampang	P3SA	1979	Ordinary
*Makassar Port	-	HYDRAL	1976	Automatic

Note : * - Tidal Stage
E.P. - Exploitasi & Pemeliharaan
PMA - Penelitian Masalah Air
P3SA - Proyek Perancang & Pengembangan Sumber 2 Air

Table 2- 7 MONTHLY MEAN TEMPERATURE

(Unit: °C)

Name of Station	Item	Jan.	Feb.	Mar.	Apr.	May.	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Makassar	Min.	22.6	22.6	22.6	22.3	22.4	21.8	21.0	21.0	21.5	21.9	22.4	22.8
	Ave.	25.6	25.5	25.7	26.1	26.2	25.9	25.5	26.0	26.6	26.7	26.5	25.8
	Max.	29.5	29.7	30.1	31.2	31.5	31.3	31.4	32.2	32.6	32.5	31.4	29.9
Panakkukang	Min.	22.9	22.7	22.9	22.5	22.5	21.8	20.4	20.4	21.1	22.0	23.0	23.2
	Ave.	25.4	25.2	25.6	26.2	26.1	25.6	25.1	25.7	26.3	27.0	26.5	25.6
	Max.	29.4	29.7	29.8	31.4	31.5	31.4	31.5	32.3	32.9	33.0	31.5	29.7
Maros	Min.	22.6	22.4	22.3	21.4	22.1	22.3	21.0	20.2	20.0	21.0	21.9	22.6
	Ave.	26.3	26.3	26.8	27.2	27.5	27.2	27.0	27.4	27.8	27.7	27.3	26.7
	Max.	28.3	28.5	29.7	30.3	30.5	30.1	30.2	30.3	30.8	31.4	30.4	28.6
Bontobili	Min.	21.6	21.6	21.6	21.2	21.4	20.7	19.6	19.8	19.5	20.7	21.4	21.8
	Ave.	-	-	-	-	-	-	-	-	-	-	-	-
	Max.	29.1	28.9	30.1	31.1	31.3	31.1	31.2	32.1	32.9	32.8	31.8	29.5
Bonto Sungu.	Min.	23.1	23.0	23.0	22.8	22.4	22.2	21.4	20.8	20.8	20.2	22.4	23.2
	Ave.	26.4	26.2	26.7	27.2	27.3	26.6	26.4	26.4	26.8	27.2	27.4	26.7
	Max.	29.6	29.2	30.4	31.4	32.2	31.0	31.2	32.0	33.1	34.1	32.4	30.2

Table 2- 8 MONTHLY MEAN HUMIDITY

(Unit: %)

Name of Station	Jan.	Feb.	Mar.	Apr.	May.	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Makassar	85.2	85.3	84.4	81.4	81.9	77.0	74.6	70.0	70.1	73.9	79.5	84.0
Panakkukang	87.0	87.0	86.2	82.6	81.6	80.4	77.1	70.8	70.9	70.8	77.0	85.6
Maros	86.7	87.3	87.0	84.0	83.0	80.8	77.8	69.0	76.7	78.7	83.0	83.3
Bonto Sunggu.	94.5	95.5	95.0	96.0	94.3	95.5	95.5	94.5	90.0	87.5	92.0	94.5

Table 2- 9 MONTHLY MEAN SUNSHINE RATIO

(Unit: %)

Name of Station	Jan.	Feb.	Mar.	Apr.	May.	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Panakkukang	37.3	31.2	34.8	62.7	60.7	61.6	69.9	80.5	71.0	71.5	51.7	29.4
Makassar	44.0	46.6	50.9	77.1	74.0	77.6	80.0	87.9	82.1	79.0	67.4	48.8

Table 2-10 MONTHLY MEAN WIND VELOCITY

(Unit: m/s)

Name of Station	Item	Jan.	Feb.	Mar.	Apr.	May.	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Makassar	Max.	25.4	24.9	26.2	17.9	17.1	16.9	16.6	19.1	19.9	21.4	20.8	20.8
	Ave.	0.4	0.4	0.4	0.4	0.3	0.4	0.4	0.5	0.5	0.4	0.4	0.4
Bonto Sunggu.	Max.	-	-	-	-	-	-	-	-	-	-	-	-
	Ave.	1.4	0.9	0.7	0.6	0.7	0.6	0.7	0.8	1.1	1.3	1.1	1.1

Table 2-11 MONTHLY MEAN EVAPORATION

(Unit: mm/day)

Name of Station	Jan.	Feb.	Mar.	Apr.	May.	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Maros	3.9	3.5	3.7	4.2	3.9	4.1	4.7	3.9	4.7	4.4	3.5	2.8
Bonto Sunggu.	4.3	3.6	4.2	4.6	3.8	3.6	3.6	4.1	5.5	6.0	5.9	4.4

Table 2-12 MONTHLY MEAN RAINFALL

(* Unit: mm)

Rainfall Station	Item	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Total	Remarks
Malino	Monthly * rainfall	895	724	532	332	210	164	91	38	40	113	286	577	4002	1959 - 1978
(N=12)	rainfall days	28	24	24	19	17	15	11	5	4	7	18	25	197	
BontoBili	Monthly rainfall	675	640	385	293	239	112	118	70	83	122	395	587	3719	1953 - 1978
(N =12)	rainfall days	21	21	18	13	12	8	7	6	4	7	17	22	156	
Sunggu-minasa	Monthly rainfall	626	638	346	129	124	78	58	76	16	71	324	531	2967	1953 - 1978
(N=13)	rainfall days	19	19	14	7	8	5	4	3	2	5	13	21	122	
Ujung-Pandang	Monthly rainfall	662	504	347	187	127	66	35	32	9	70	182	579	2800	1941 - 1956
(N=9)	rainfall days	22	18	16	9	8	7	5	3	1	6	11	22	128	

(NOTE: N - Number of Samples)

Table 3-1 PROBABLE DAILY RAINFALL*(POINT RAINFALL)

(Unit; mm/day)

Return period (yr)	Malino	Bohto-bili	Sunggu-minasa	Ujungpandang
200	347	299	319	338
100	317	278	293	309
80	307	271	285	299
50	286	257	268	279
30	264	241	249	258
10	214	207	207	210
5	181	184	180	178
2	132	150	138	131

* : By Gumbel Method

Table 2-3 ANNUAL MAXIMUM AREAL MEAN DAILY RAINFALL

Rank	Bili-Bili Upper basin		Sungguminasa Upper basin		Tallo river basin		Inner basin	
	Date	Rainfall	Date	Rainfall	Date	Rainfall	Date	Rainfall
1	1960-2-25	202mm	1976-1-12	188mm	1977-1-24	194mm	1938-1-8	185mm
2	59-1-8	174	60-2-25	182	78-12-25	186	50-4-10	185
3	53-2-6	163	53-2-6	149	53-2-7	183	77-1-24	179
4	77-1-24	134	77-1-24	148	60-2-25	154	53-2-7	156
5	56-1-16	123	72-1-9	147	76-1-12	153	76-1-12	148
6	76-1-12	122	59-1-8	143	72-1-9	147	29-1-10	147
7	78-1-11	120	55-11-14	119	59-3-12	125	54-12-17	143
8	73-1-22	102	56-12-15	106	55-11-14	102	33-2-21	135
9	79-1-9	94	78-1-11	104	56-12-15	98	55-5-17	128
10	58-1-4	92	73-1-11	101	54-1-20	94	40-1-4	127
11	54-1-27	69	61-1-18	97	73-1-11	87	39-1-25	125
12	75-1-13	36	67-2-5	95	75-11-30	87	78-12-25	123
13			58-12-29	92	79-1-7	83	34-1-22	123
14			68-1-14	87	58-12-5	82	36-4-9	123
15			79-1-10	84	57-2-20	73	30-12-21	116
16			62-2-2	83	74-12-13	56	56-1-27	113
17			74-3-1	78	66-1-17	54	28-1-22	112
18			54-2-2	77			35-3-5	105
19			57-2-20	68			47-12-30	98
20			52-2-23	65			37-2-4	97
21			75-11-30	64			73-1-11	97
22			66-2-14	61			41-1-2	89
23			69-2-4	34			79-1-7	88
24							58-12-5	87
25							75-11-30	87
26							32-12-9	79
27							49-1-12	76
28							31-4-1	70
29							74-12-14	65
30							66-1-17	54
31							57-2-25	47
32							52-2-23	45
33								
34								
35								

Table 3- 4 PROBABLE AREAL MEAN DAILY RAINFALL

(Unit; mm/day)

Return Period (T year)	Bili-Bili Upper basin ($A=384.4km^2$)	Sungguminasa Upper basin ($A=673.0km^2$)	Tallo river basin ($A=417.3km^2$)	Inner basin ($A=60.5km^2$)
200	321	274	319	267
100	302	249	289	245
80	292	237	280	237
50	271	224	259	222
30	247	206	237	204
20	229	191	220	191
10	197	165	189	167
5	146	138	157	142
2	113	97	109	105

Note :

1) : In this study, the rainfall of 407mm at Tamalatang station was not included.

2) : This probable rainfall was based on Gumbel Method.

Table 4-1 TIME LAG IN FLOOD PEAKS

River Syst.	Section	Stream length (km)	Gradient (I)	Water depth (m)	Roughness coeffi-cient	Average velocity (m/s)	Time lag (hr)	Total (hr)
Jeneberang	Concentra-tion	5	1/50	0.2	0.6	1	1.0	1.0
	-Bili Bili	45.5	1/150	2	0.045	3	4.0	5.0
	-Kampili	11.5	1/375	2	0.040	2	1.5	6.5
	-Sunggu-minasa	11	1/1000	3	0.035	2	1.5	8.0
	-River mouth	9	1/1000	3	0.035	2	1.9	9.5
Jenelata	Concentra-tion	-	-	-	-	-	-	1.0
	Confluence point	34	1/80	1.5	0.045	3	3	4.0
Tallo	Concentra-tion	-	-	-	-	-	-	1.0
	-River mOuth	60	1/3000	2	0.035	0.9	19.0	20.0

Table 4-2 ROUGHNESS COEFFICIENT

H (m)	H+b (m)	V (m/s)	n	Remarks
0.44	0.61	0.73	0.051	I = 1/375
0.23	0.39	0.94	0.030	b = 0.17
0.39	0.56	0.62	0.047	$n = \frac{1}{V} I^{1/2}H^{2/3}$
0.05	0.22	0.17	0.038	
0.48	0.65	0.655	0.059	
Ave.			0.045	

I : Surface Water Slope

b : Water Depth from River Bed to Gauge Zero

H : Water Depth from Gauge Zero

V : Velocity

n : Roughness Coefficient

Table 4-3 RUN-OFF COEFFICIENT IN THE BILI-BILI UPPER BASIN

(Water Stage from Gauge Zero)

Date			Water Stage (m)	Discharge (m^3/s)	$\bar{R}5$ (mm/hr)	f	Remarks
Year	M'th	Day					
1977	1	6	2.14	700	11.4	0.57	
"	"	7	2.45	900	12.0	0.70	
"	"	10	2.20	737	10.0	0.68	
"	2	14	3.01	1329	13.5	0.92	
"	"	17	2.78	1143	14.4	0.75	
1978	1	10	2.45	890	17.2	0.49	
"	"	11	1.73	475	7.5	0.59	
Average						0.70	

Discharge : $Q = 11.48\ (H+0.166)^2$

$\bar{R}5$: Rainfall Intensity During 5 Hours at Malino Rainfall Station

$$f = \frac{Q \times 3.6}{\bar{R}5 \times A}$$

Table 4-4 RUN-OFF COEFFICIENT FOR DIFFERENT AREA

Area	Run-Off Coefficient
Mountainous (Am)	0.7
Paddy Field (Ap)	0.7
Urban (Au)	0.8
Inundation (Ai)	1.0

Table 4-5 RUN-OFF COEFFICIENT IN THE DRAINAGE AREA

Stage	City-Side Area					Mountain-Side Area				
	Am	Ap	Au	Ai	f	Am	Ap	Au	Ai	f
Existing	0	20%	50%	30%	0.85	5%	60%	5%	30%	0.80
First Stage	0	5%	65%	30%	0.86	5%	60%	5%	30%	0.80
Second Stage	0	5%	65%	30%	0.86	5%	45%	20%	30%	0.81
Third Stage	0	5%	65%	30%	0.86	5%	25%	40%	30%	0.83

Table 4-6 MAXIMUM FLOOD STAGE AND WATER STAGE OF FIXED TIME AT BILI-BILI STATION

Date			Maximum Flood Stage*	Water Stage of Fixed time*
year	month	day		
1977	1	6	2.14 m	1.21 m
"	"	7	2.39	2.29
"	"	8	2.18	2.00
"	"	10	3.01	1.94
"	"	24	2.10	3.07
"	2	14	3.01	3.00
"	"	17	2.78	2.78
1978	1	11	2.43	2.07
"	"	24	3.07	3.07
Average			2.53	2.30

Note:
*: Height from Gauge Zero

Table 4- 7 ANNUAL MAXIMUM DISCHARGE AT KAMPILI WEIR

Date			1) Water stage* (m)	2) Water stage* (m)	3) Discharge (m^3/s)	Remarks
Year	M'th	day				
1966	2	26	3.35	3.69	1,409	
67	2	5	6.55	6.55	3,352	
68	12	29	3.30	3.63	1,378	
69	2	4	2.60	2.86	964	
70	2	14	2.20	2.42	750	
71	1	27	1.90	2.09	602	
72	1	13	2.45	2.70	881	
73	11	23	1.45	1.60	401	
74	2	13	3.40	3.74	1,441	
75	4	17	2.45	2.70	881	
76	1	15	2.50	2.75	909	
77	2	17	4.40	4.84	2,121	
78	1	11	3.00	3.30	1,194	
79	1	11	2.80	3.08	1,077	

1) : Water Stage Data of Kampili Weir

2) : Water Stage Data of Kampili Weir
After Modification by Using this equation
$H(2) = H(1) \times 1.1$

3) : $Q = C \times B \times H^{3/2}$ (C =2.0, B = 100m)
(H: Water Depth)

* : Water Stage from Gauge Zero

Table 4-8 CORRELATION OF WATER STAGE, KAMPILI VERSUS BILI-BILI

No.	Date			Water Stage *		Remarks
	Year	M'th	day	Bili-Bili	Kampili	
1	1975	1	4	1.52 m	0.88 m	
2	"	12	12	1.56	0.93	
3	1976	3	19	1.61	1.05	
4	1977	1	7	2.02	1.85	
5	"	"	8	1.88	1.85	
6	"	"	9	1.78	1.43	
7	"	"	10	1.74	1.05	
8	"	"	11	1.62	0.88	
9	"	"	25	1.99	1.68	

$$H_{Bili2} = 0.42 \times H_{Kampili} + 1.21$$

H_{Bili2}: Water stage of Bili-Bili

$H_{Kampili}$: Water stage of Kampili

* : Water Stage from Gauge Zero

Table 4-9 PEAK DISCHARGE RATIO

No.	Date			Kampili Point		Bili-Bili Point		Ratio
	Year	M'th	day	Water Stage	Discharge (1)	water Stage	Discharge (2)	(2)/(1)
				m	m^3/s	m	m^3/s	
1	1966	2	26	3.35	1,226	2.62	1,022	-
2	1967	2	5	6.55	3,352	3.96	2,244	-
3	1968	12	29	3.30	1,198	2.60	1,005	-
4	1974	2	13	3.40	1,254	2.64	1,036	-
5	1977	2	17	4.40	1,846	3.06	1,370	-
	Average			-	1,775	-	1,335	0.75

Table 4-10 PROBABLE DISCHARGE

(Unit: m^3/s)

	Jeneberang river				Tallo river	Inner basin				
Return period (Year)	Bili-Bili (384.4km2)	Kampili weir (623.9km2)	Sungguminasa (673.6km2)		Whole basin (417.3km2)	Mountain-side area (45.4km2)			City-side area (15.1km2)	
						(1)*	(2)*	(3)*	(1)**	(2)**
100	3,122 (8.1)	4,163 (6.7)	-	4,163 (6.2)	2,125 (5.7)	386 (8.5)	390 (8.6)	400 (8.8)	339 (22.5)	343 (22.7)
50	2,748 (7.1)	3,664 (5.9)	-	3,664 (5.5)	1,905 (5.1)	349 (7.7)	354 (7.8)	363 (8.0)	307 (20.3)	311 (20.6)
30	2,471 (6.4)	3,294 (5.3)	-	3,294 (4.9)	1,743 (4.7)	321 (7.1)	325 (7.2)	333 (7.3)	282 (18.7)	286 (18.9)
20	2,249 (5.9)	2,998 (4.8)	-	2,998 (4.5)	1,618 (4.3)	301 (6.6)	304 (6.7)	312 (6.9)	264 (17.5)	267 (17.7)
10	1,862 (4.8)	2,483 (4.0)	2,085Δ (3.1)	2,483 (3.7)	1,390 (3.7)	263 (5.8)	266 (5.9)	273 (6.0)	231 (15.3)	234 (15.5)
5	1,461 (3.8)	1,948 (3.1)	1,670Δ (2.5)	1,948 (2.9)	1,155 (3.1)	223 (4.9)	226 (5.0)	232 (5.1)	196 (13.0)	199 (13.2)
2	854 (2.2)	1,138 (1.8)	1,090Δ (1.6)	1,138 (1.7)	802 (2.2)	165 (3.6)	167 (3.7)	171 (3.8)	145 (9.6)	147 (9.7)

Note :
- Jeneberang river basin : Probable discharge based on the discharge data
- Tallo river and Inner basin : Probable discharge based on the rainfall and run-off analysis
- Urban development stage : (1)* - Existing, First stage (2)* - Second stage (3)* - Third stage
  (1)** - Existing (2)** - First, Second and Third stage
- Δ : Discharge after regulation
- ( ) : Specific discharge

Table 4-11 DESIGN DISCHARGE OF RIVERS IN INDONESIA

No.	Name of River	Province	Catchment Area (km^2)	Design Flood (m^3/s)	Return Period (yr)
1	Sungai Cimanuk	West Jawa	3,006	1,440	25
2	Kali Serang	Central Jawa	937	900	25
3	Sungai Citanduy	West Jawa	3,680	1,900	25
4	Sungai Ular	North Sumatra	1,080	800	25
5	Kali Pemali	Central Jawa	1,228	1,300	25
6	Sungai Cipanas	West Jawa	220	385	25
7	Bengawan Solo	Central/East	3,320	2,000	40
8	Kali Madiun	East Jawa	2,400	2,300	40
9	Sungai Wampu	North Sumatra	3,840	1,320	20
10	Sungai Arakundo	Aceh	5,495	1,800	20
11	Sungai Kring Aceh	Aceh	1,775		20
12	Kali Brantas	East Jawa	10,000	1,500	50
13	Sungai Bah Bolon	North Sumatra	2,776	1,200	20
14	Sungai Walanae	South Sulawest	3,190	2,900	20
15	Sungai Bila	South Sulawest	1,368	1,900	20

Table 4-12 FLOOD CONTROL CAPACITY AND FLOW DOWN DISCHARGE

Flow down discharge	Flood control capacity	
	Net volume	Actual capacity*
500 m3/s	48.70×10^6 (m3)	58.5×10^6(m3)
800	38.9	46.7
1,000	33.1	39.7
1,500	20.3	24.4
1,550	19.2	23.2
1,700	16.0	19.2
2,000	9.5	11.4

* Considering 20% of safety facter to the net volume

Table 4-14 FLOW REGIME OF JENEBERANG RIVER

Location	Year	(A)	(B)	(C)	(D)	Annual Run-off Volume
Bili-Bili	1953	37.6 m^3/s	13.0 m^3/s	4.8 m^3/s	4.1 m^3/s	986 $\times 10^6 m^3$
	1956	46.9	18.6	8.2	3.3	1,069
	1957	46.9	10.4	4.5	3.8	[illegible]
	1958	39.1	17.9	4.7	4.2	875
	1959	45.3	19.2	5.1	4.4	1,105
	1960	34.3	17.0	5.1	4.3	947
	1975	45.3	21.9	3.8	3.2	947
	1976	30.3	9.5	4.0	3.3	796
	1977	54.7	11.6	4.3	3.5	1,311
	1978	38.4	16.8	8.2	4.2	904
Kampili	1953	44.5	16.6	6.5	5.5	1,205
	1956	54.7	26.0	11.9	5.3	1,487
	1957	48.0	10.7	6.6	5.5	1,234
	1958	50.2	18.4	6.5	5.6	1,099
	1959	60.0	22.2	7.0	5.8	1,496
	1960	52.5	22.2	7.0	5.8	1,430
	1975	65.0	29.1	5.4	4.9	1,349
	1976	51.5	12.7	6.1	5.0	1,228
	1977	70.8	16.1	6.6	5.3	1,923
	1978	58.3	23.9	10.7	6.3	1,343

Note: (A); The 95th largest discharge of the year
(B); The 185th largest discharge of the year
(C); The 275th largest discharge of the year
(D); The 355th largest discharge of the year

Table 4-15 UTILIZABLE DISCHARGE AT KAMPILI

Year	Utilizable Discharge
1953	23 m^3/s
1956	32
1957	20
1958	24
1959	27
1960	25
1975	24
1976	21
1977	24
1978	32

Table 5-1 MEAN HIGH AND LOW WATER SPRINGS

* Tidal Stage from Gauge Zero

Date			Tidal stage * (cm)		Remark
Year	Month	Day	High tide	Ebb tide	
1976	3	13	163	72	
	4	5	172	66	
	4	17	176	71	
	5	2	147	31	
	5	17	167	38	
	5	31	156	48	
	6	15	160	41	
	7	5	165	52	
	10	26	166	40	
	11	9	168	57	
1977	1	6	187	73	
	2	15	170	55	
	3	11	165	45	
	4	8	184	45	
	4	21	172	65	
	5	6	173	54	
	5	21	165	61	
	6	5	167	46	
	8	3	148	73	
	8	16	148	47	
	10	3	157	52	
	10	16	162	48	
	10	29	164	54	
	11	13	175	48	
	12	10	175	62	
Average			166	54	

Fig. 2-1 LOCATION MAP OF METEOROLOGY STATIONS

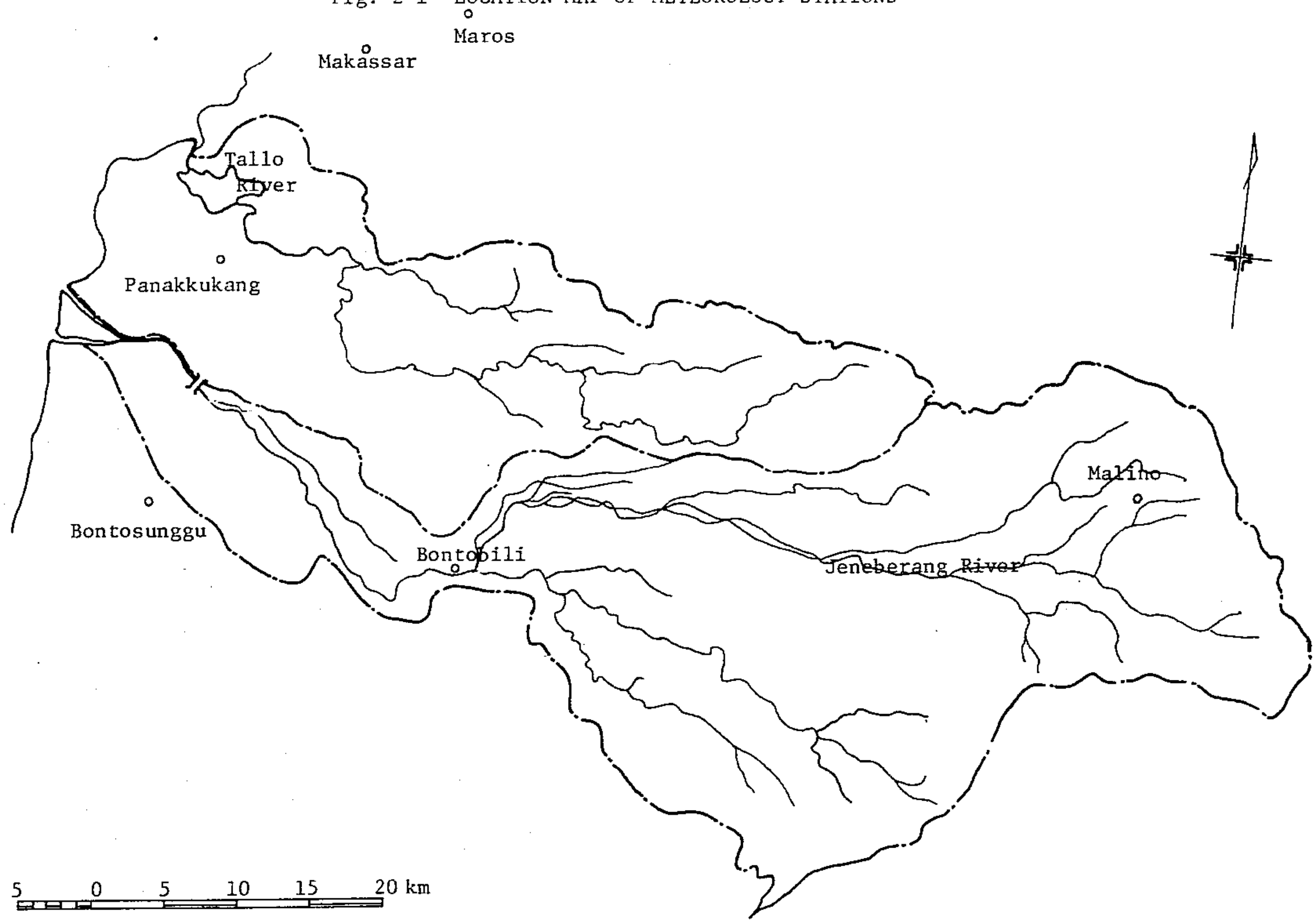


Fig. 2-2 EXISTING RAINFALL DATA

○ : Daily Rainfall Data available
△ : Partially No Data

	Year / Location	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	Remark
1	Manipi (419)																																																												○	△	△	△	○		○	○	○	○		
2	Malino (419b)																																										△	△	△			△	△	△	○	△	○	○	△	△	○	○	○	○	○	△	○	○	△		○	○	○	○	△	
3	U.Pandang (422a)																			△	○	△	○	○	○	○	○	○	△	○	○	△	○							○	○	○	○	△	○	○	○	△	△	△	△			△										△	○	○	○					
4	Sungguminasa (423)			○	○	○	○	○	○	○	○	○	○	○	○	○	△		△	○	○	○	△	△	△	△	△	△	△	△	△	△	△							△	△	△	○	△	△	○	○	○	○	△	○	○	△	△			△	○	△	△	△		△		○	△	○		○	○		
5	Bontowada (Pagentung)																																						○	○	○	○	○	○	○	○	○	○	○	○	○	△	△						△				△	△	○	○	○					
6	BorongRapoa																		○	○	○	○	○	○	○	○	○	○	○	○	○	○	△											△	○	△	○																				○					
7	Bontobili																																											△	○	○	○	△	○	○	○	△											△	○	○	○	△	○	△	○		
8	Limbung				△	○	○	△	○	○	○	△	○	○	○	○	△	△	△	○	○	○	△	○	○	○	○	○	○	○	○	○	△								△	△																														
9	Bontomanai																																																																		△	○	○	○	△	
10	Tamalayang (BPL I)																																																																		△	○	○	○	△	
11	Borong Lo'E																																																																		△	○	○	○	△	
12	Barembeng																																																																			○	○	○	△	
13	Ko'bang																																																																		△	○	△	○	△	
14	Sungguminasa (BS VIII)																																																																		△	○	○	○	△	
15	Macini Baji (BL I lab)																																																																		△	○	○	○	△	
16	Senre																																																																		△	○	○	○	△	
17	Intake BiliBili																																																																		△	○	○	○	△	
18	Kampili																																																														△	△		△	○	○	○	○	△	
19	Mandalle																																																																		△	○	○	○	△	
20	Palleko																																																																		△	○	○	○	△	
21	Tete Batu Cambara																																																																		△	○	○	○	△	
22	Julu Bori (b.L.I)																																																																		△	○	○	○	△	
23	Tinggi Mae (BK III)																																																																		△	○	○	○	△	
24	KalaBajeng (BL IIb)																																																																		△	○	○	○	△	
25	Tete Batu I (BP II)																																																																		△	○	△	○	△	
26	Malino																																																																				△	○	△	
27	Bonto Sallang (BS I)																																																																		△	○	○	○	△	
28	Sanro Bone																																																																		△	○	○	○	△	
29	BontoKassi (BB IV)																																																																		△	○	○	○	△	
30	Sampagaya																																																																		△	○	○	○	△	
31	Salo Jirang																																																												○	○	○	○	○	○	○	○	○	○	△	
32	Bajeng (BPLE)																																																																		△	○	○	○	△	
33	Pokelli																																																																		△	○	○	○	△	
34	Tamangapa Kassi																																																																		△	○	○	○	△	
35	Manrimisi																																																																		△	○	○	○	△	
36	Panyalingan																																																																		△	○	○	○	△	
37	Maroanging																																																																		△	○	○	○	△	
38	Batu Bassi																																																													○	○	○	○	○	○	○	○	○	△	
39	Tanra lili																																																													○	○	○	○	○	○	○	○	○	△	
40	Bonti-Bonti																																																													○	○	○	○	○	○	○	○	○	△	

Fig. 2-3 LOCATION MAP OF DAILY RAINFALL STATIONS

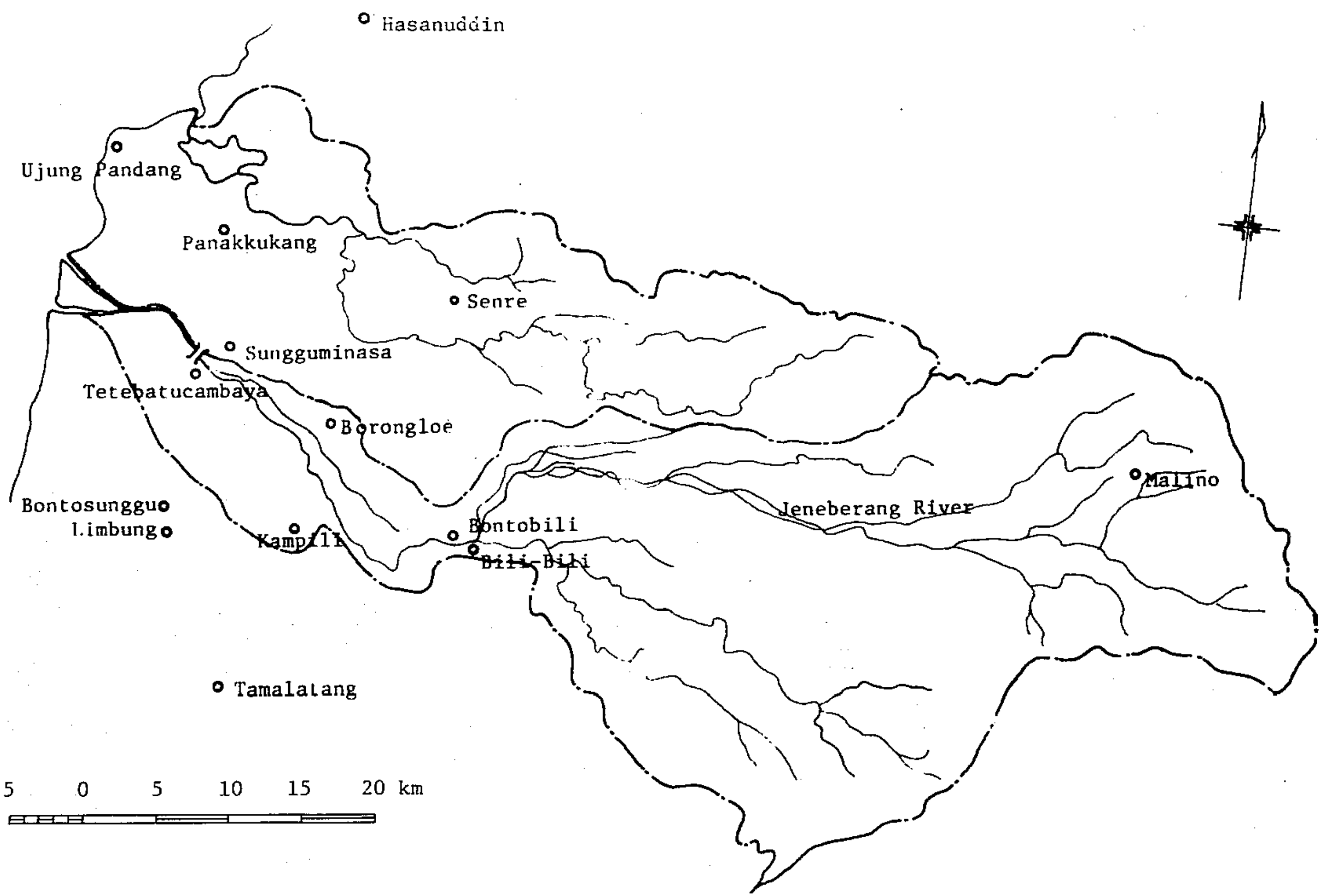


Fig. 2-4 LOCATION MAP OF AUTOMATIC RAINFALL STATIONS

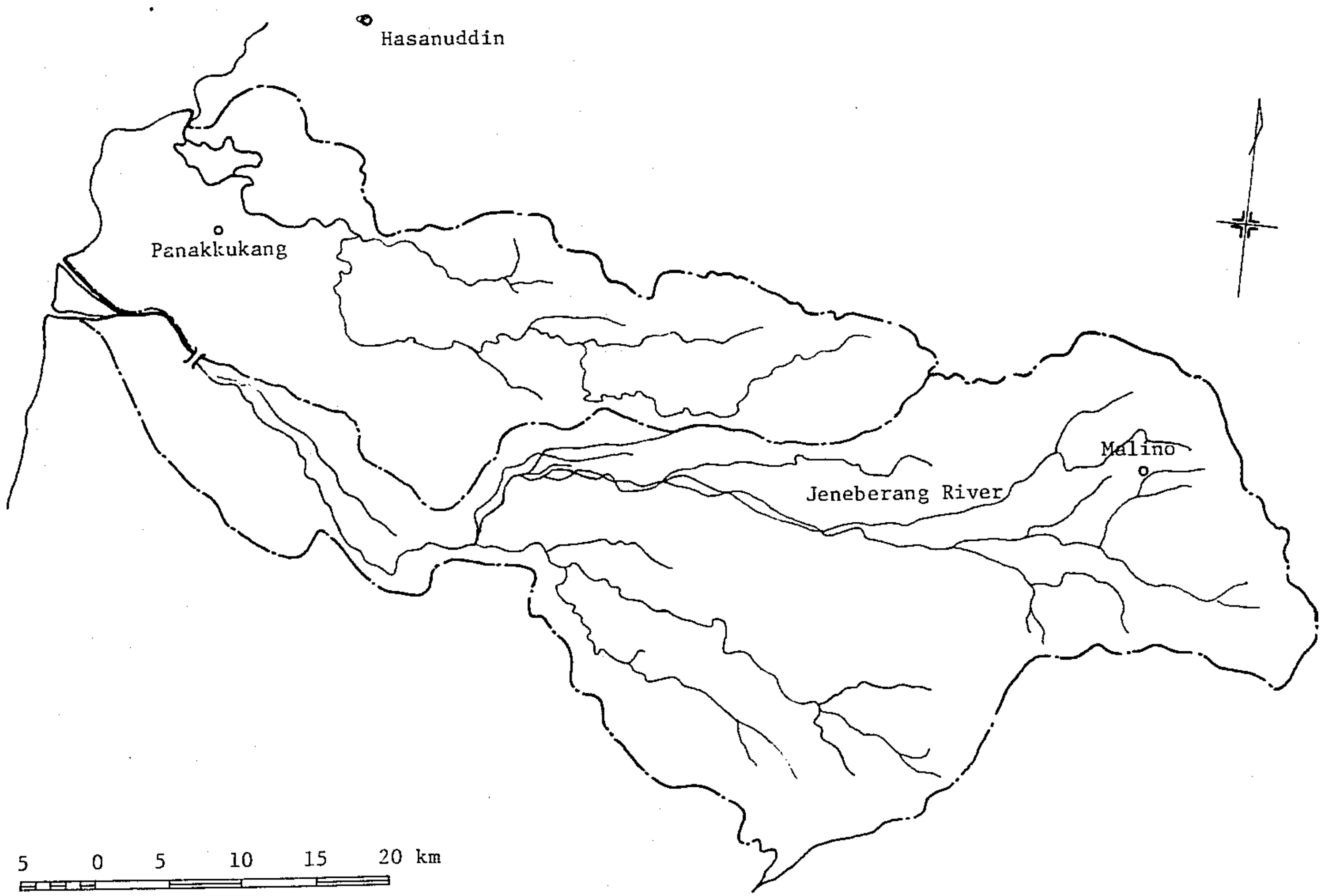


Fig. 2-5 LOCATION MAP OF WATER STAGE STATIONS

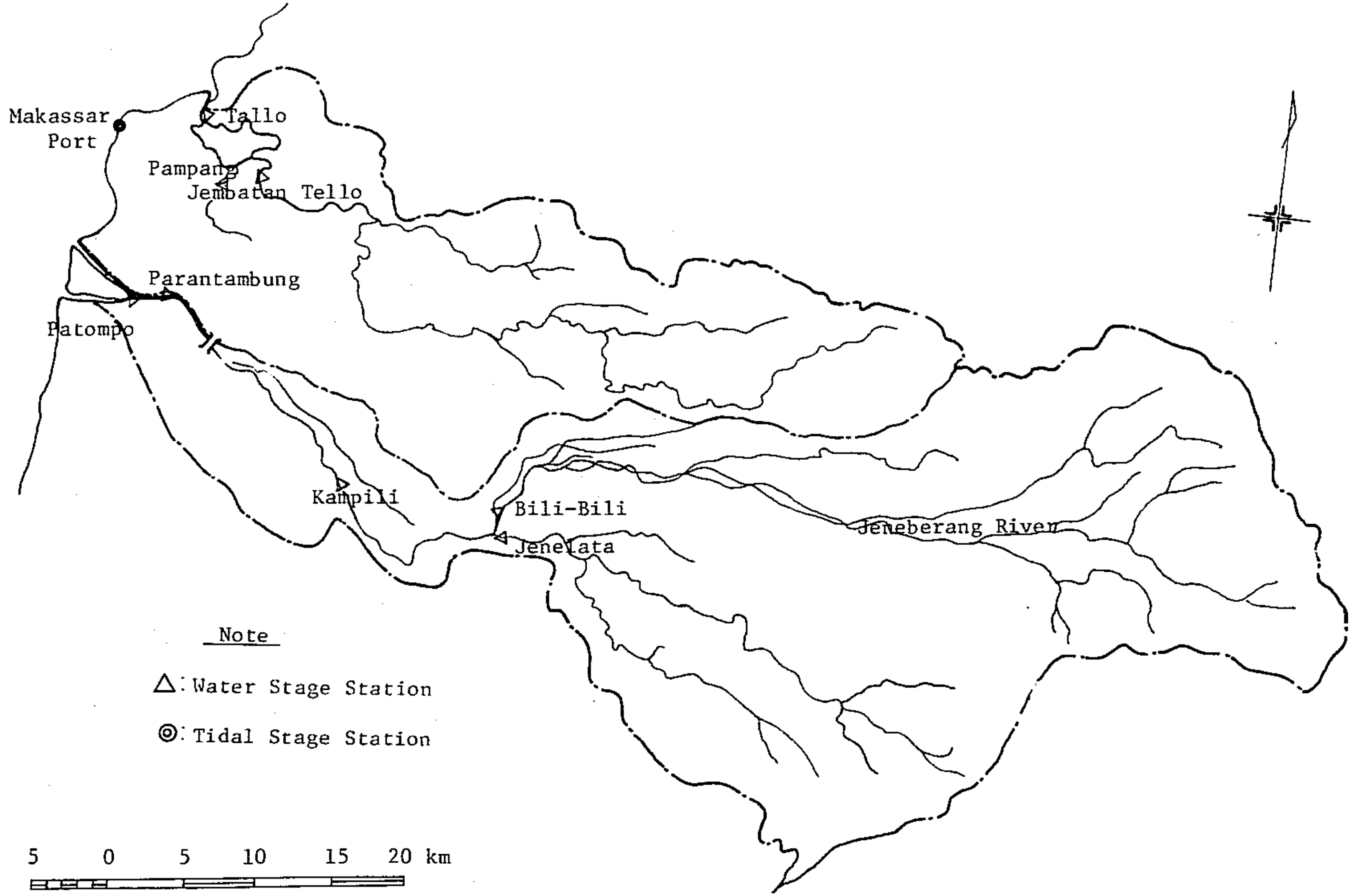


Fig. 2-6(1) MONTHLY MEAN TEMPERATURE

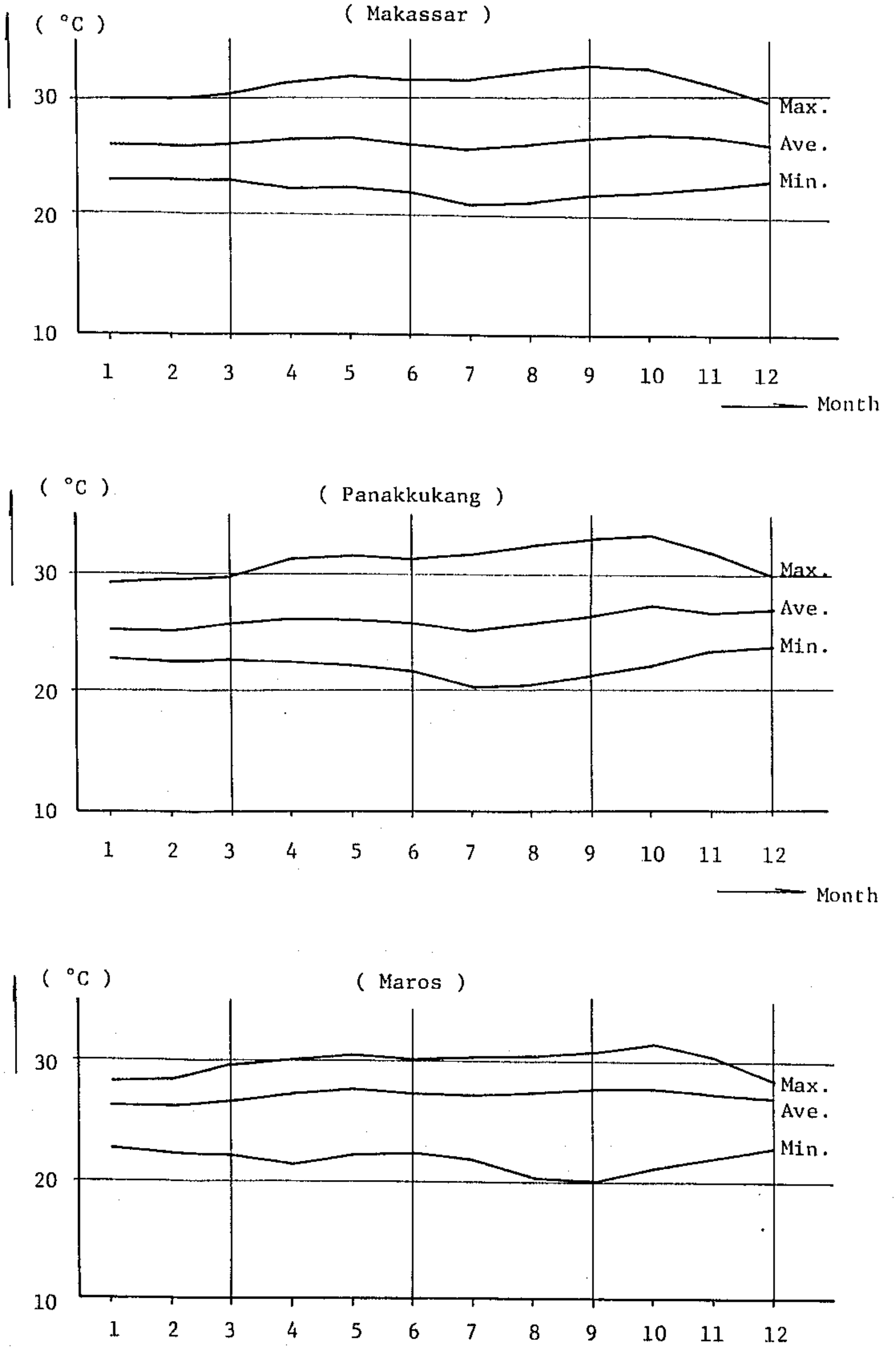
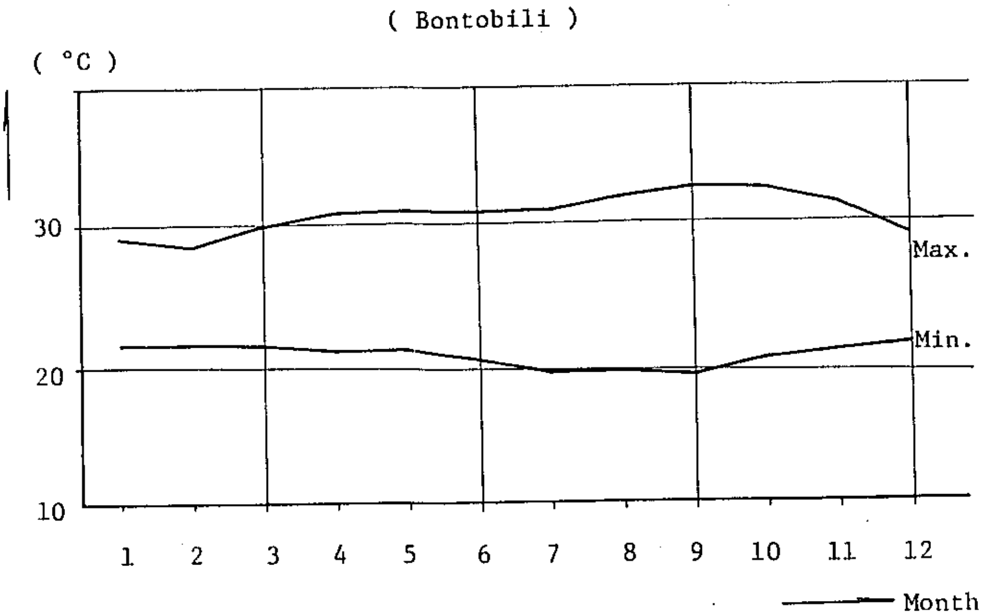


Fig. 2-6(2) MONTHLY MEAN TEMPERATURE



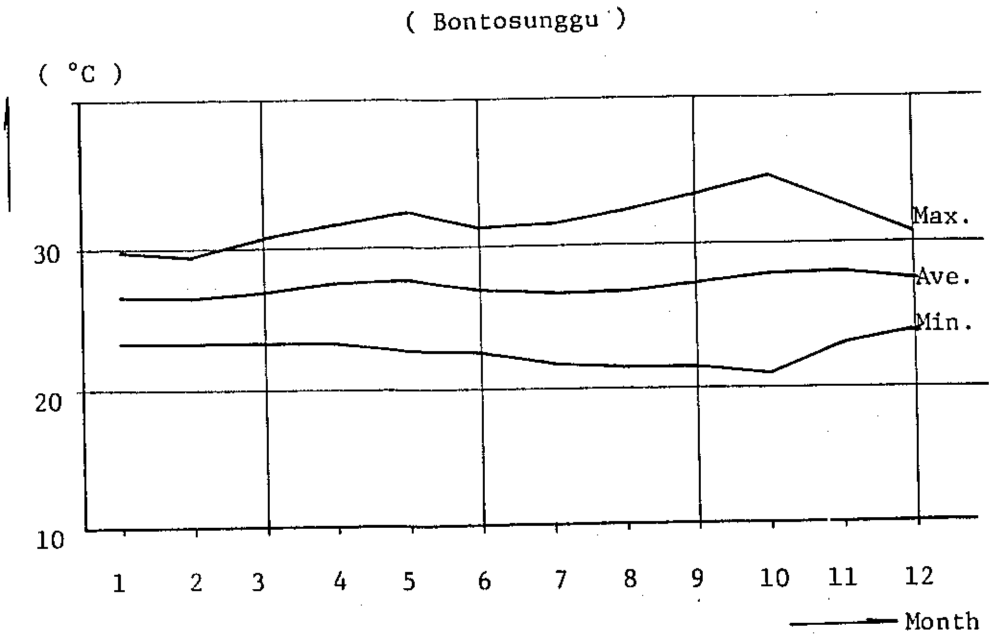


Fig. 2-7 MONTHLY MEAN HUMIDITY

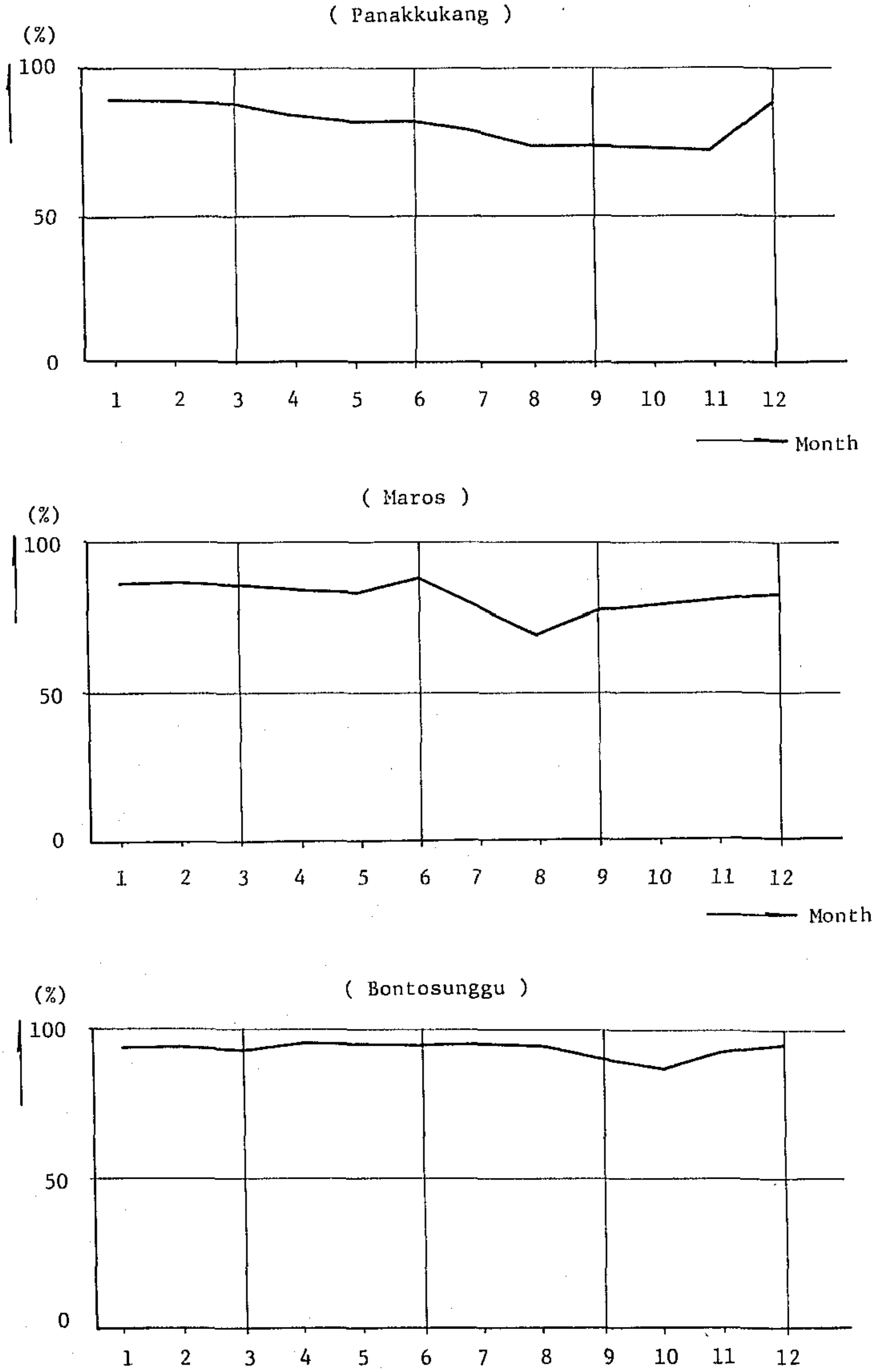
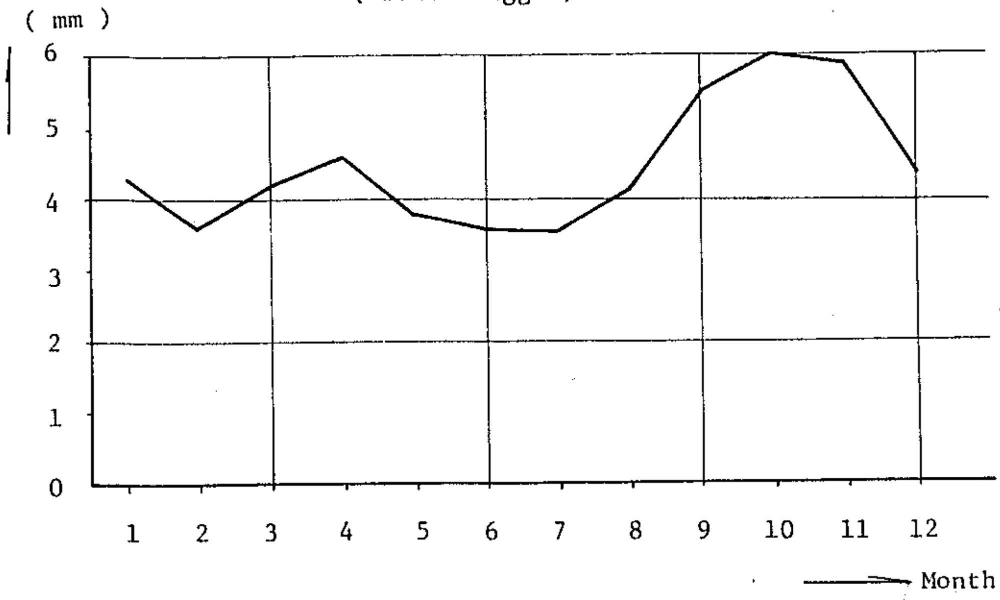


Fig. 2-8 MONTHLY MEAN EVAPORATION

(Bontosunggu)



(Maros)

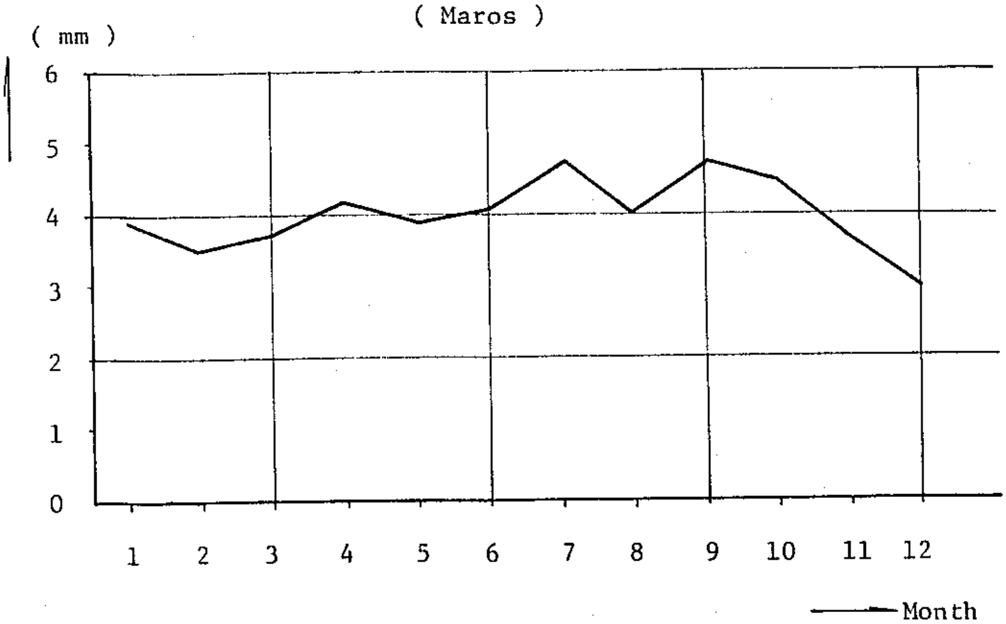
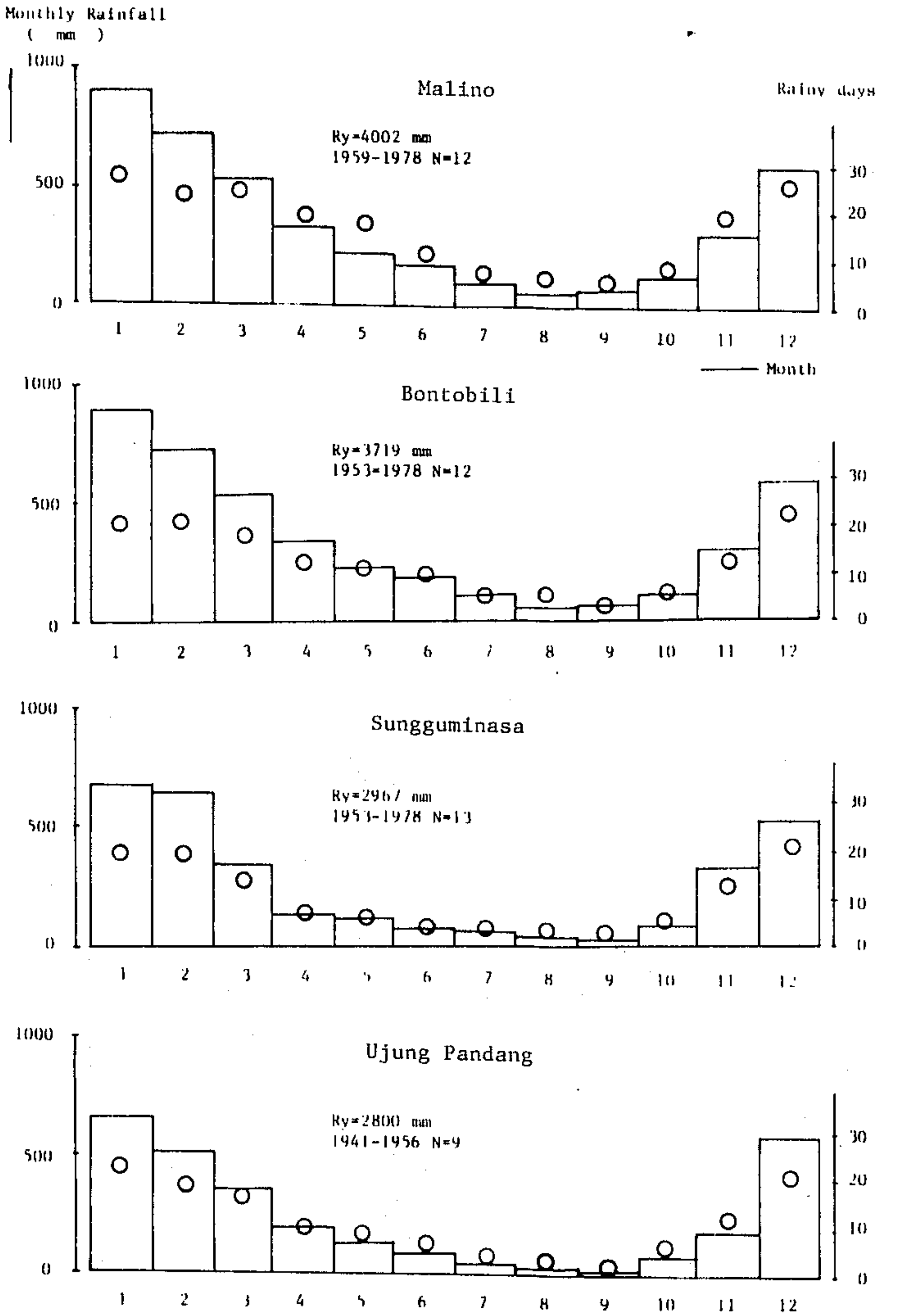


Fig. 2-9 MONTHLY MEAN RAINFALL



Ry:Annual Rainfall

:Rainfall

O :Rainy days

Fig. 2-10(1) DAILY RAINFALL DURING MAJOR FLOODS

(28, 29, 30 /11/1975)

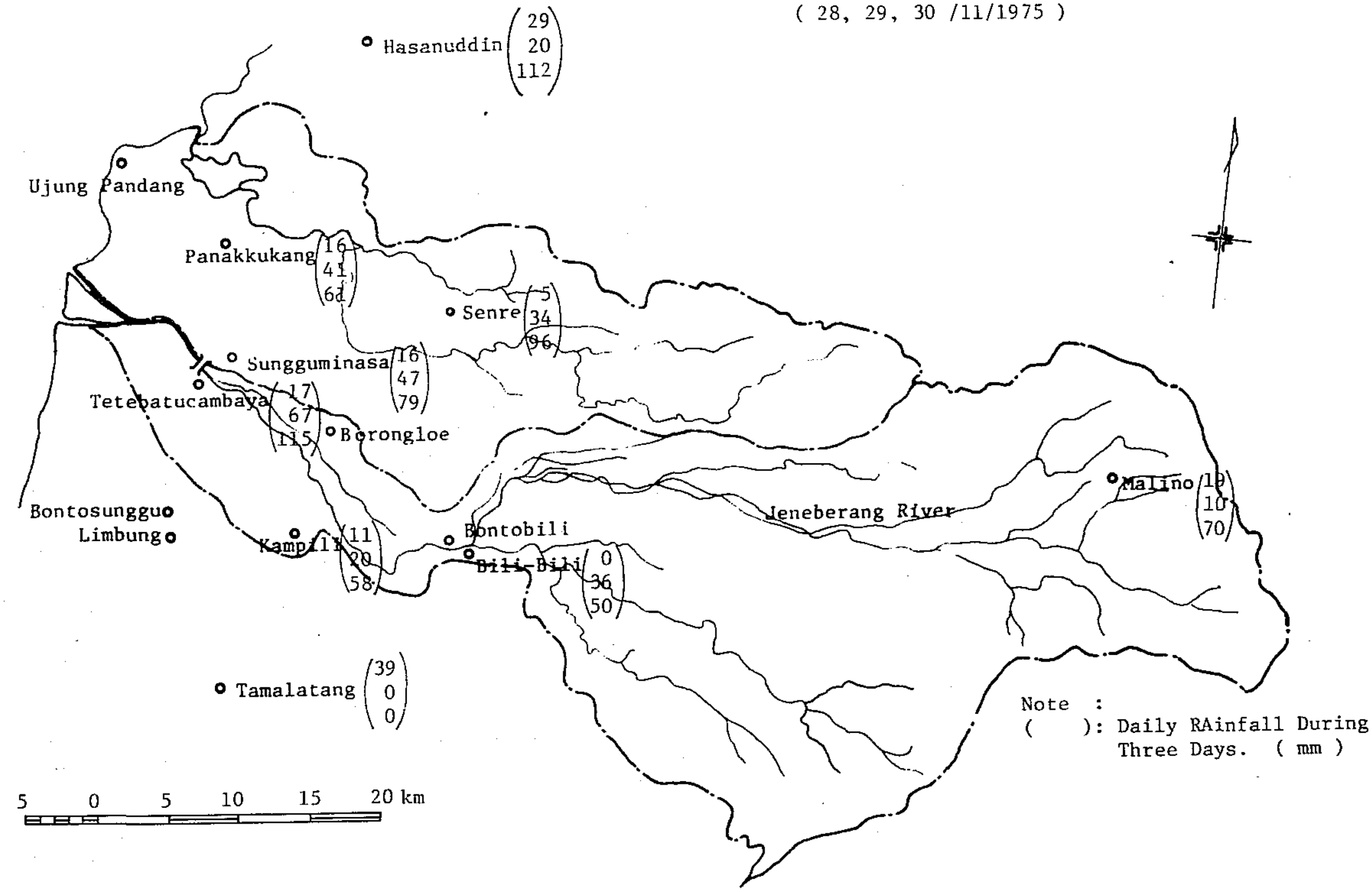
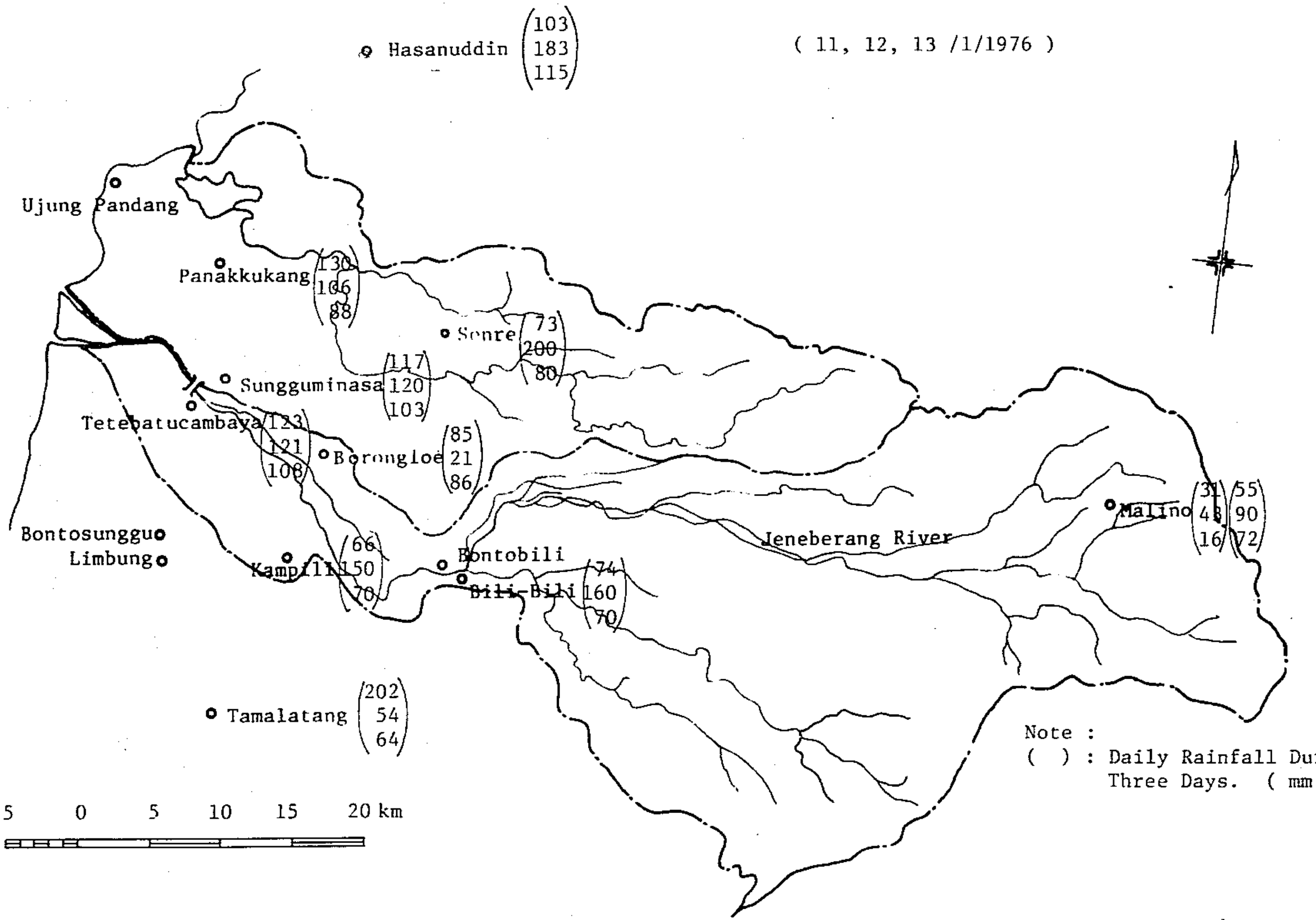


Fig. 2-10(2) DAILY RAINFALL DURING MAJOR FLOODS

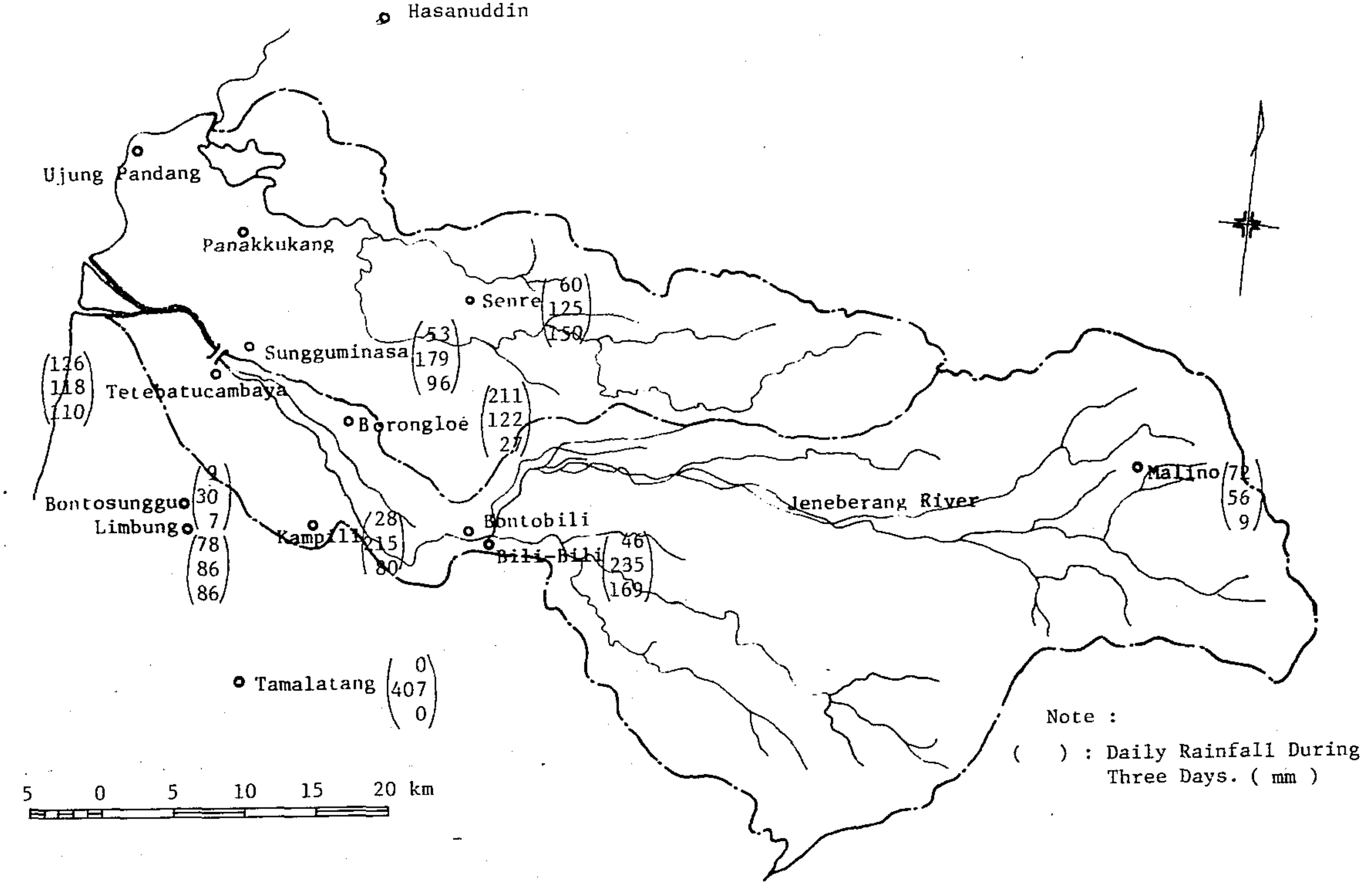
(11, 12, 13 /1/1976)



Note :
( ) : Daily Rainfall During Three Days. (mm)

Fig. 2-10(3) DAILY RAINFALL DURING MAJOR FLOODS

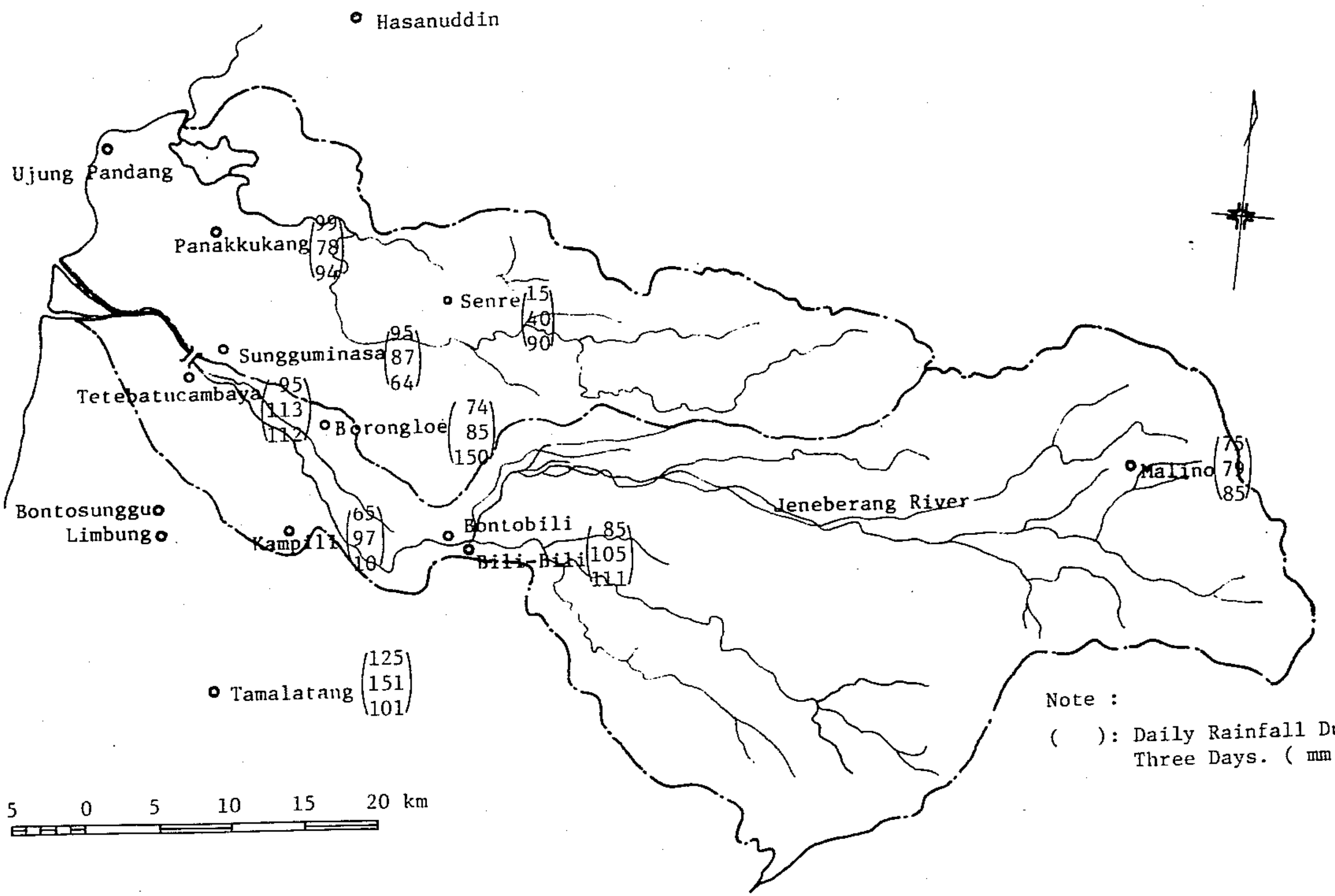
(12, 24, 25 /1/1977)



Note :
( ) : Daily Rainfall During Three Days. (mm)

Fig. 2-10(4) DAILY RAINFALL DURING MAJOR FLOODS

(7, 8, 9 /1/1978)

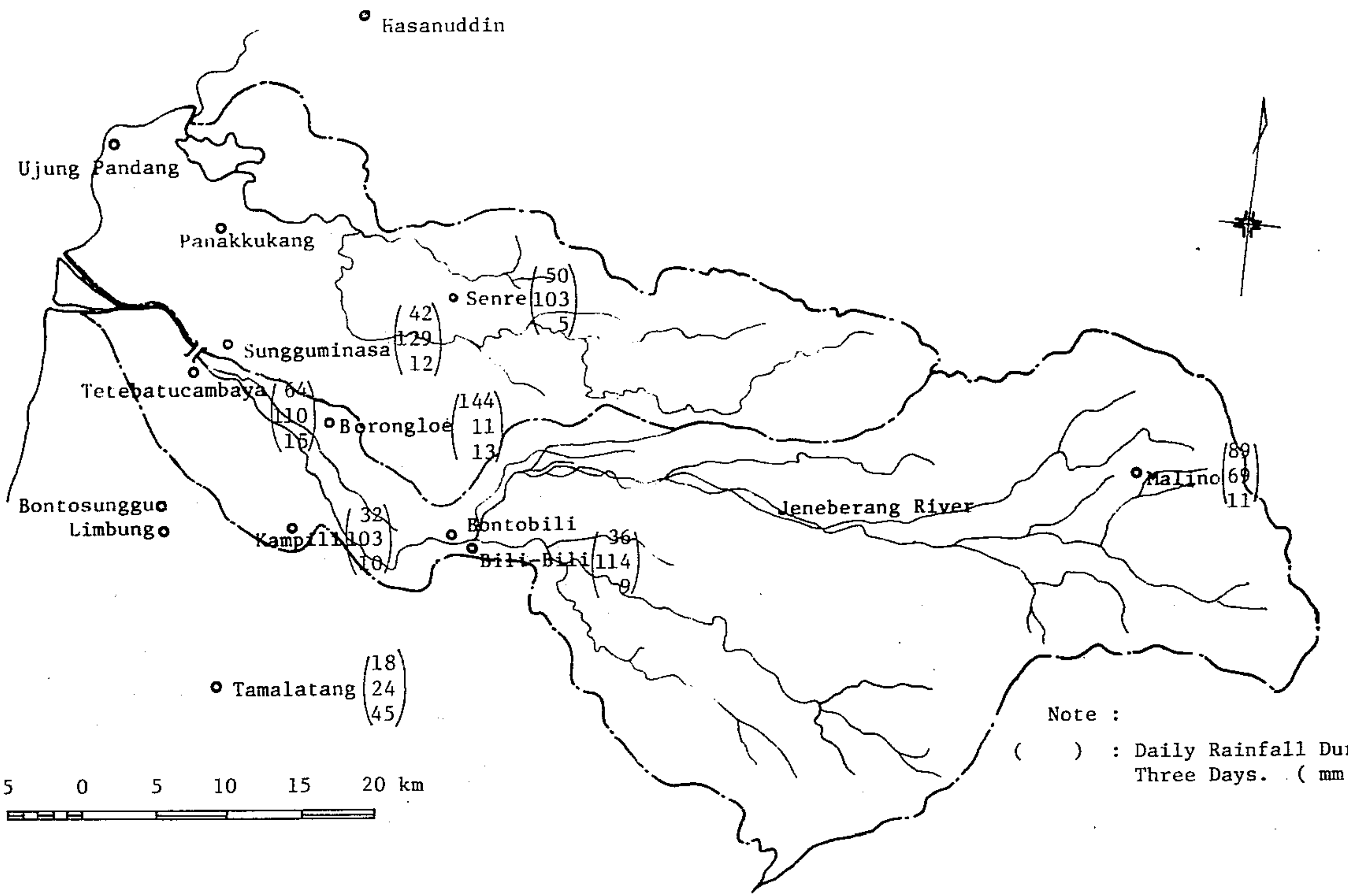


Note :

( ): Daily Rainfall During Three Days. (mm)

Fig. 2-10(5) DAILY RAINFALL DURING MAJOR FLOODS

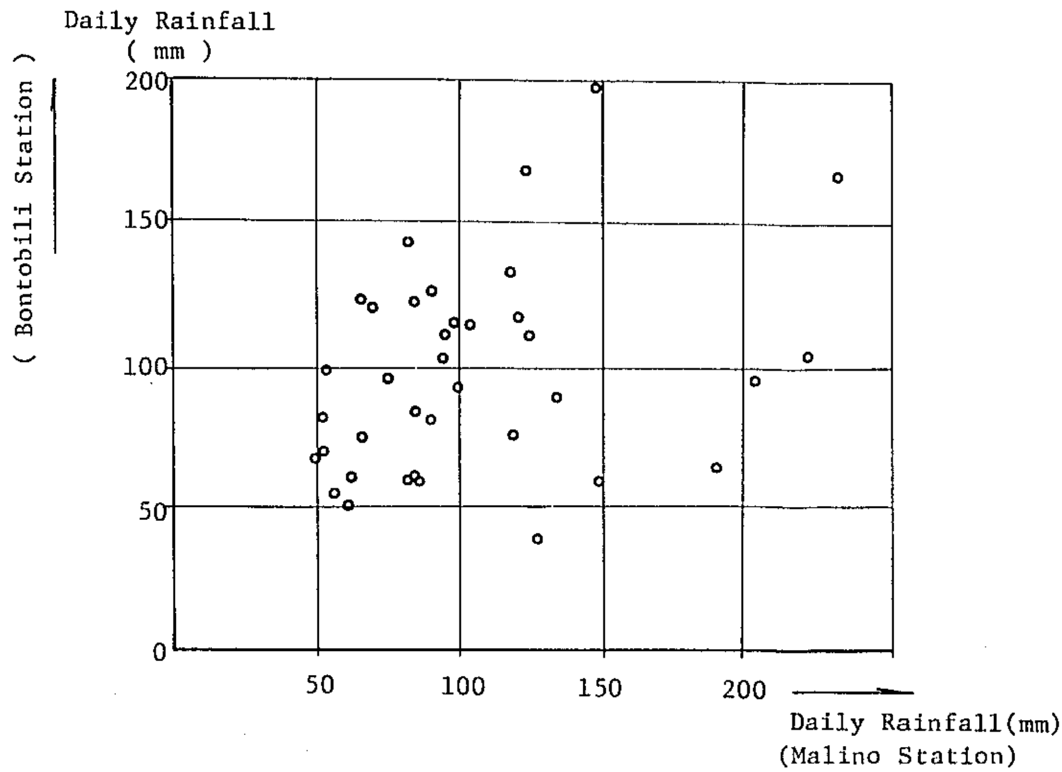
(24, 25, 26, /11/1978)



Note :

( ) : Daily Rainfall During Three Days. (mm)

Fig. 2-11(1) CORRELATION OF THE DAILY RAINFALL BY STATIONS



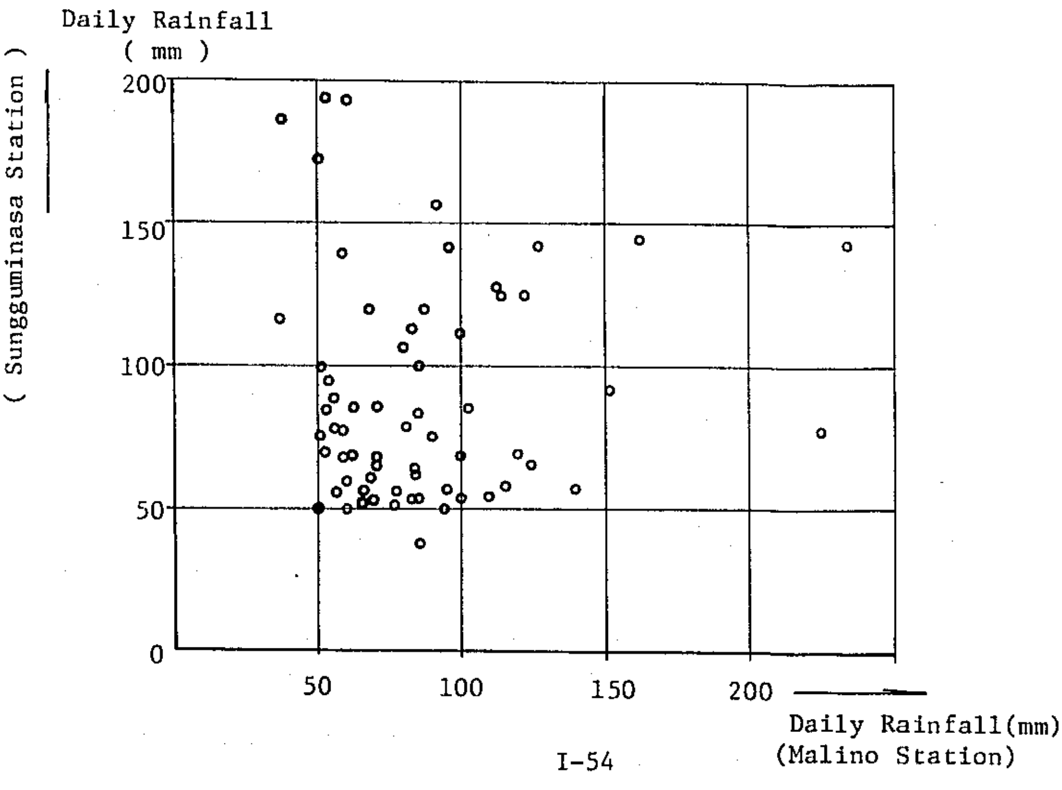
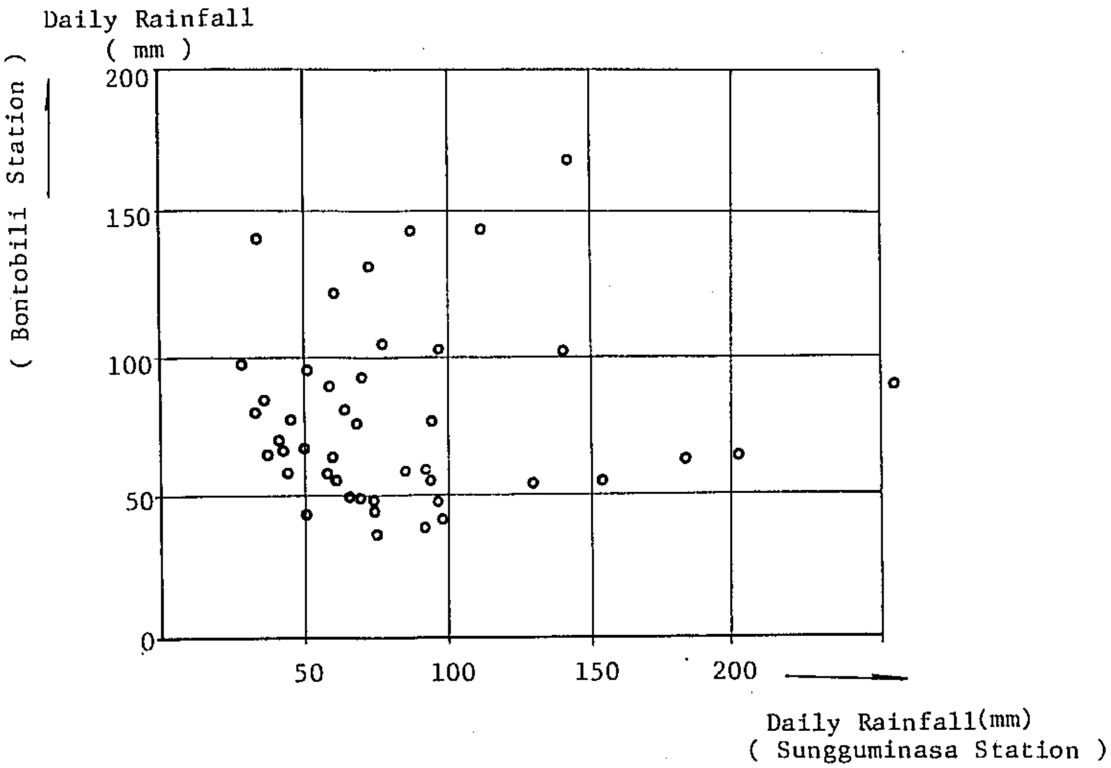


Fig. 2-11(2) CORRELATION OF THE DAILY RAINFALL BY STATIONS



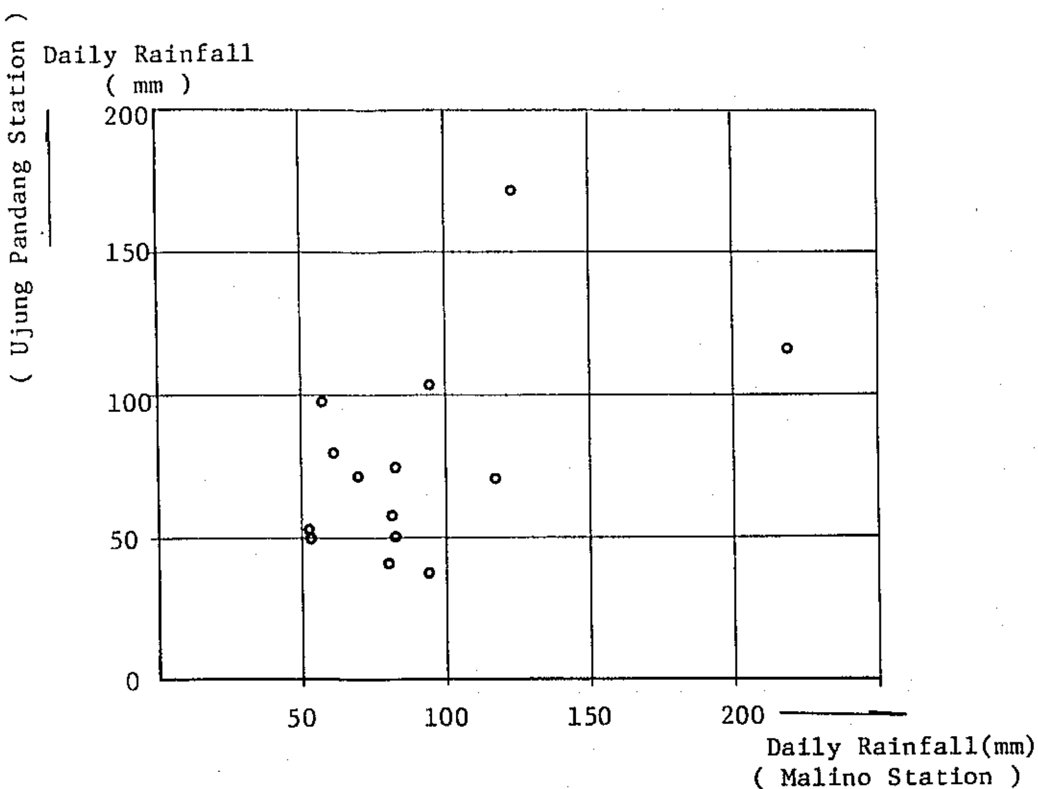
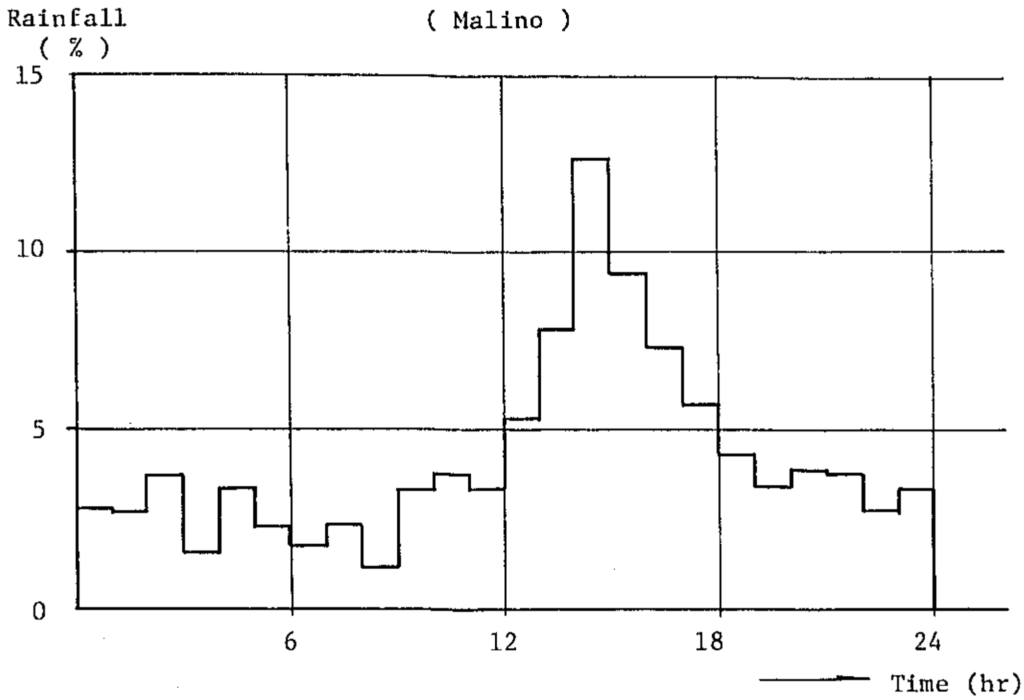


Fig. 2-12(1) HOURLY RAINFALL DISTRIBUTION



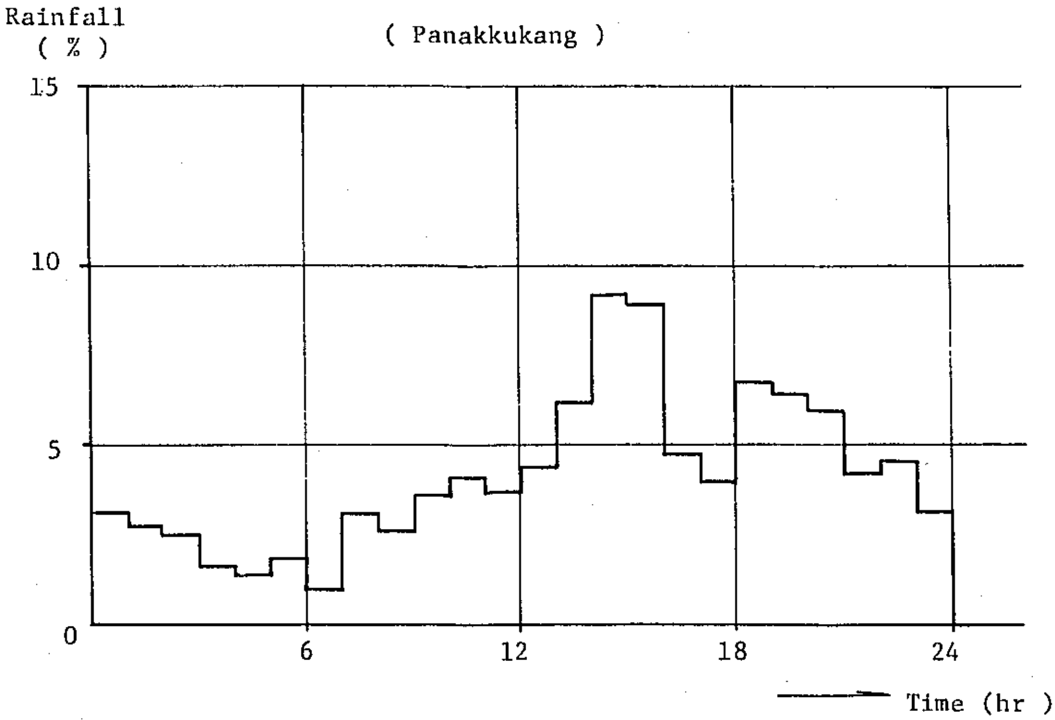


Fig. 2-12(2) HOURLY RAINFALL DISTRIBUTION

(Hasanuddin)

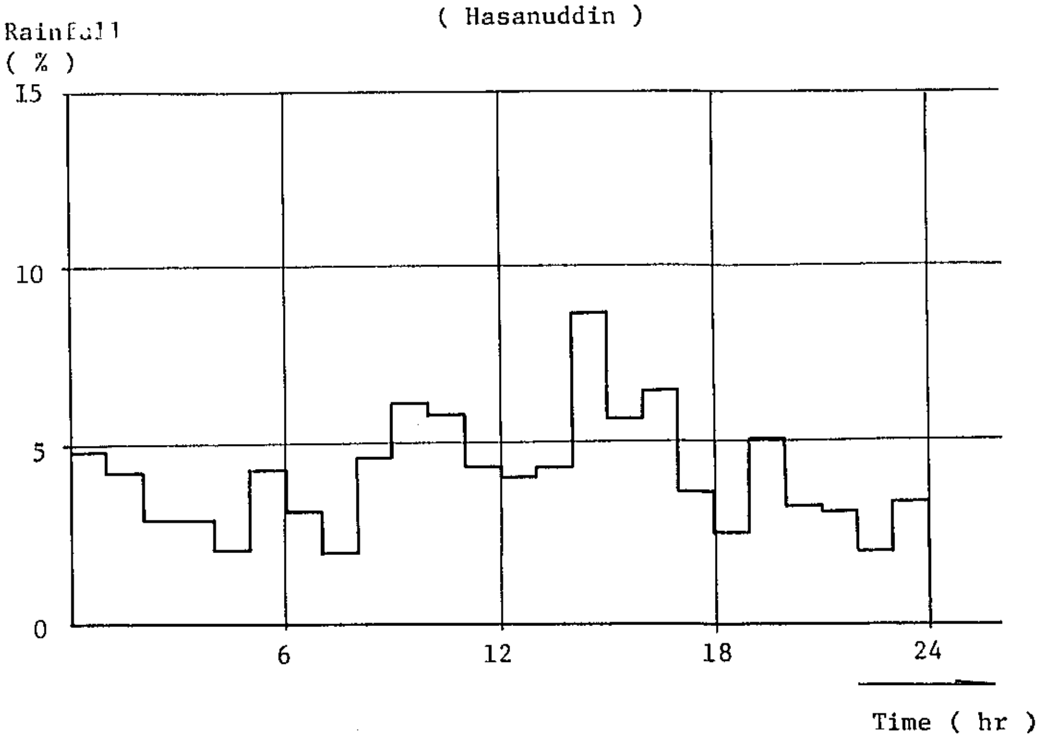
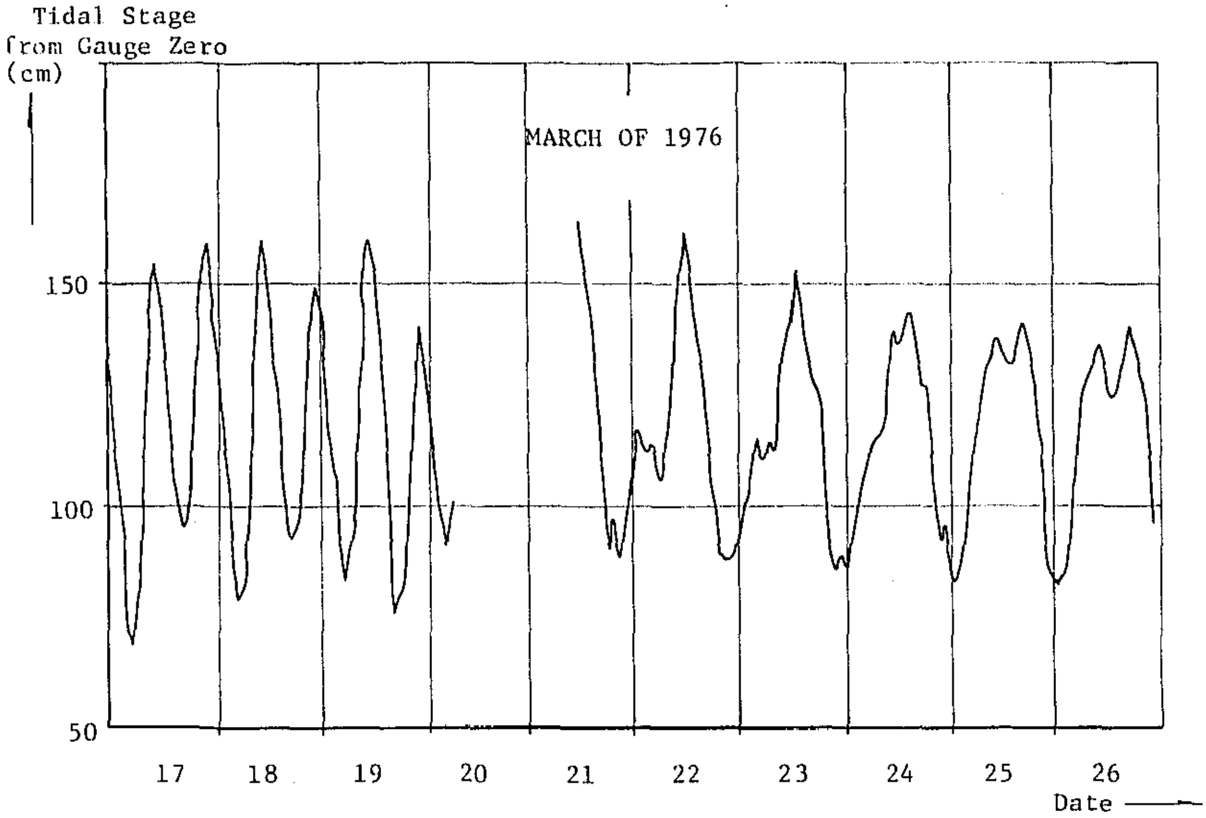


Fig. 2-13 TIDAL STAGE HYDROGRAPH

(Makassar Port)



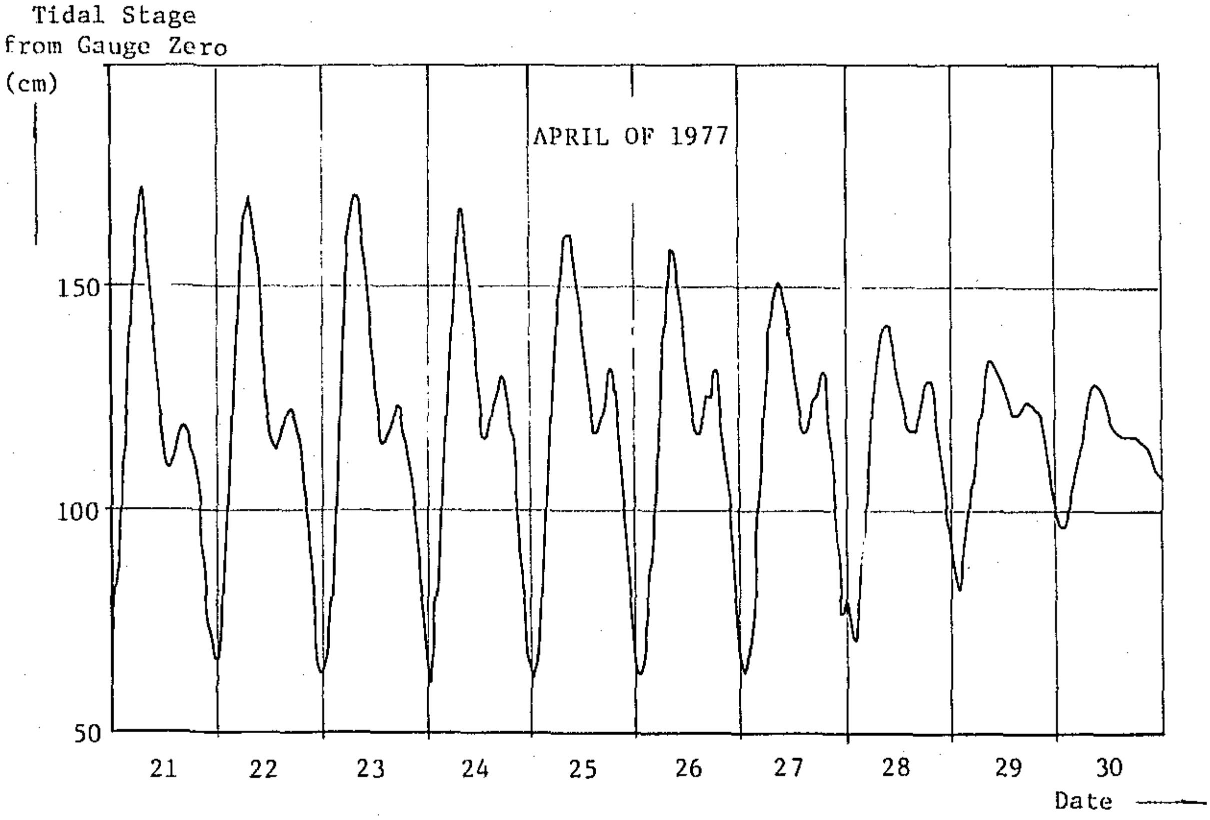


Fig. 3-1 CORRELATION BETWEEN 2-STATION AND 8-STATION AREAL MEAN RAINFALL

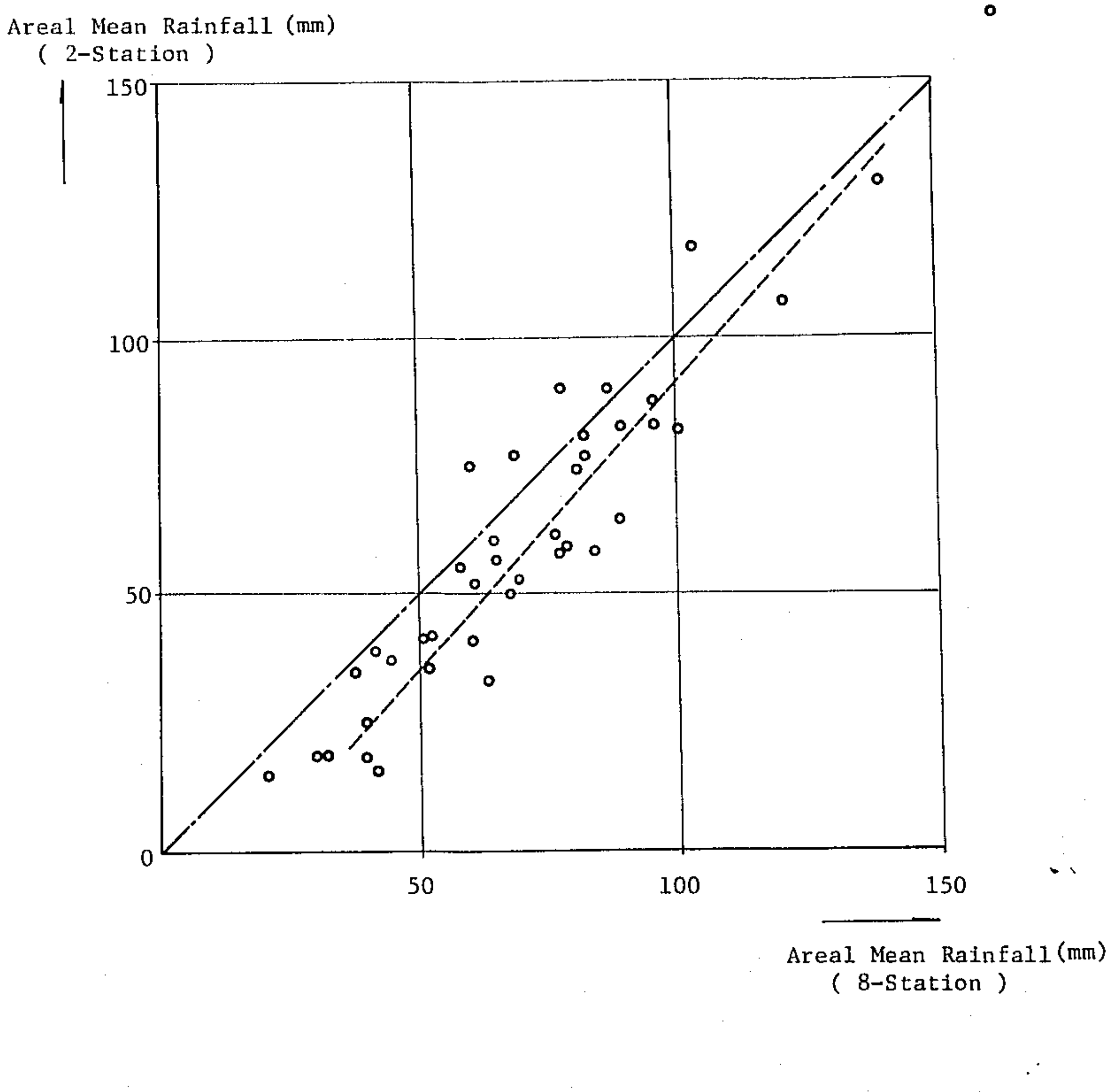


Fig. 3-2(1) PROBABLE AREAL MEAN DAILY RAINFALL

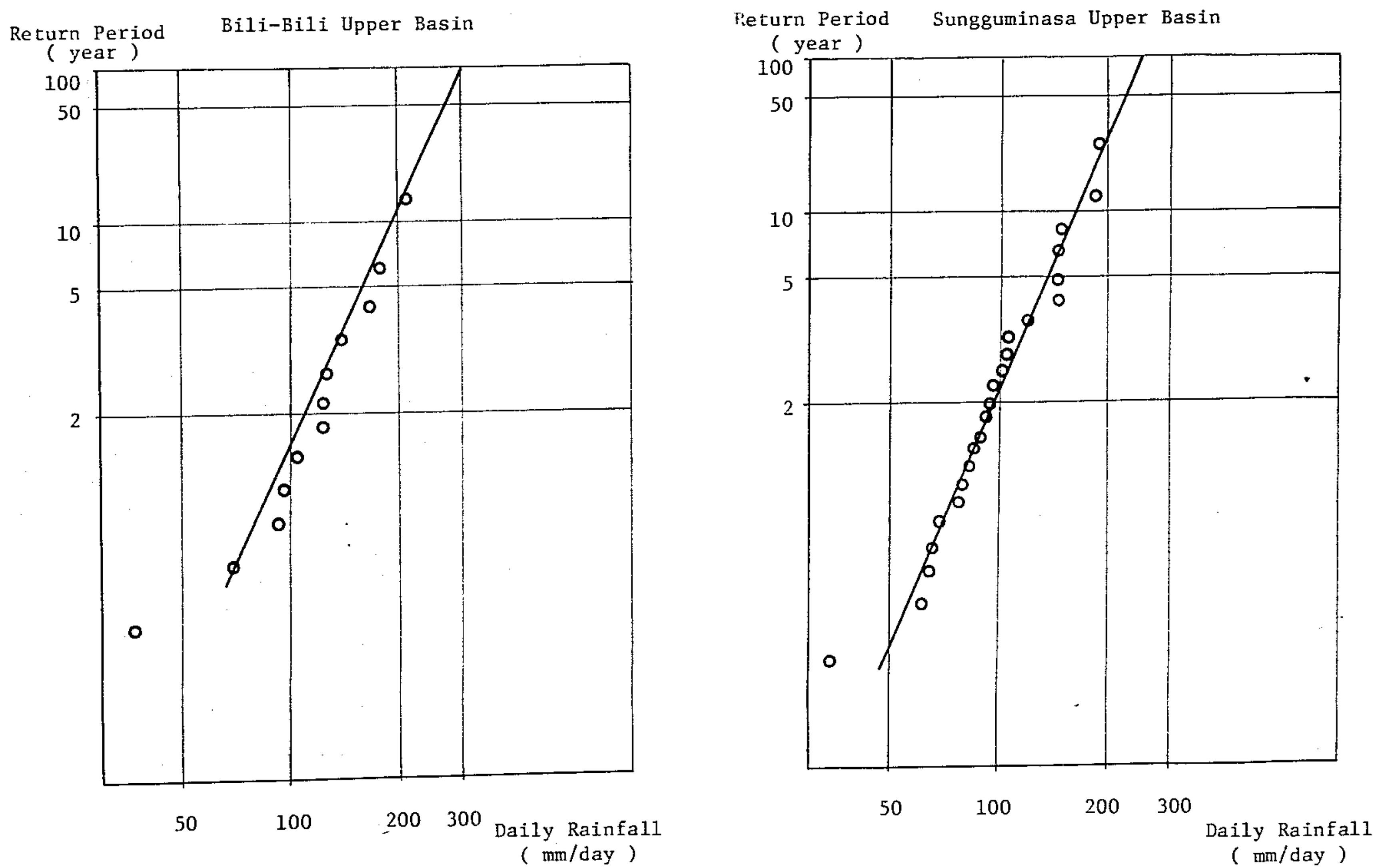
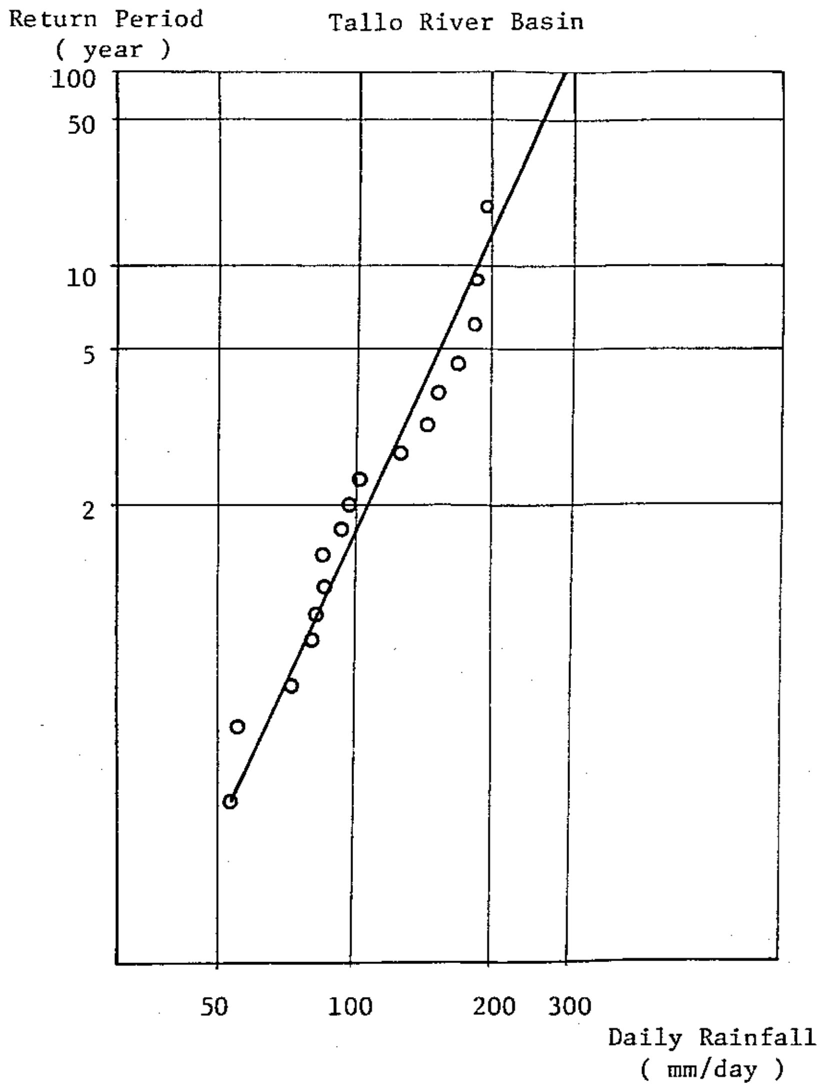


Fig. 3-2(2) PROBAPLE AREAL MEAN DAILY RAINFALL



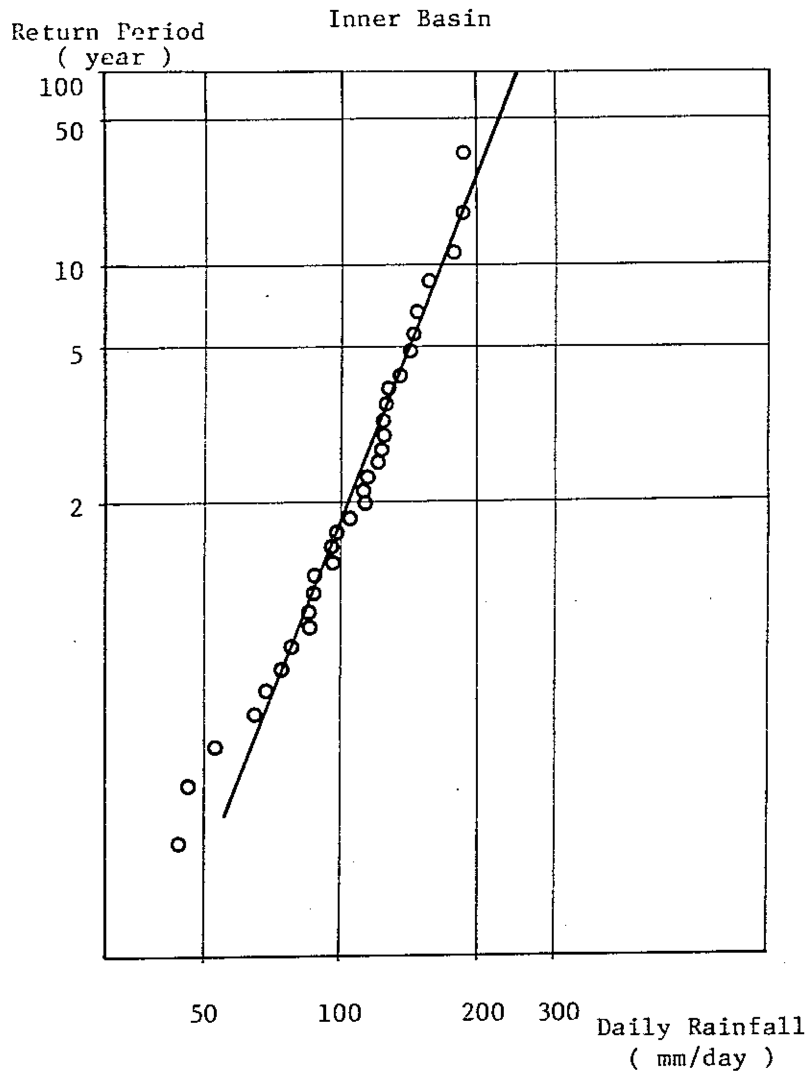
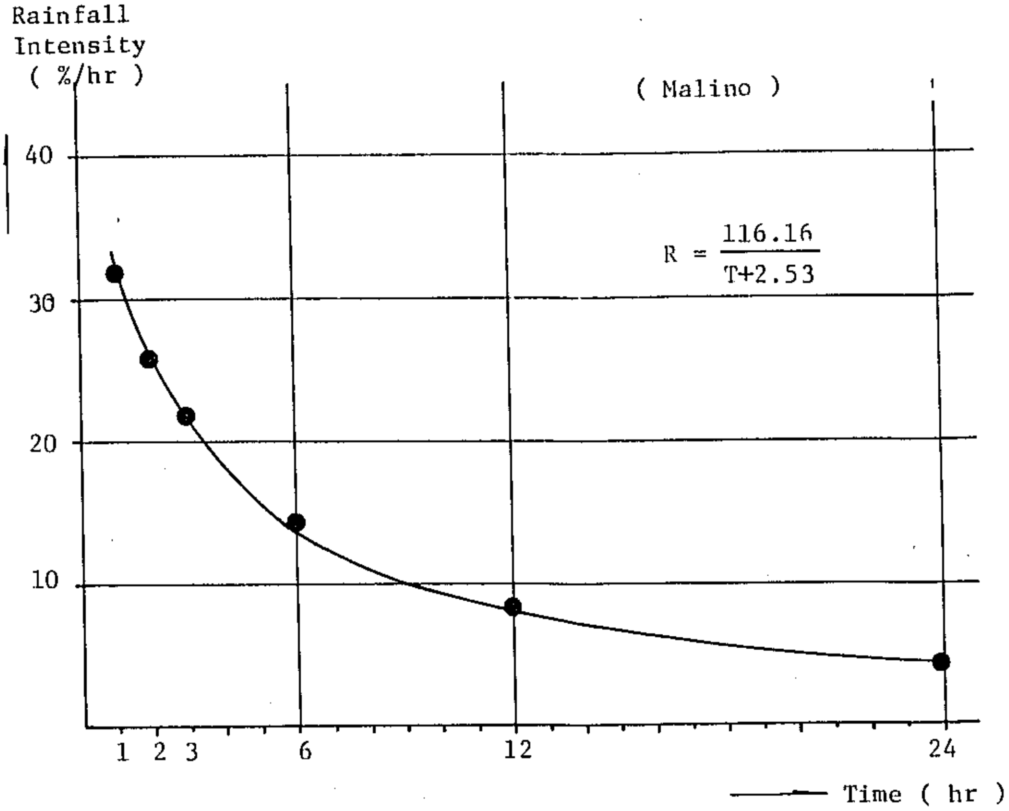


Fig. 3-3(1) RAINFALL INTENSITY CURVE



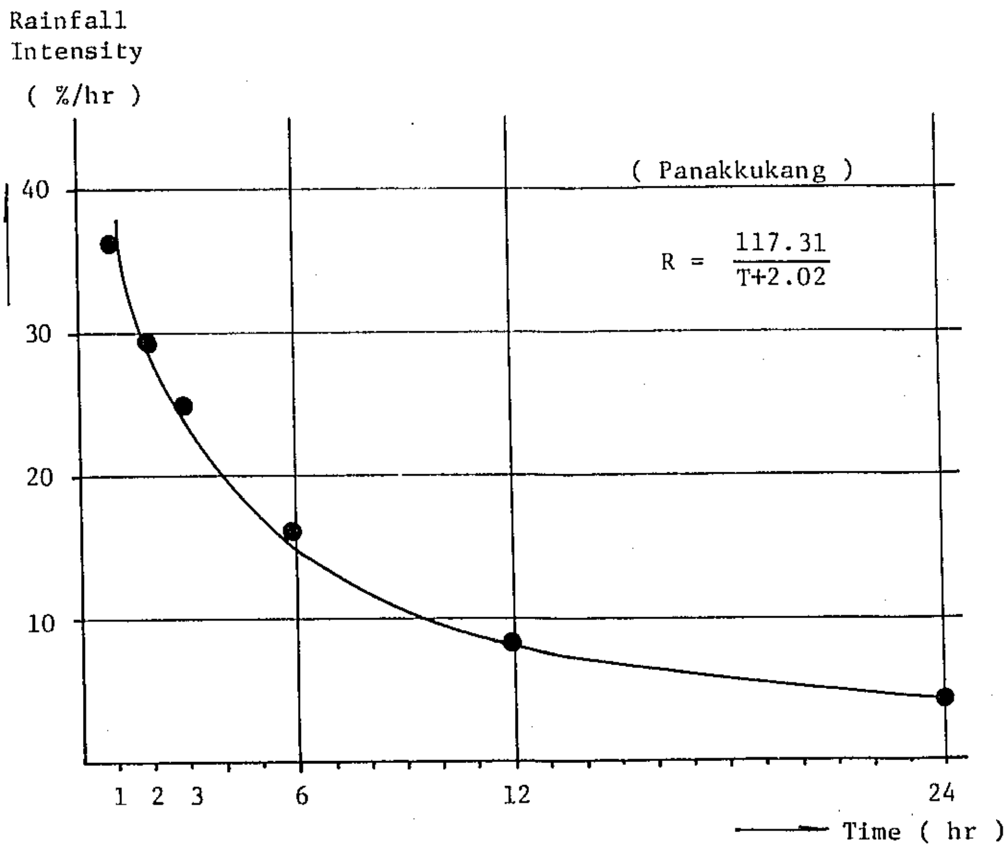


Fig. 3-3(2) RAINFALL INTENSITY CURVE

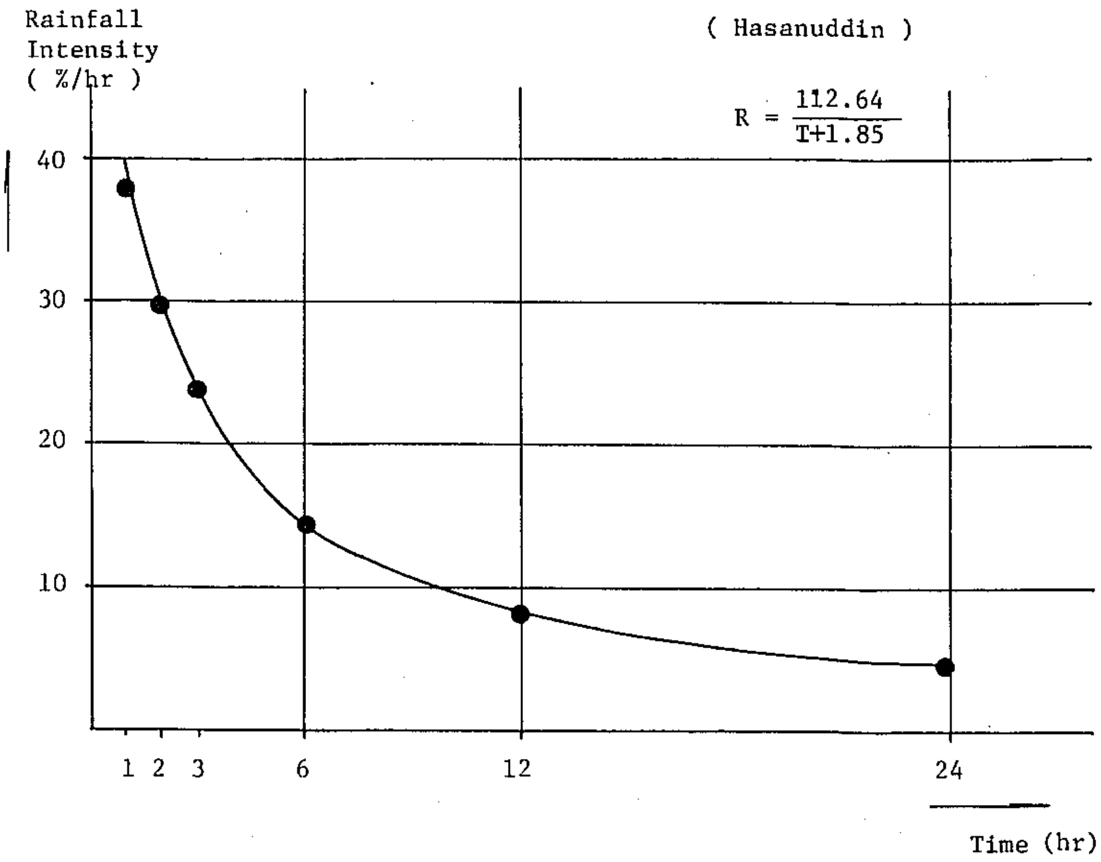
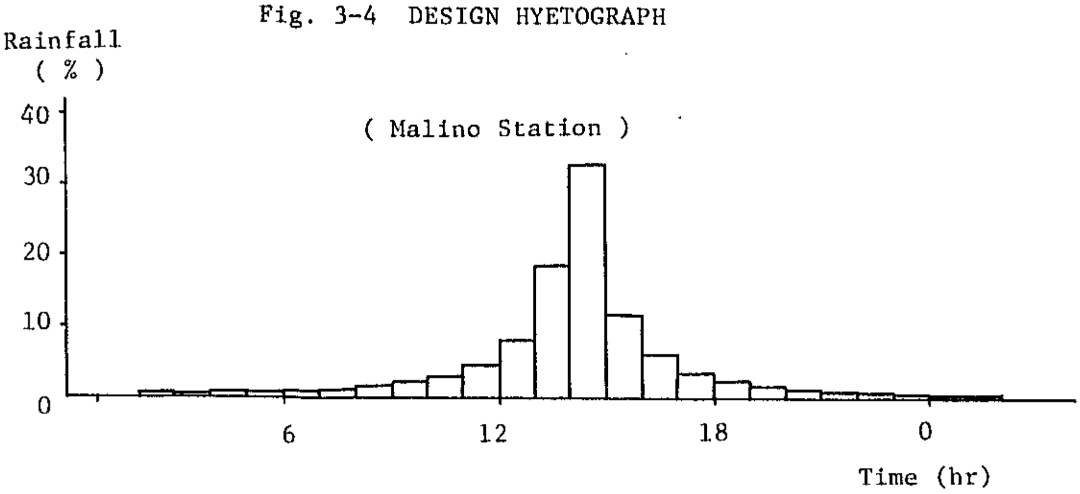
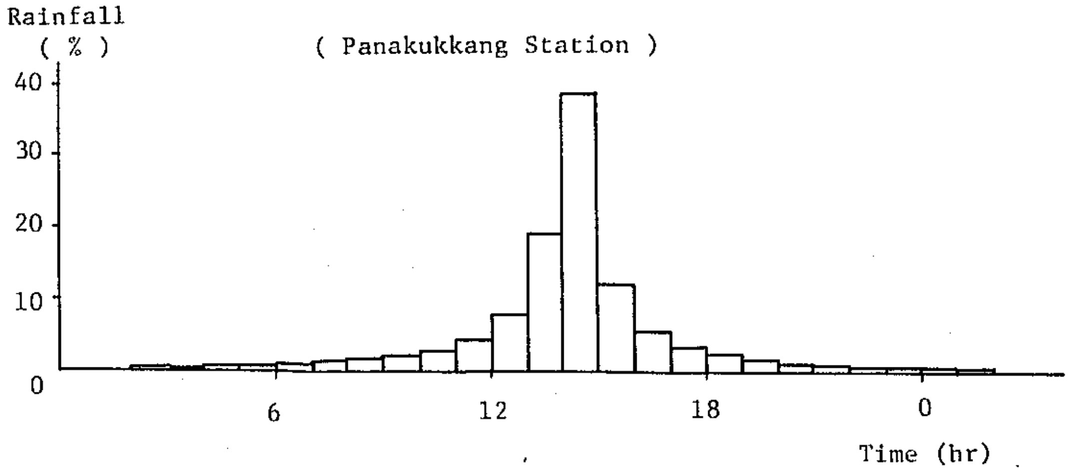


Fig. 3-4 DESIGN HYETOGRAPH





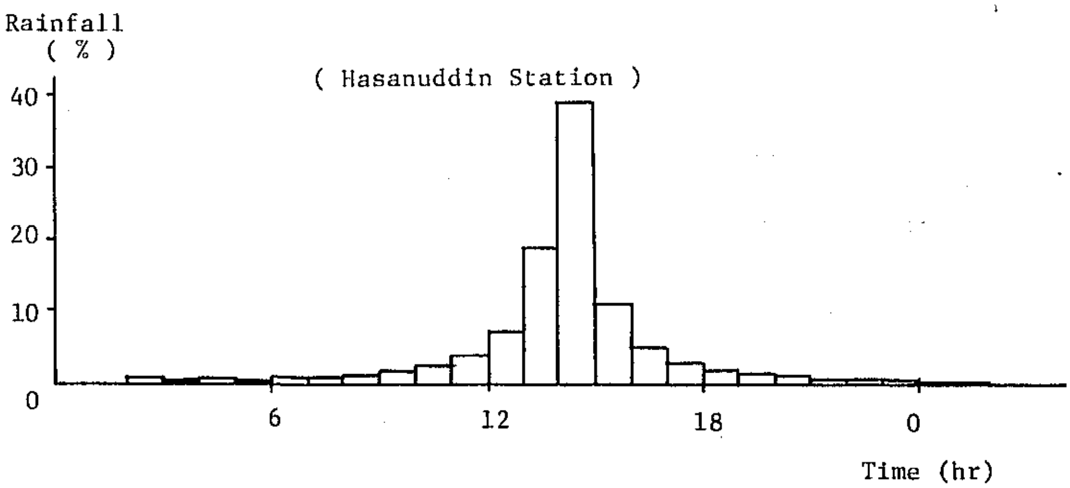


Fig. 3-5 DAILY RAINFALL DISTRIBUTION PER FLOOD

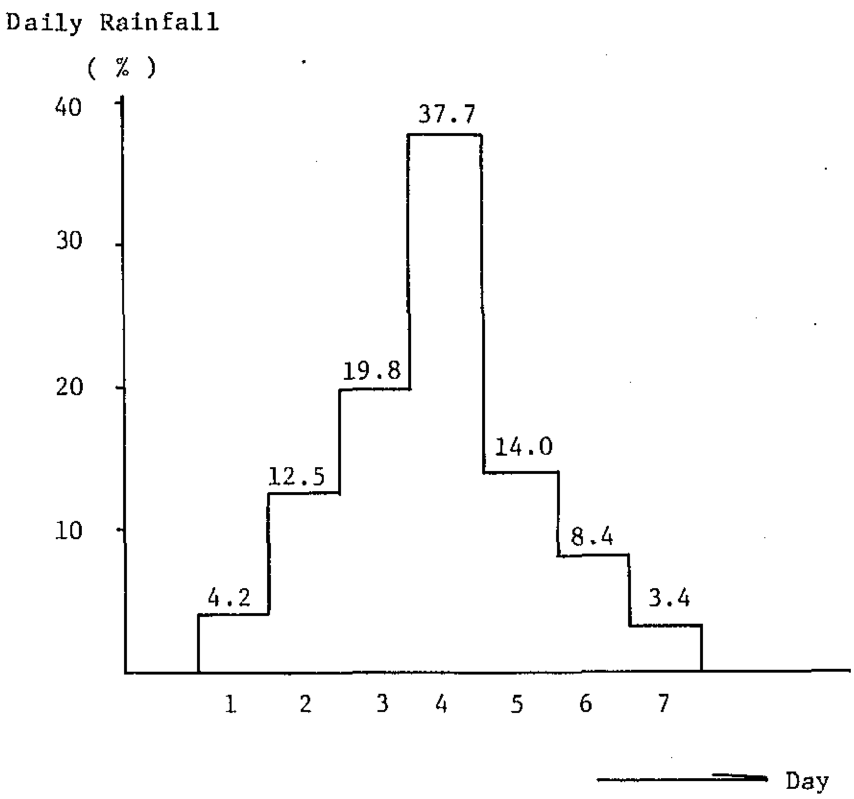


Fig. 4-1 RIVER BED ELEVATION ALONG THE JENEBERANG RIVER COURSE

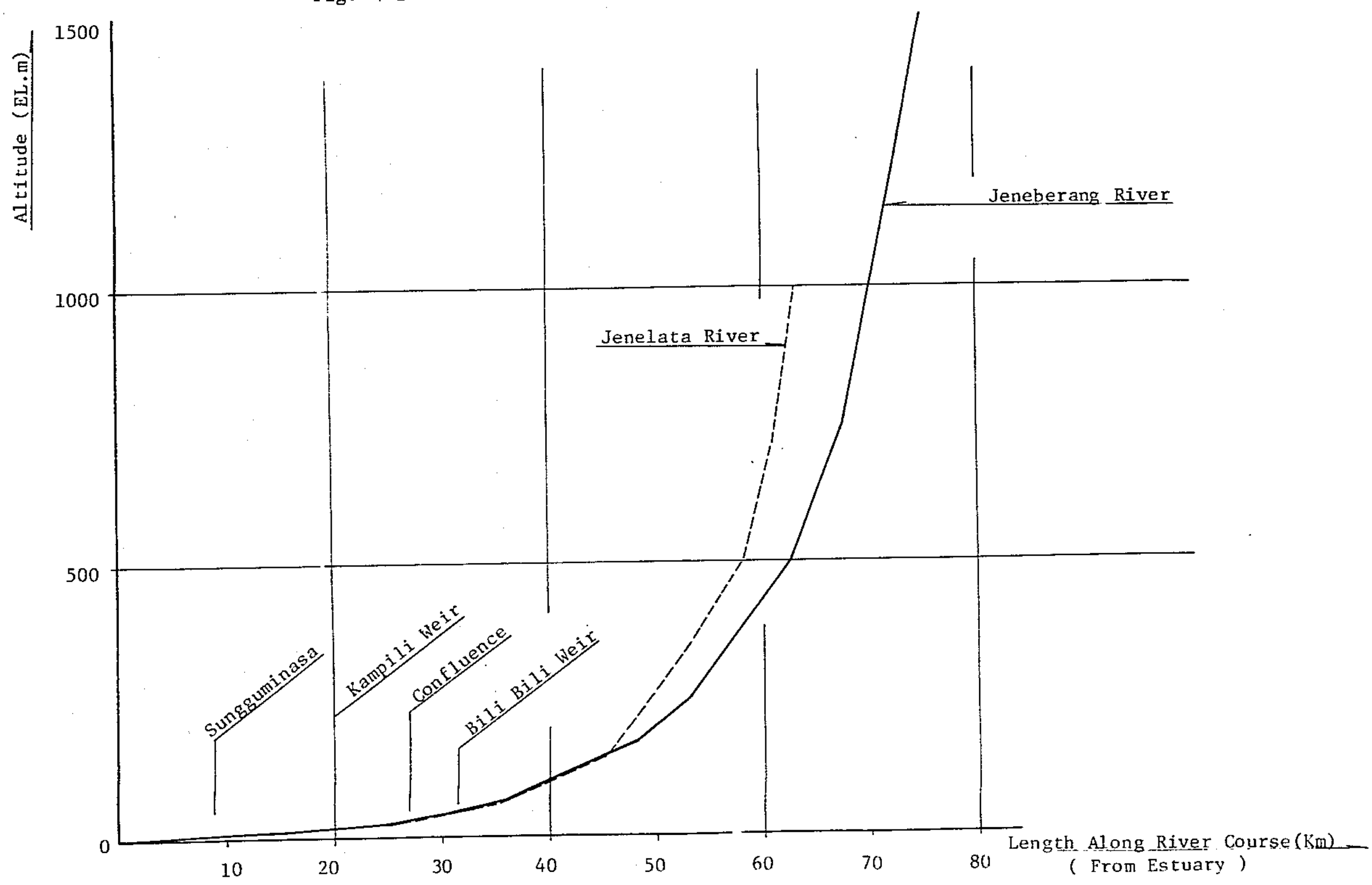


Fig. 4-2(1) WATER STAGE RECORDS AT BILI-BILI AND HOURLY RAINFALL AT MALINO STATION

(Jan. 1977)

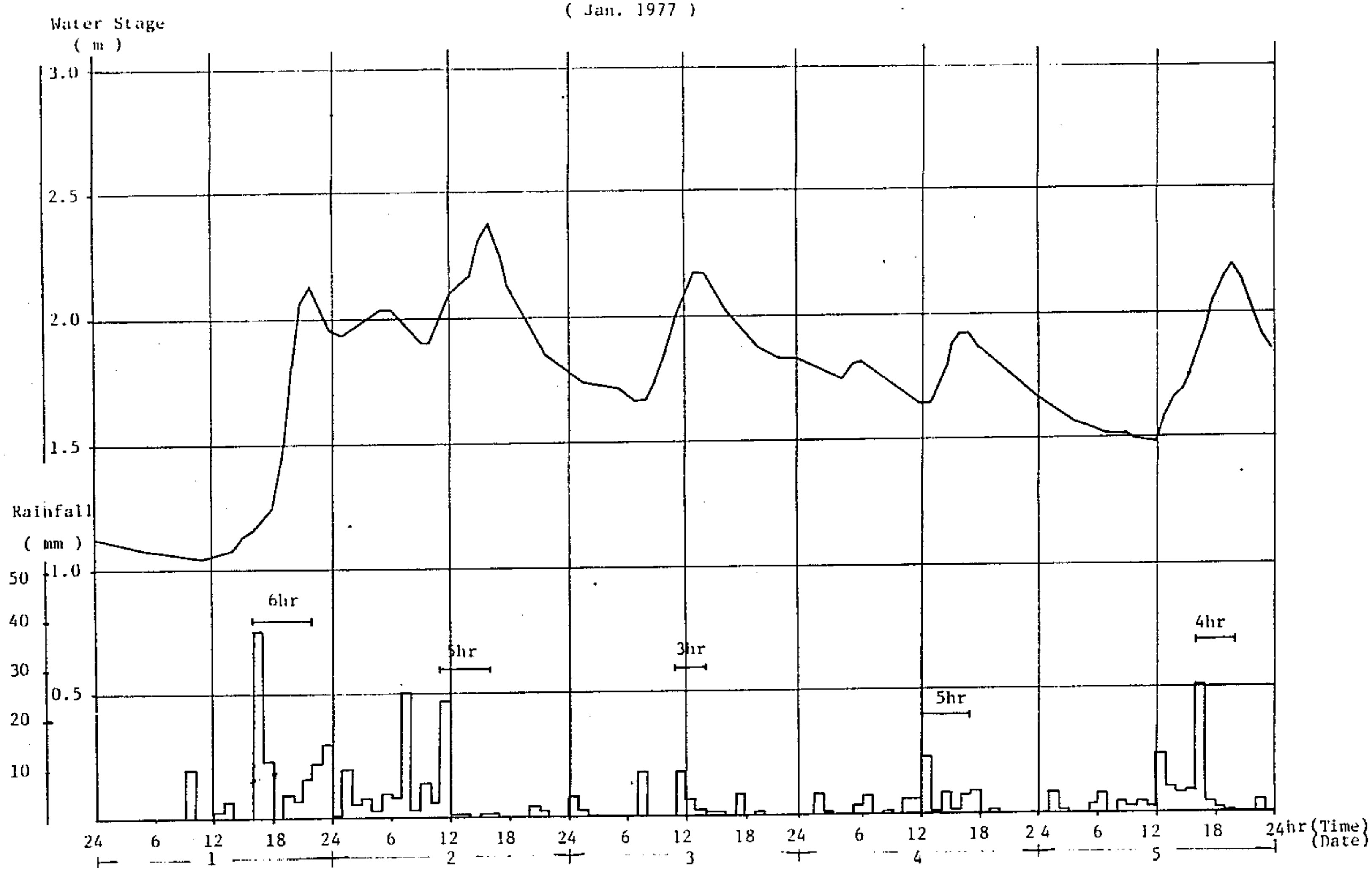


Fig. 4-2(2) WATER STAGE RECORDS AT BILI-BILI AND HOURLY RAINFALL AT MALINO STATION
(Feb. 1977)

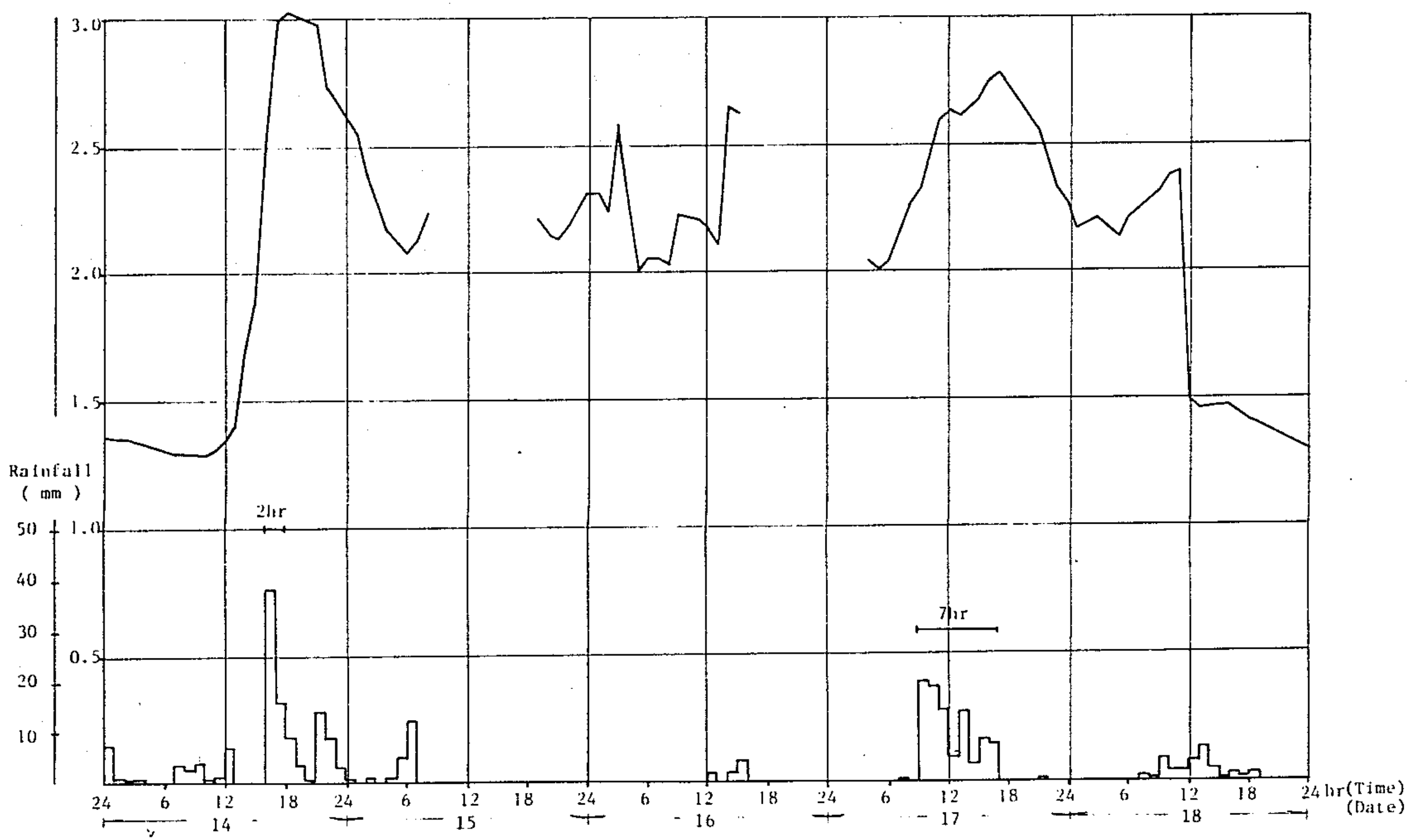


Fig. 4-2(3) WATER STAGE RECORDS AT BILI-BILI AND HOURLY RAINFALL AT MALINO STATION

(Jan. 1978)

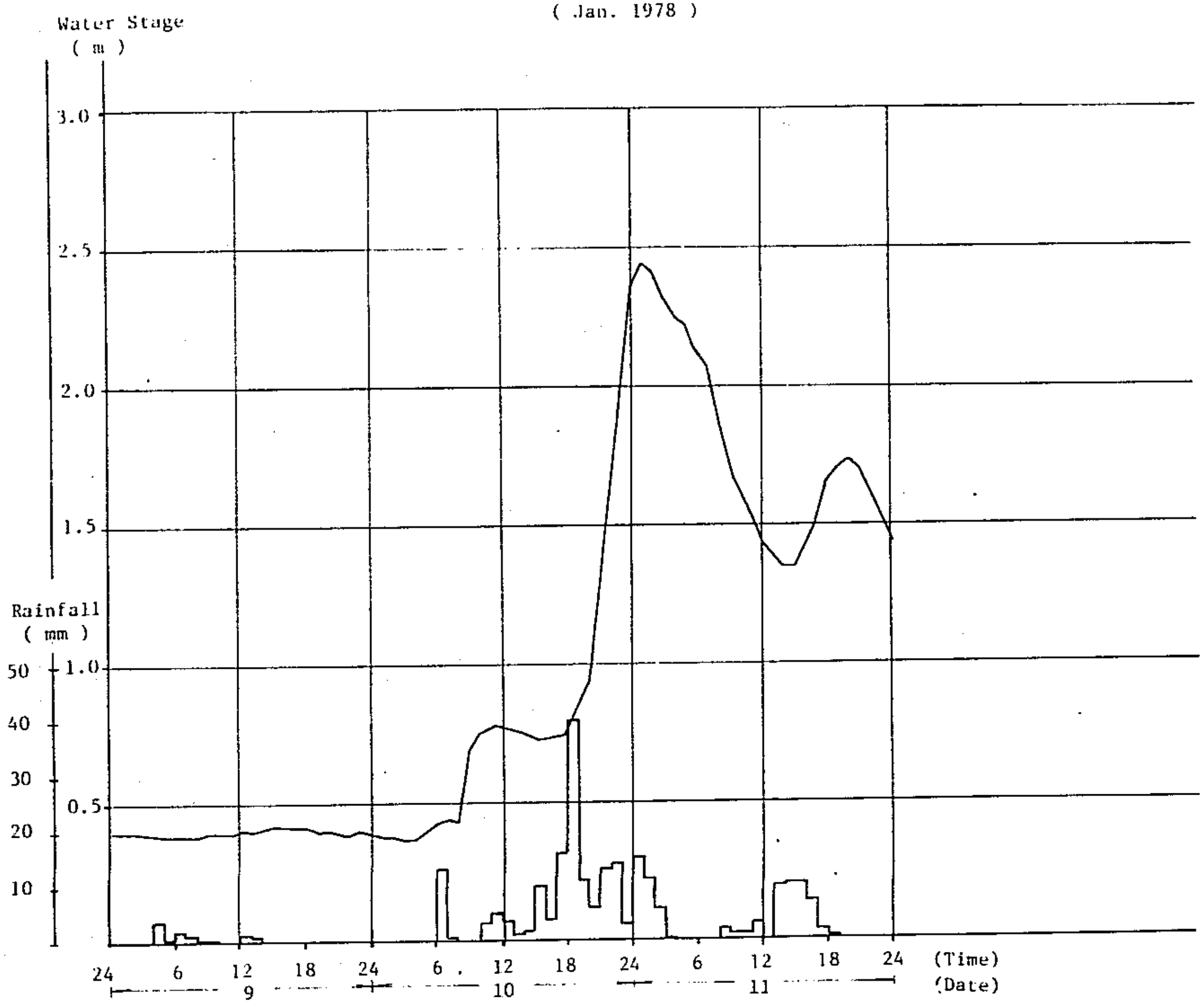


Fig. 4-3 ROUGHNESS COEFFICIENT V.S. RIVER BED GRADIENT

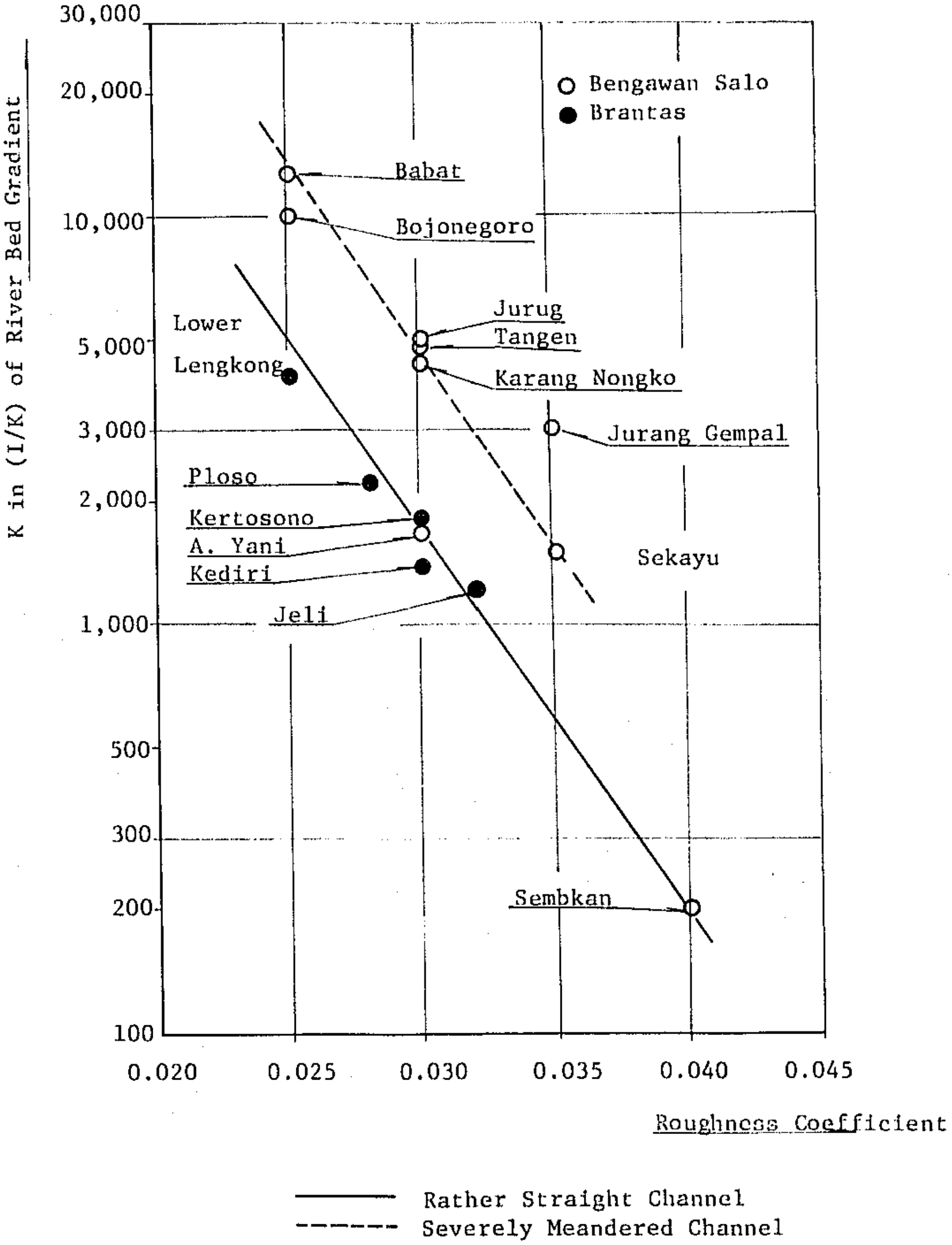


Fig. 4-4 SCHEMATIC DIAGRAM OF RIVER SYSTEMS

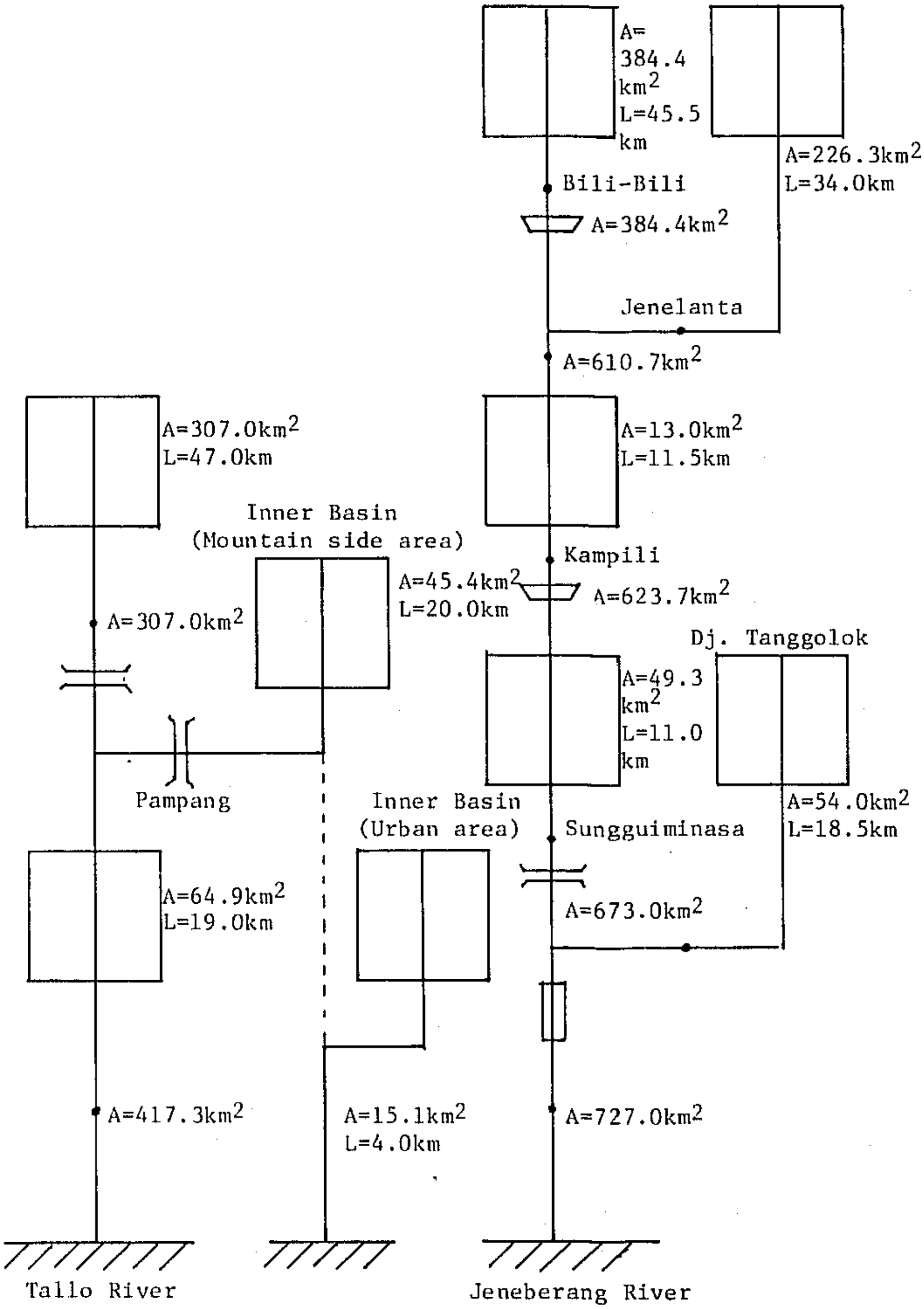
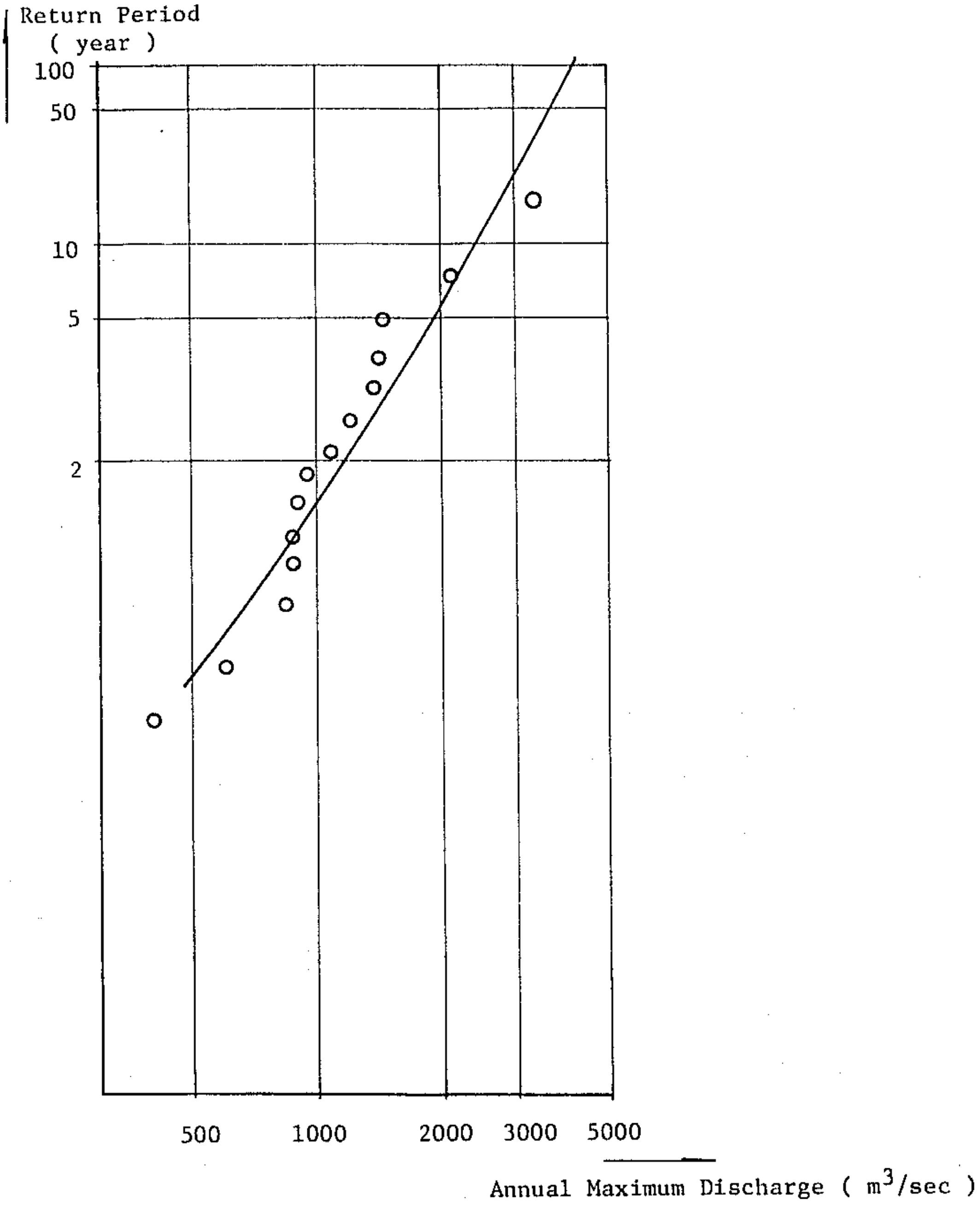


Fig. 4-5 PROBABLE DISCHARGE AT KAMPILI WEIR*



* : Obtained by Gumbel Method and by using discharge data

Fig. 4-6 RATING CURVE AT BILI-BILI

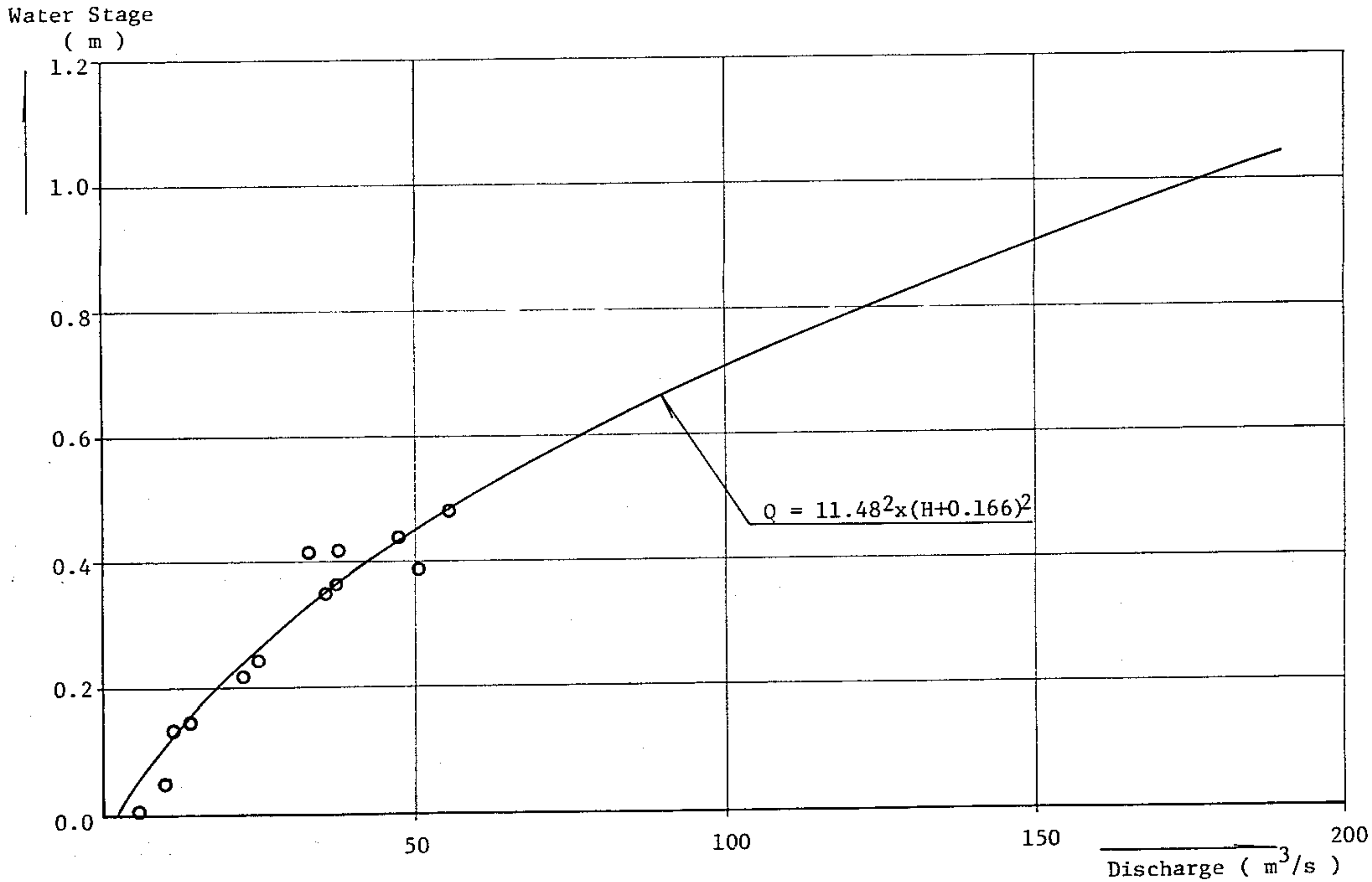


Fig. 4-7 SPECIFIC DISCHARGE OF RIVERS IN INDONESIA

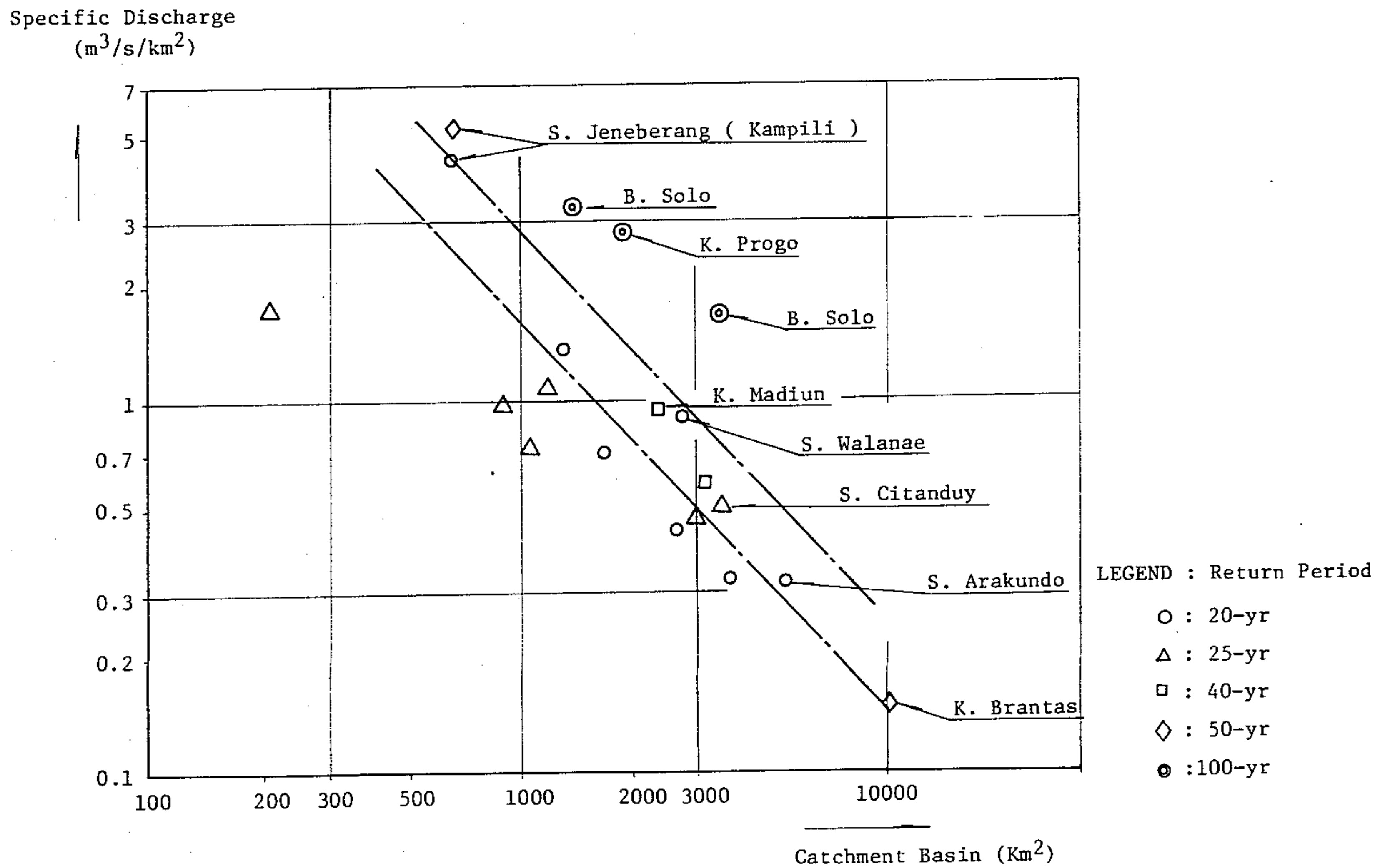
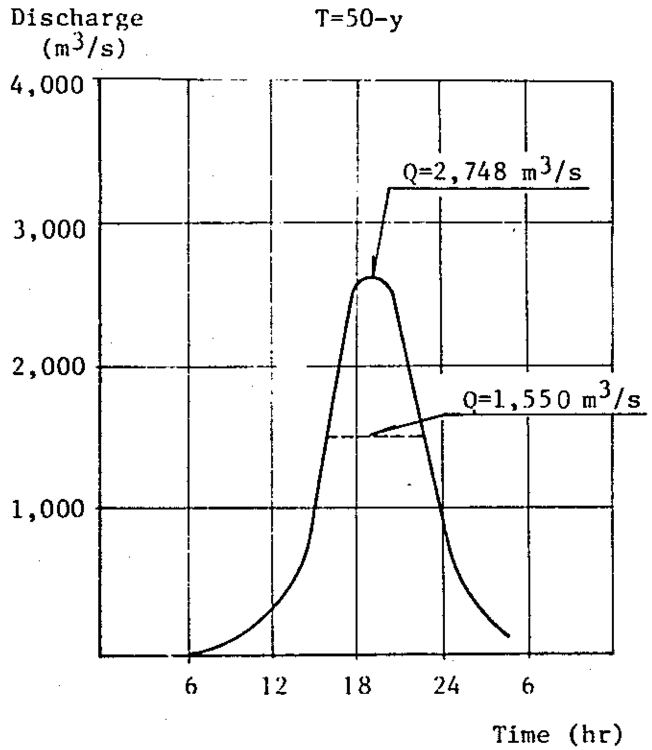
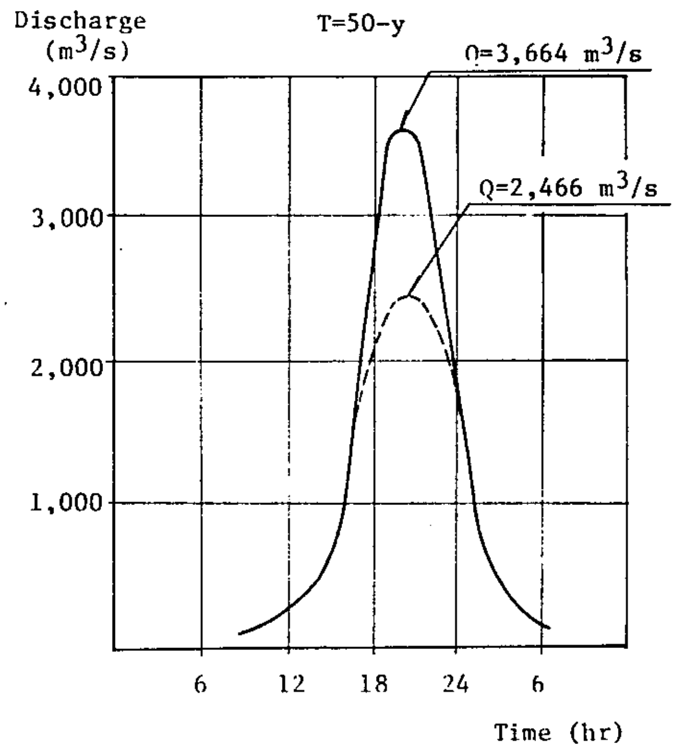


Fig. 4-8 STANDARD PROJECT AND DESIGN FLOOD HYDROGRAPH

Bili-Bili



Kampili



——— Standard Project Flood

----- Design Flood

Fig. 4-9 FLOOD CONTROL STORAGE CAPACITY

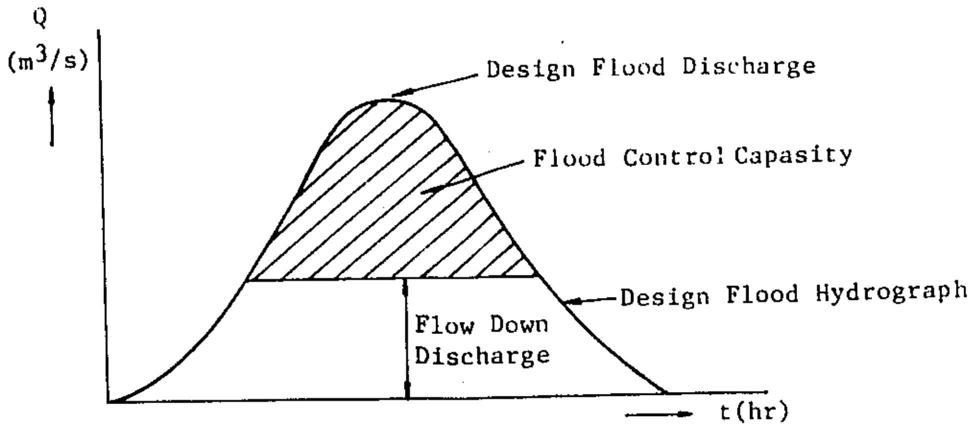
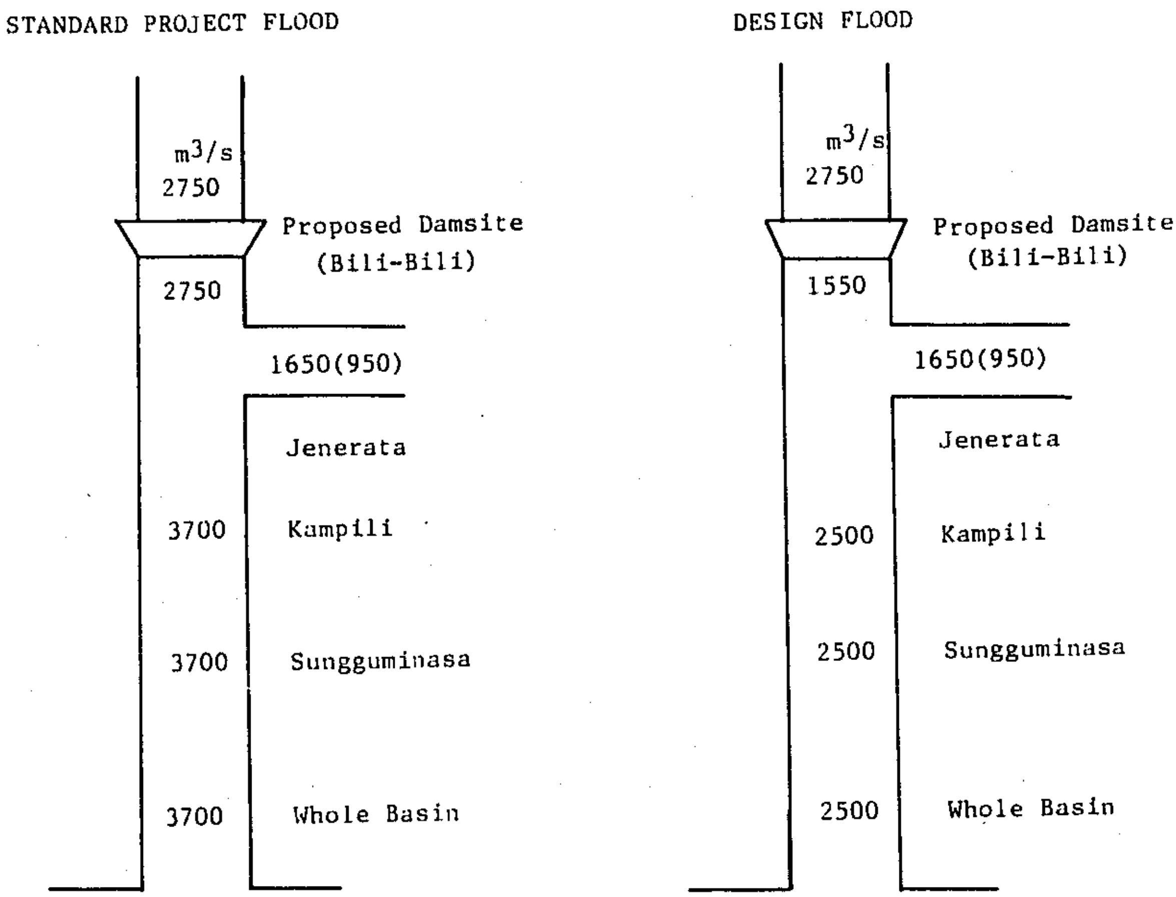


Fig. 4-10 STANDARD PROJECT AND DESIGN FLOOD DISTRIBUTION



Note ; Figures in parentheses represent
discharge joining the main stream

Fig. 4-11 NATURAL BUFFER

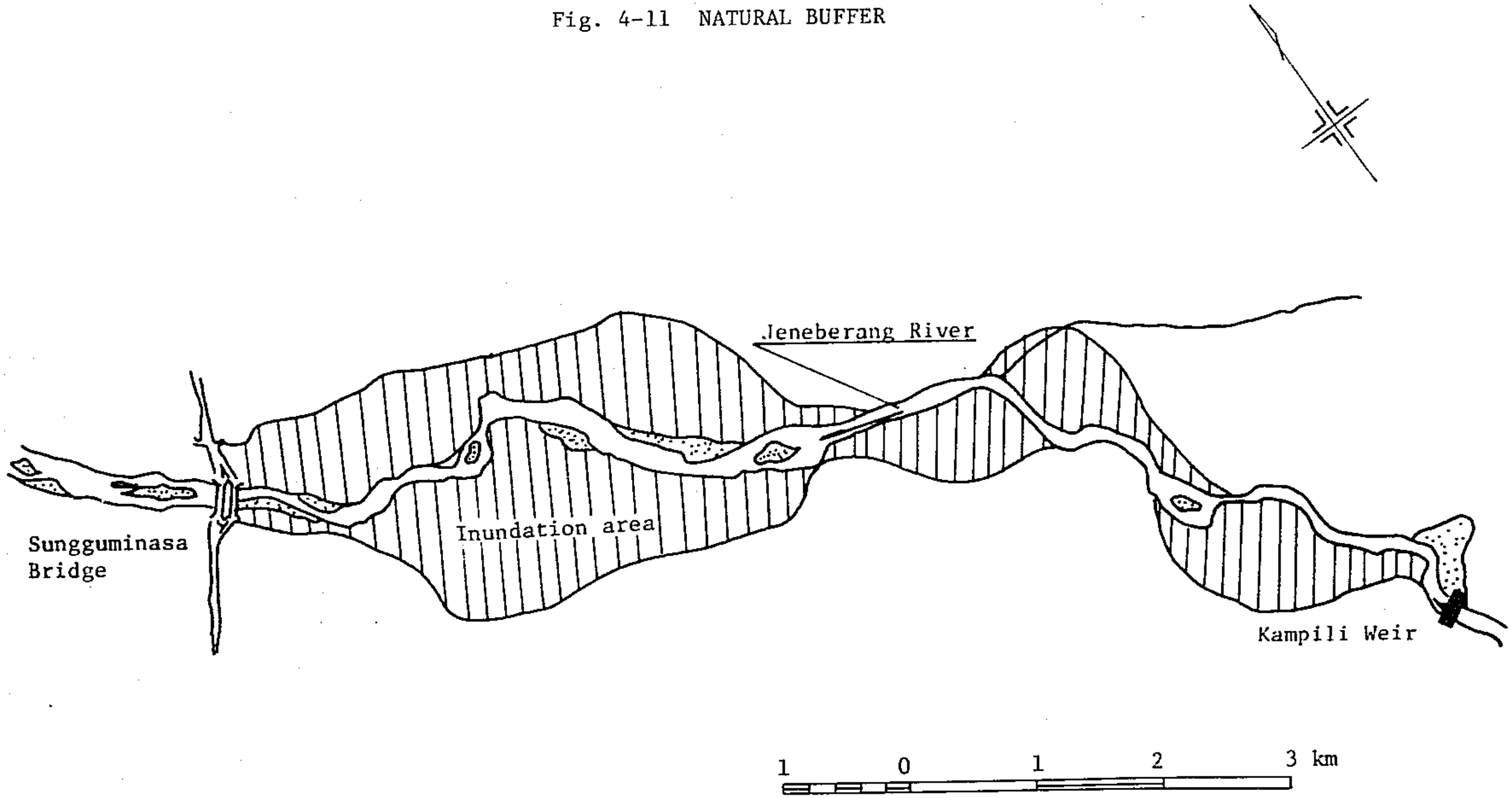


Fig. 4-12 SIMULATION MODEL

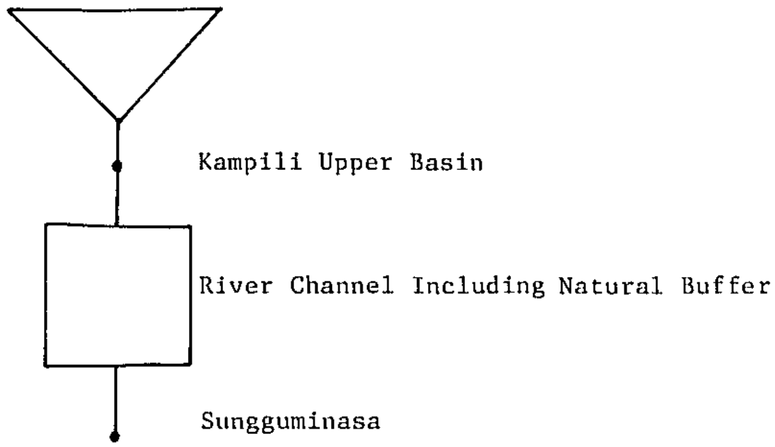


Fig. 4-13 STORAGE AND DISCHARGE RELATION

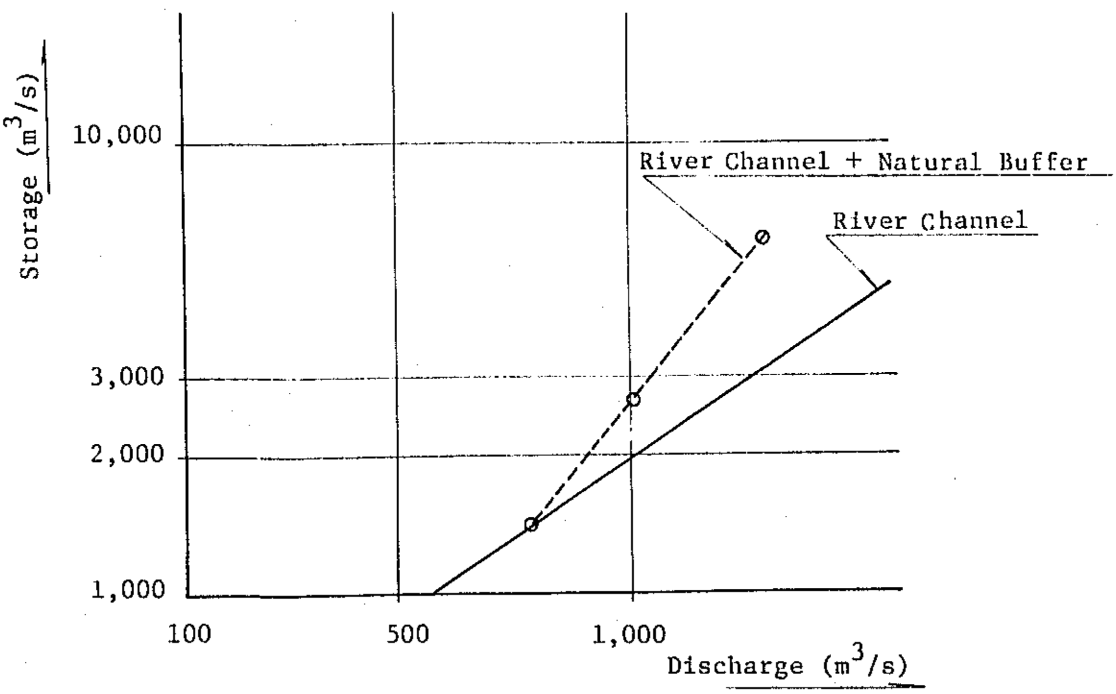


Fig. 4-14 REGULATION EFFECTS OF NATURAL BUFFER

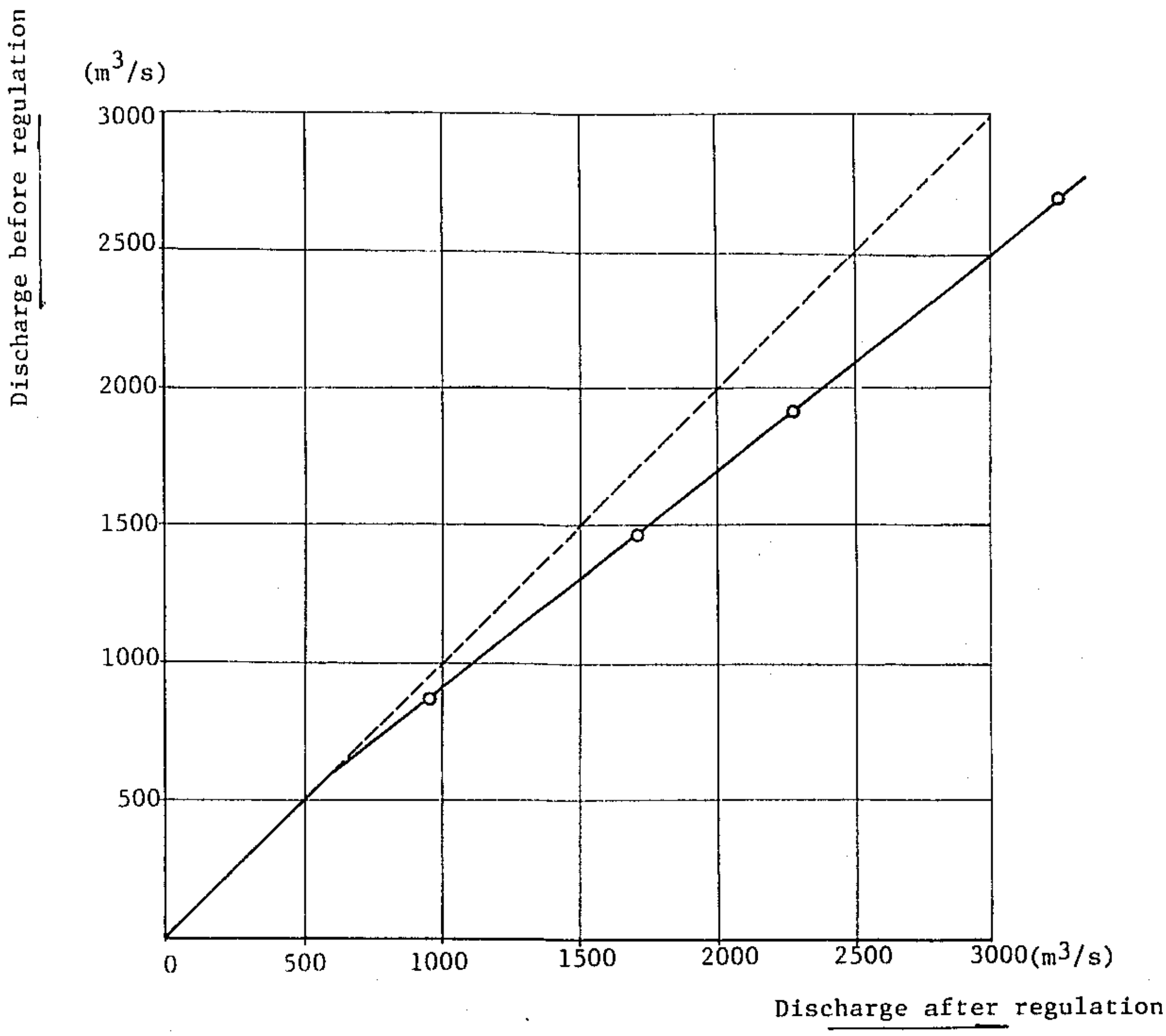
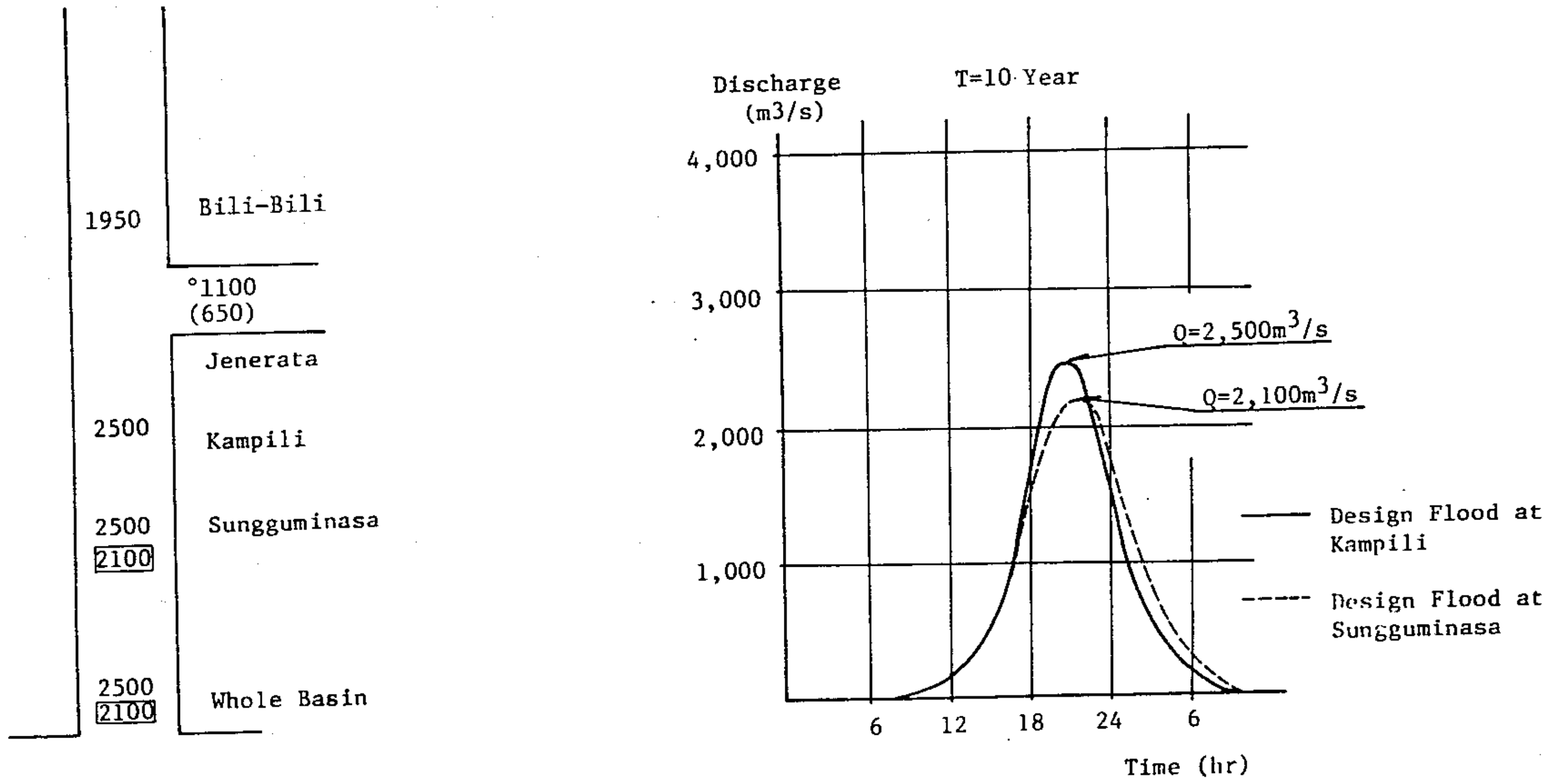


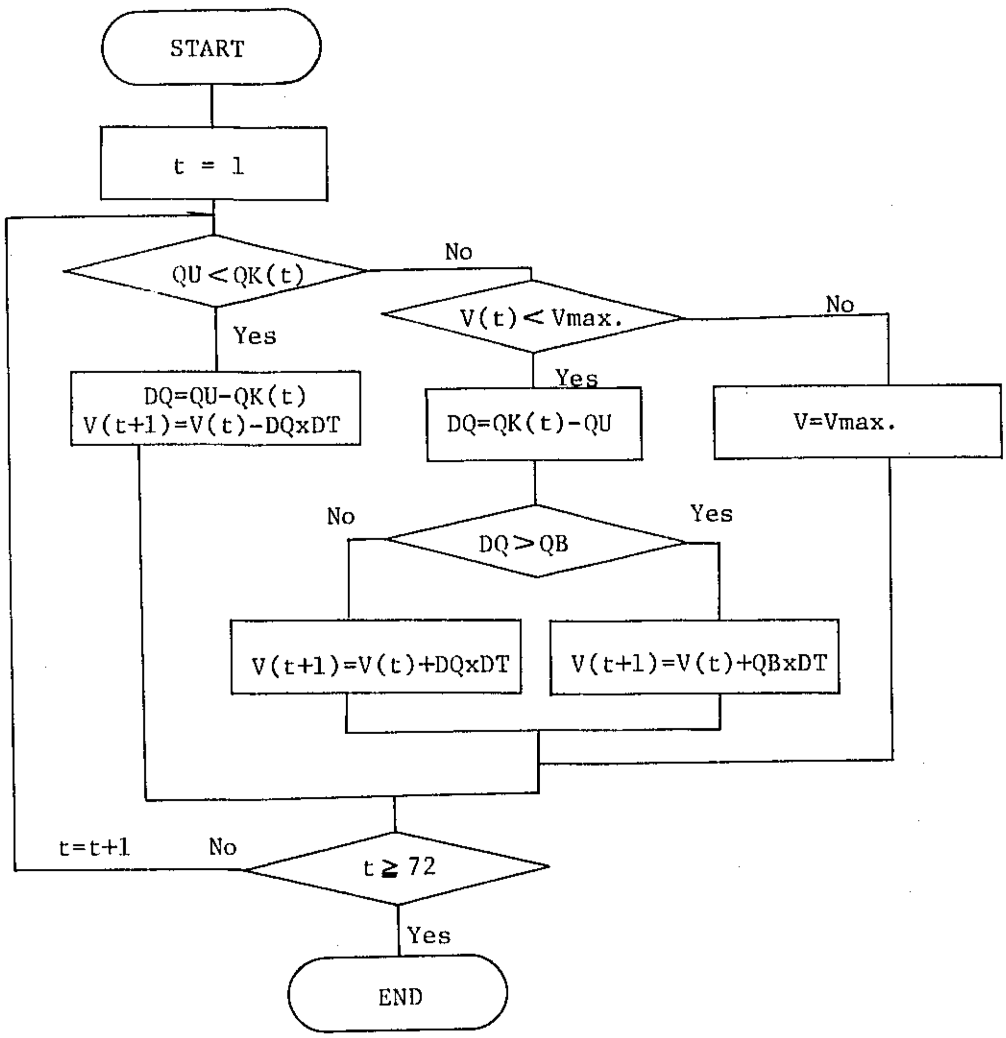
Fig. 4-15 DESIGN FLOOD DISTRIBUTION AND HYDROGRAPH



☐ : Discharge after Regulation by Natural Buffer

( ) : Discharge joining the Main Stream

Fig. 4-16 UTILIZABLE DISCHARGE FLOW



Note;

t : Calculation step at every 5 days

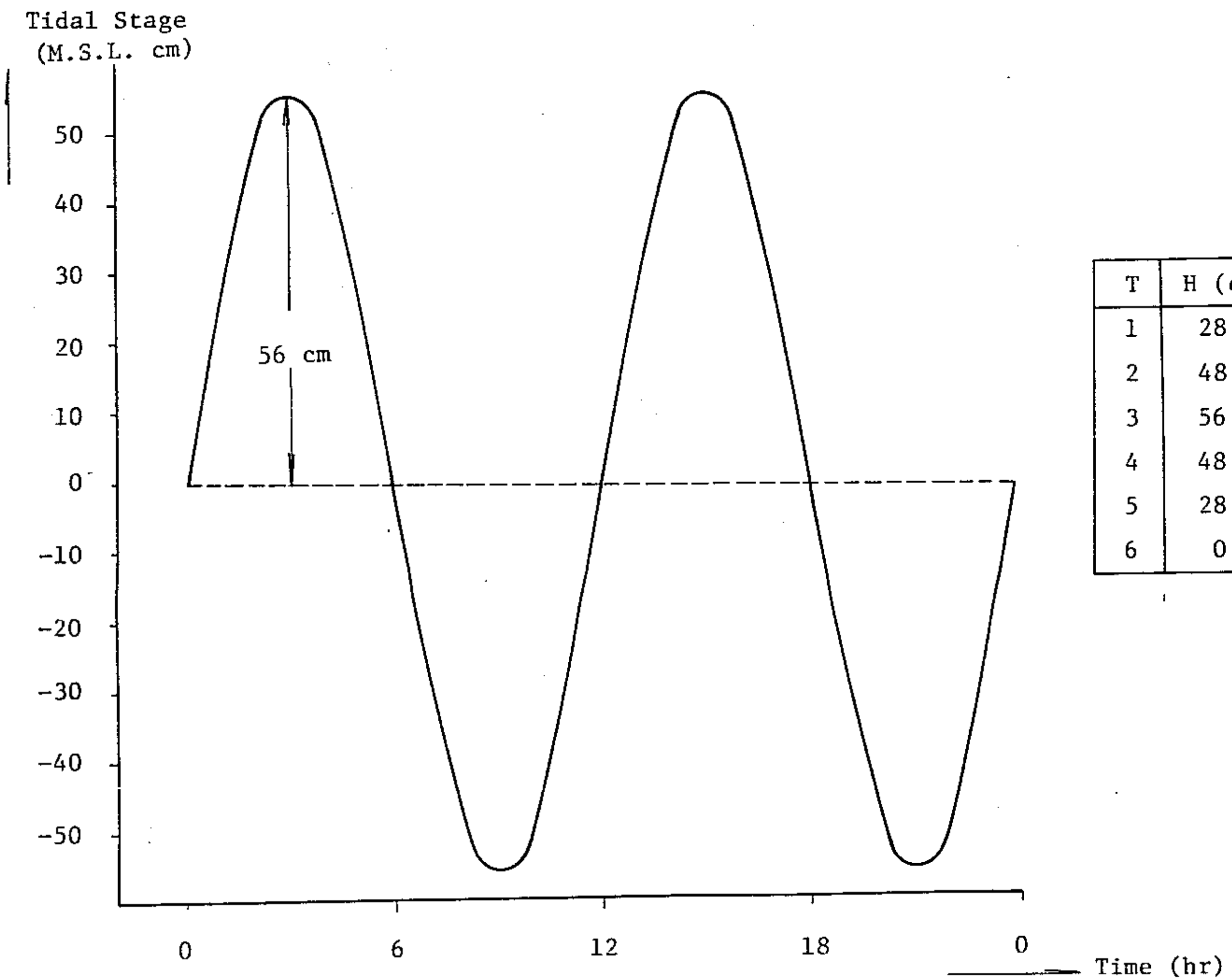
Qu : Utilizable discharge at Kampili

Qb, Qk : 5-day mean discharge at Kampili and Bili-Bili

Vmax : Storage capacity for utilizable discharge

v : Utilized storage at the end of every 5 days

Fig. 5-1 . MODEL TIDAL HYDROGRAPH



T	H (cm)	T	H (cm)
1	28.0	7	-28.0
2	48.5	8	-48.5
3	56.0	9	-56.0
4	48.5	10	-48.5
5	28.0	11	-28.0
6	0.0	12	0.0

II GEOLOGY

C O N T E N T S

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1. GEOLOGY OF THE PROJECT AREA

1.1 General

The topography of the project area is characterized by Lompobatang Volcano. Although Lompobatang Volcano still has the features of the volcano, it is not known whether or not this is active. Especially the southern part is dominated by the volcanic features, and this part has a gentle and long slope. Since Lompobatang Volcano is well eroded, in the project area, the basement rock of volcano is cropped out. The basement rock of the volcano is identified to be the Tertiary volcanic rocks according to the Geological Map of Indonesia.

The basement of the Sulawesi Island is known to be the Paleozoic rocks, schists or ultrabasic rocks. But none of them exposed in the project area. The project area provides, 1) Tertiary sedimentary rocks, 2) Dikes, 3) Lompobatang volcanic products and 4) Alluvium. Fig. 1-1 shows the geological features of the project area.

Tertiary sedimentary rocks consist mainly of volcanic breccia, tuff, sandstone and siltstone, which are distributed in the middle reaches of the Jeneberang river. Likewise, dikes lie in this middle reaches, and are composed of diabase and microdiorite intruding into Tertiary sedimentary rocks.

Lompobatang volcanic products are distributed in the upper reaches and consist of andesite lavas, pyroclastic rocks, mud flows and volcanic ashes.

The Alluvium is distributed along the rivers and at the plain of the lower reaches of the Jeneberang.

The Alluvium along the rivers consists of sand and gravel in the middle and upper reaches. In the lower reaches it consists of sand and silt. The Alluvium in the plain is composed of sand and clay whose soil properties are well known.

1.2 Strata

The geological map of the project area, as shown in Fig. 1-1, was drawn on the basis of observation of the aerophotographs and field investigation and by using the existing data /1 . The map shows that the area provides Tertiary sedimentary rocks, Tertiary Dikes, Quarternary Lompobatang volcanic products and Alluvium.

/1 : a) RAB SUKAMTO (1975) : Geological Map of Indonesia, Ujung Pandang Sheet, Geological Survey of Indonesia.

b) Dinas Geologi Teknik Hidrologi Sekis Geologi Teknik (1972): Laporan Pendahuluan Geologi Dikota Makassar Sulawesi Selatan.

Tertiary Sedimentary Rocks

These are distributed in the middle reaches of the Jeneberang river and also in the middle and upper reaches of the Tallo river.

The facies consist mainly of relatively hard massive impermeable volcanic breccia and tuff breccia bearing subangular to subrounded andesite breccia. The matrix consists of relatively hard medium-grained tuff. Uniaxial compressive strength is estimated at around 400 kg/cm^2. Soft sandstone and siltstone are found near the Bili-Bili dam site.

The relation of the above-mentioned strata is not clarified. However, it is inferred that sandstone and siltstone are intercalated in the volcanic breccia or that they have the relationship of interfinger.

The geological age is in the Neogene (Miocene).

Tertiary Dikes

The Tertiary dikes are distributed in the middle reaches of the Jeneberang river and of the Tallo river.

The facies consist of diabase and microdiorite. Diabase is characterized by big (ø 10 mm) phenocrysts of pyroxene and hornblende. Microdiorite is composed of small (ø 1 mm) phenocrysts of hornblende, plagioclase and quartz.

Both of these are the intrusive rocks into Tertiary sedimentary rocks. The contacts are found near the proposed Bili-Bili dam site. The dips of intrusive rocks range from 80° to 90°. Tertiary sedimentary rocks near the contact are undergone by the thermal metamorphism.

The width and length of dikes are around several hundred meters and several kilometers respectively. They have many tension cracks caused by cooling of the intrusive body, resulting in high permeability of the rock mass.

The geological age is in the Neogene (Miocene).

Lompobatang Volcanic Products

The Lompobatang volcanic products are distributed in the upper reaches of the Jeneberang river.

The facies consist of andesite lavas, tuff breccias, mudflows, volcanic ashes and their alternating beds.

The Lompobatang Volcano has several calderas. One of the calderas is situated in the uppermost reaches of the Jeneberang river. The splendid outcrop of the alternating beds of andesite lava and tuff breccia is observed on the caldera wall.

Mud flows are seen along the Jeneberang river between the lower end of the caldera and Gentoong, the latter of which is located around 4 km southwest of Malino. Mud flows also lie along the Keonisik river, a tributary of the Jeneberang river, near Padammaloeloek.

The mud flow distribution area is well cultivated as a paddy field. Mud flows sometimes form terrace along the Jeneberang river. Collapses in the caldera wall and in the terrace scarp produce a large volume of sediments.

The geological age is in the period from the Pleistocene to the Holocene.

Alluvium

The Alluvium is distributed in a river bed and in the plain lying in the lower reaches.

The facies of the Alluvium in the river bed of the upper reaches of the Kampili weir consist of cobbles, pebbles and sand, and often bear boulders in the upper reaches of Saluttowa.

The Aluvium which lies in the plain consists of sand and clay.

Based mainly on the existing data of drilling and Dutch cone penetration test, the schematic geological profile is shown in Fig. 1-2. The thickness of the Alluvium is around 20 m. Resistance to cone penetration of clay and sand proved to be $qc = 10\ kg/cm^2$ and $qc = 40\ kg/cm^2$ respectively.

There is a tendency that the upper part of the Alluvium consists of sand and the lower one consists of clay.

Tertiary sedimentary rocks with qc in excess of 250 kg/cm^2 lie under the Alluvium.

The geological age is in the Holocene.

1.3 Geological Structure

As described before, the project area provides post-Cretaceous strata and igneous rocks. In the area no pre-Tertiary rock is observed, though the existing data indicate that the basement which is composed of schist, ultra basic rock, etc. is distributed at 50 km north of Bili-Bili. It is presumably inferred that Tertiary rocks also cover the basement unconformably in this area.

The geological structure is not definite, but it seems that bedding plane is gentle, judging from the data obtained at some places; e.g., near the Bili-Bili dam site and Arakkuyu.

The project area is characterized by prevalence of intrusive dikes into Tertiary sedimentary rocks. Dikes trend dominantly E-W and NW-SE, and also NE-SW in lesser amount, and dip nearly vertically.

Quarternary Lompobatang volcanic products cover the Tertiary rocks unconformably.

The major fault is not described in the existing data/1, and also was not found during this investigation. However direction minor faults running in N-S direction are found in the aerophotographs, though their appearance is not clear.

Seismicity of this area is not active. The isoseismic map of Indonesia shows that the lower reaches of the Jeneberang river are included in the zone where the maximum acceleration is between 0.01 and 0.02g (g is the acceleration of gravity). It is reported, however, that there is an active zone in the range from 0.15 to 0.3 g around the Lompobatang Volcano. Further investigation on the seismicity in this area is required.

2. GEOLOGICAL CONDITIONS AT THE STRUCTURE SITES

2.1 Dam Sites

Figs. 2-1, 2-2 and 2-3 are the geological maps drawn on the basis of the field investigation and the aerophotographs of the three proposed dam sites, namely, Bili-Bili, Pasaratowaya and Jonggoa.

Bili-Bili Dam Site

The Bili-Bili dam site consists of Tertiary sedimentary rocks, dikes and river bed deposits. The first of which is composed of relatively soft siltstone, sandstone and tuff breccica. The second consists of diabase and microdiorite. These trend NW-SE at the left bank and N-S at the right bank. The dikes have many cracks caused by cooling of the body. Some of the cracks appear open several centimeters at the top of the right bank hill, and therefore, have high permeability.

The contact between the dikes and siltstone is found near the Bili-Bili staff gauge. The outcrop indicates that siltstone is undergone by thermal metamorphism several meters in width due to intrusion of the dike.

Siltstone, sandstone and tuff breccia are impermeable except for the section near the contact where cracks are dominant. The permeability near the contact can be improved by means of grouting.

/1 : Ujung Pandang Sheet of the Geological Map of Indonesia

Siltstone and sandstone have insufficient strength as a foundation of gravity dam, though sufficient for a rock fill dam.

Embankment materials can be obtained near the dam site. Dikes provide rock materials. Sandstone and siltstone provide core materials. River bed deposits provide filter materials. Though it is expected that the sufficient volume of core materials is distributed, the exact volume is not identified. Therefore, further investigation is needed.

Pasaratowaya Dam Site

The Pasaratowaya dam site is composed of Tertiary sedimentary rocks and dikes. The dikes trend E-W on both sides of the river. The right one is 400 m in width and 1300 m in length. The left one is 300 m in width and 1000 m in length. The dikes are composed of very hard diabase having many tension cracks. The topography is characterized by the high ridge of dike along its strike. Therefore, it is easy to find dike from the aerophotographs.

The Tertiary sedimentary rocks consist of tuff and tuff breccia, and the facies of them are similar to that of the Bili-Bili dam site.

The fault which trends E-W is observed along the northern edge of the right bank dike, judging from the aerophotographs. It is also inferred that N-S direction faults exist at some places, and that these faults are caused by displacement of dikes. However, the conditions of these faults are not known yet.

Talus deposits and river bed deposits are also found at this site.

Jonggoa Dam Site

The Jonggoa dam site consists of Tertiary sedimentary rocks. Their facies at this site are tuff breccia and tuff. Talus and river bed deposits cover the rock.

Tuff breccia is impermeable and have sufficient strength as a foundation of a rock fill dam.

A comparative study of the proposed dam sites will be performed in this Supporting Report, "Dam and Reservoir".

2.2 Pumping Station

The pumping station will be located near the Pampang bridge. The foundation of the station is composed of the Alluvium consisting of soft sand and clay. The existing data verify that the thickness of the Alluvium is estimated to be around 20 m.

Since the topographic mound which is composed of Neogene sedimentary rocks is located 200 m east of the station along Jl. Gowa Raya, the exact thickness of the Alluvium might be smaller than the estimated value.

The soil properties are known by the existing data /1 (refer to Table 2-1).

2.3 River Banks

The foundation of the Jeneberang river banks consists of sand and silt. For details, the foundation is composed of sand in the upper reaches of the plain area and of silt and fine-grain sand in its lower reaches. Both of them have a sufficient bearing capacity for embankment.

/1: a) Direktorat Geologi (1973): Laporan Singkat Mengenai Lapisan Batuan Untuk Fondasi Disekitar Daerah Rencana Cathay International Hotel Ujung Pandang.

b) Dinas Geologi Teknik Hidrologi Seksi Geologi Teknik (1972): Laporan Pendahuluan Geologi Dikota Makassar Sulawesi Selatan.

c) Detail Engineering Emergency Program Assainering Kotamadya Ujung Pandang (1977):

- Tables of Laboratory Test Result
- Hand Auger Boring Columns
- Cone Resistance Figure

d) Nurse Education Facilities in Ujung Pandang in the Republic of Indonesia Feb. 1979: Boring Columns.

e) Peta geologi Daerah Kotamadya Ujung Pandang

Table 2-1 SOIL MECHANIC FEATURES

Characteristics of Soil		Symbol	Unit	Sand	Clay
Resistance to Cone Penetration		qc	Kg/cm2	20-70 Ave.= 40	5-12 Ave.=10
Specific Gravity		-	-	2.53	2.43
Density	Dry	γd	gr/cm3	0.978	0.939
	Wet	γw	gr/cm3	1.732	1.572
Water Contents		Wn	%	73.39	67.48
Atterberg	Liquid Limit	LL	%	Np	78.0
	Plastic Limit	PL	%	Np	49.37
	Plasticity Index	PI	%	Np	28.63
Unconfined Compressive Strength		qu	Kg/cm^2	-	1.44
Triaxial	Cohesion	c	Kg/cm^2	0.144	0.438
	Friction	ϕ	degree	34°	18°
Consolidation	Compression Index	Cc	-	-	0.276
	Coefficient of Consolidation	Cv	cm^2/sec	-	$7.6x10^{-4}$

Fig. 1-1 GEOLOGICAL MAP

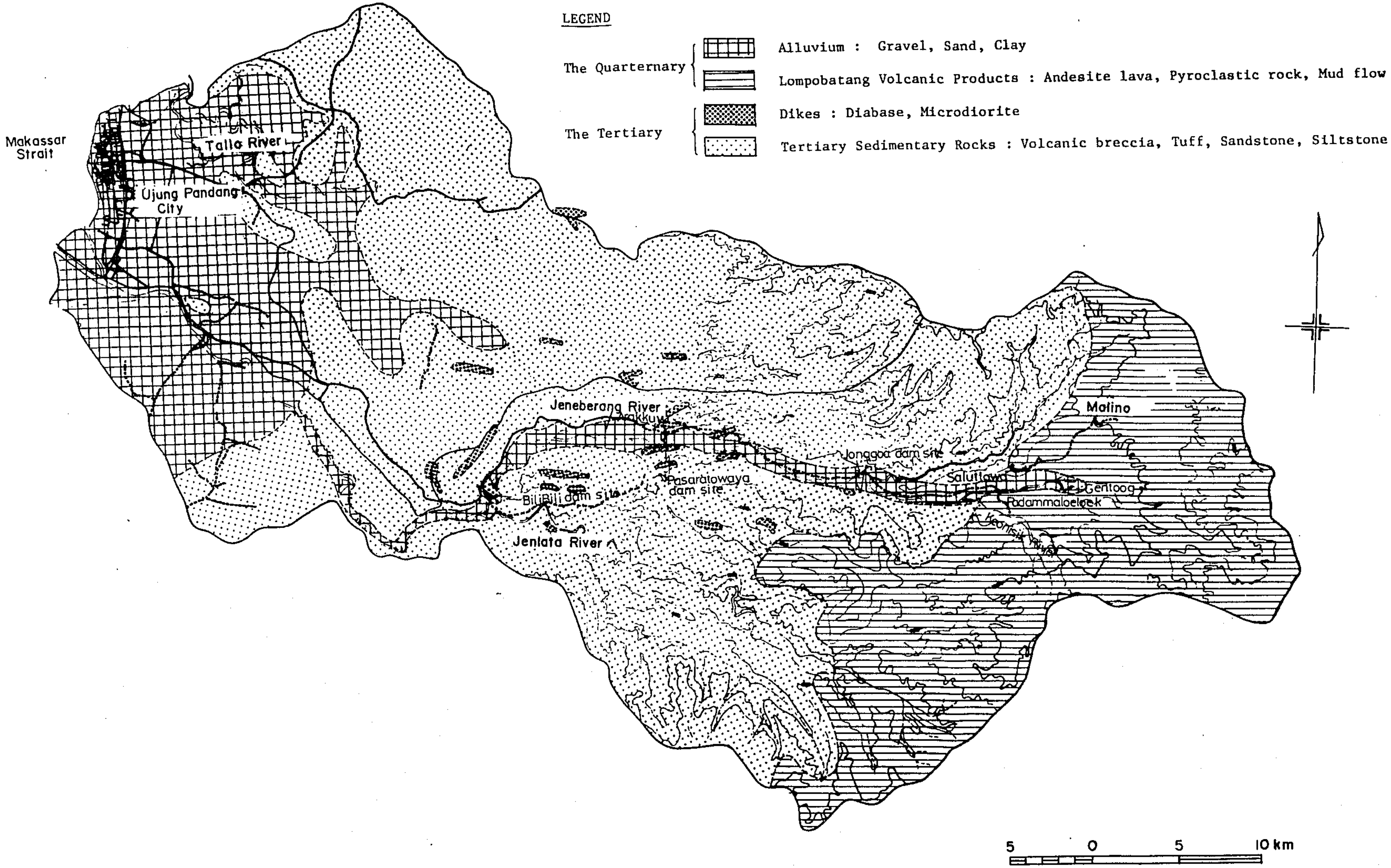
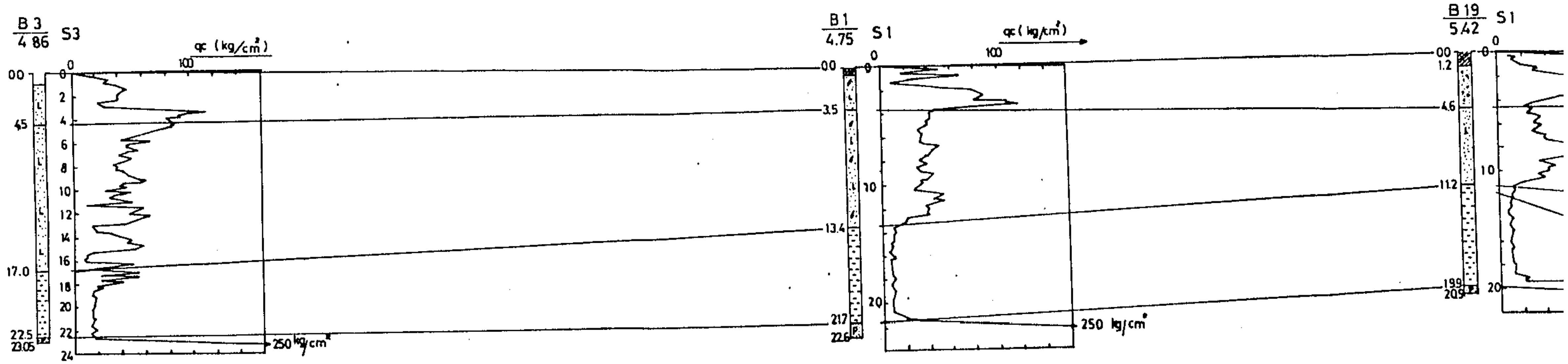


Fig.1-2 SCHEMATIC GEOLOGICAL PROFILE NEAR UJUNG PANDANG

Scale $\frac{\text{Horizontal 1:5000}}{\text{Vertical 1:400}}$



LEGEND

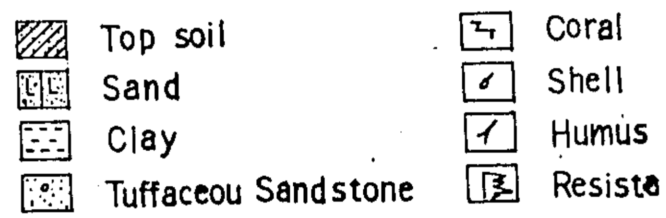
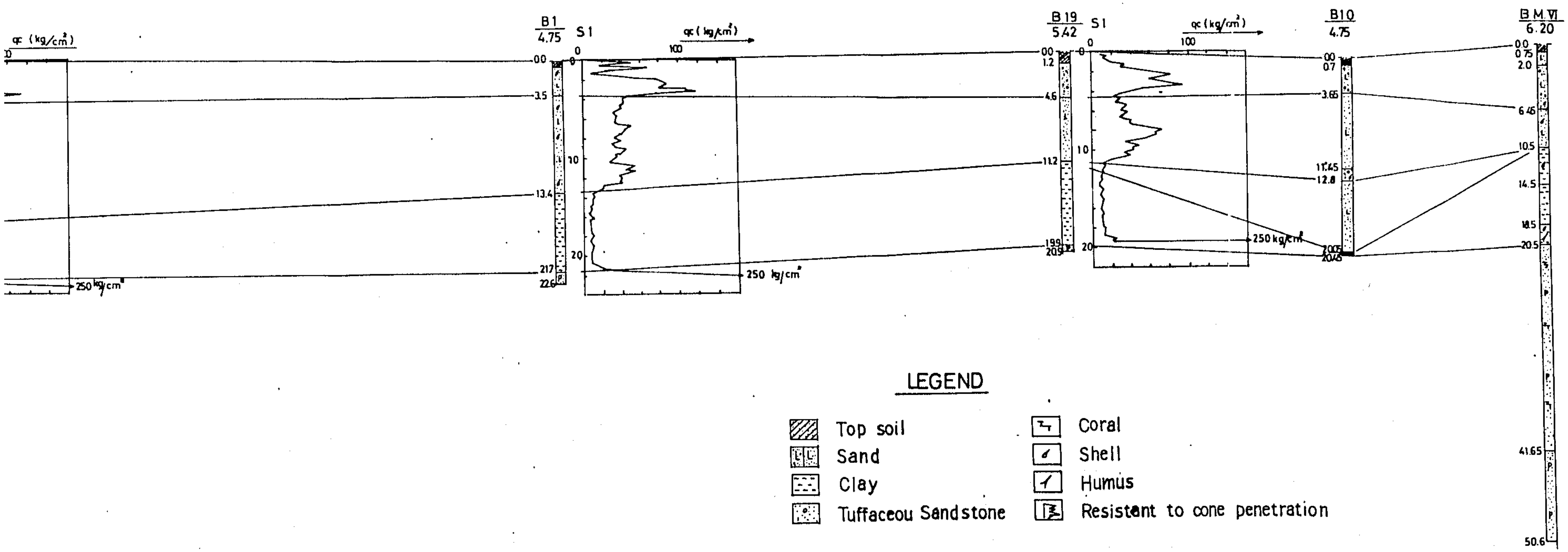


Fig.1-2 SCHEMATIC GEOLOGICAL PROFILE NEAR UJUNG PANDANG

Scale $\frac{\text{Horizontal } 1:5000}{\text{Vertical } 1:400}$



LEGEND

- Top soil
- Sand
- Clay
- Tuffaceou Sandstone
- Coral
- Shell
- Humus
- Resistant to cone penetration

Fig.2-1 GEOLOGIC MAP OF BILI-BILI DAM SITE SCALE 1 : 20,000

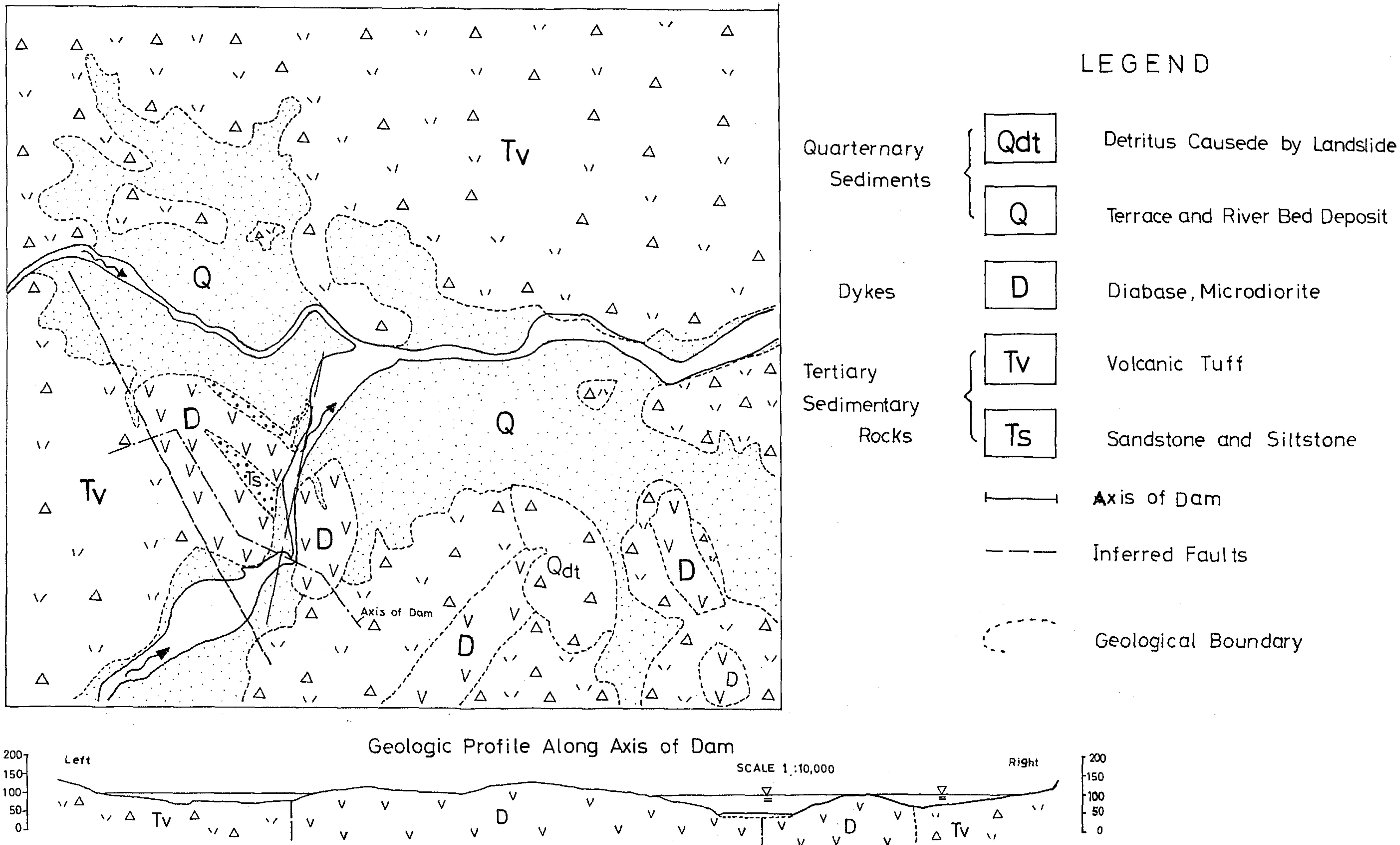
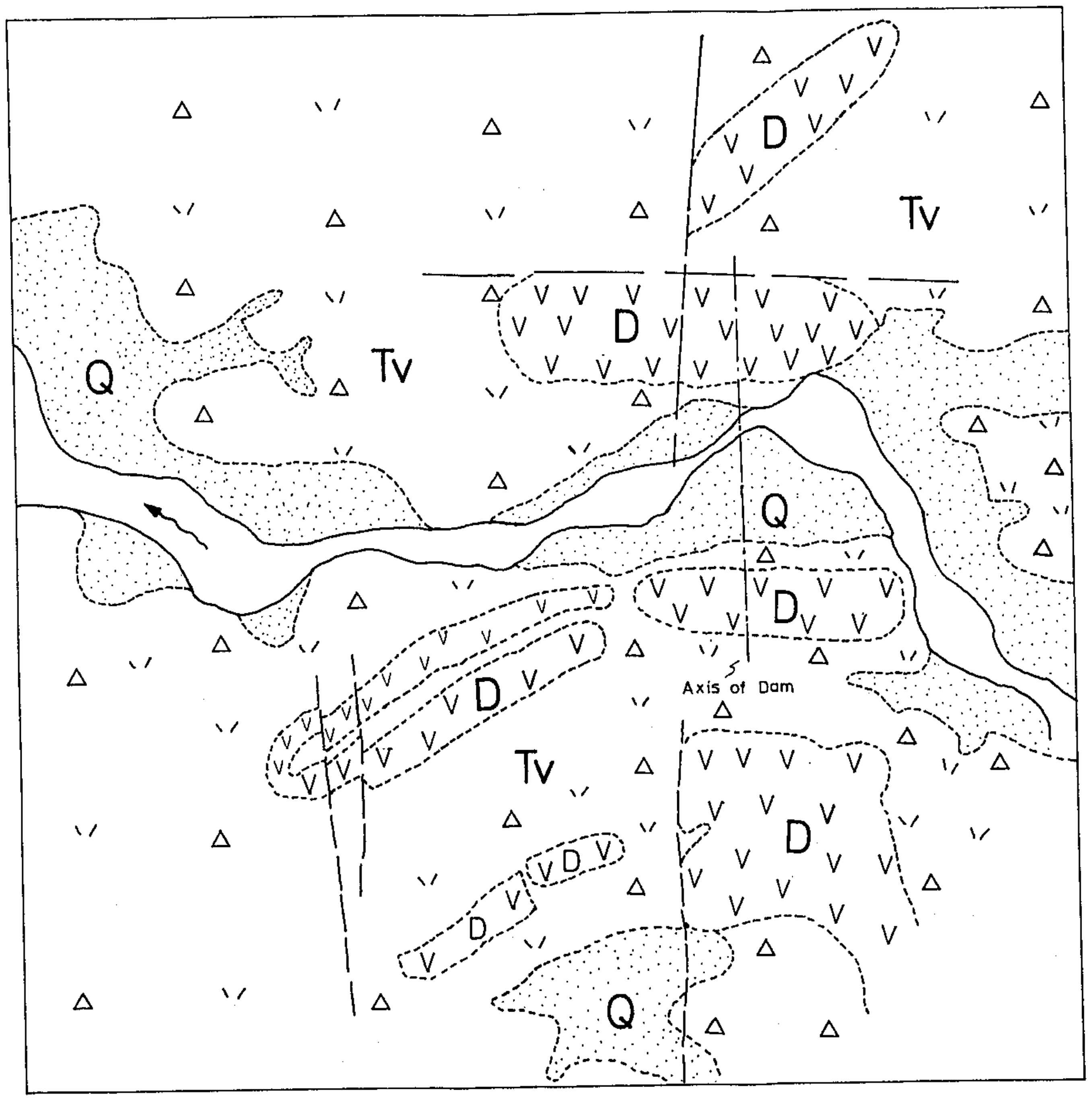


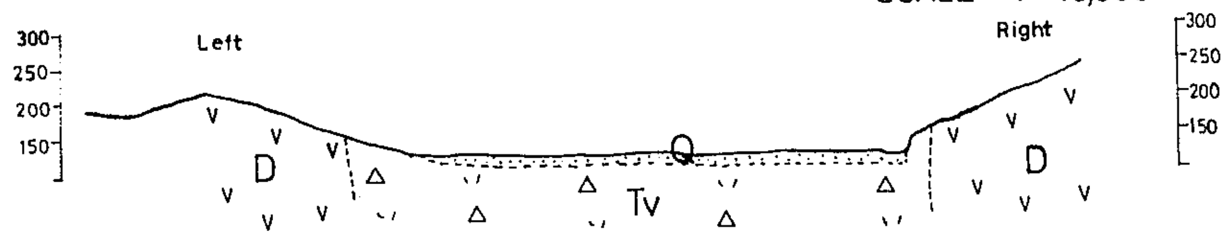
Fig.2-2 GEOLOGIC MAP OF PASARATOWAYA DAM SITE

SCALE 1 : 20,000



Geologic Profile Along Axis of Dam

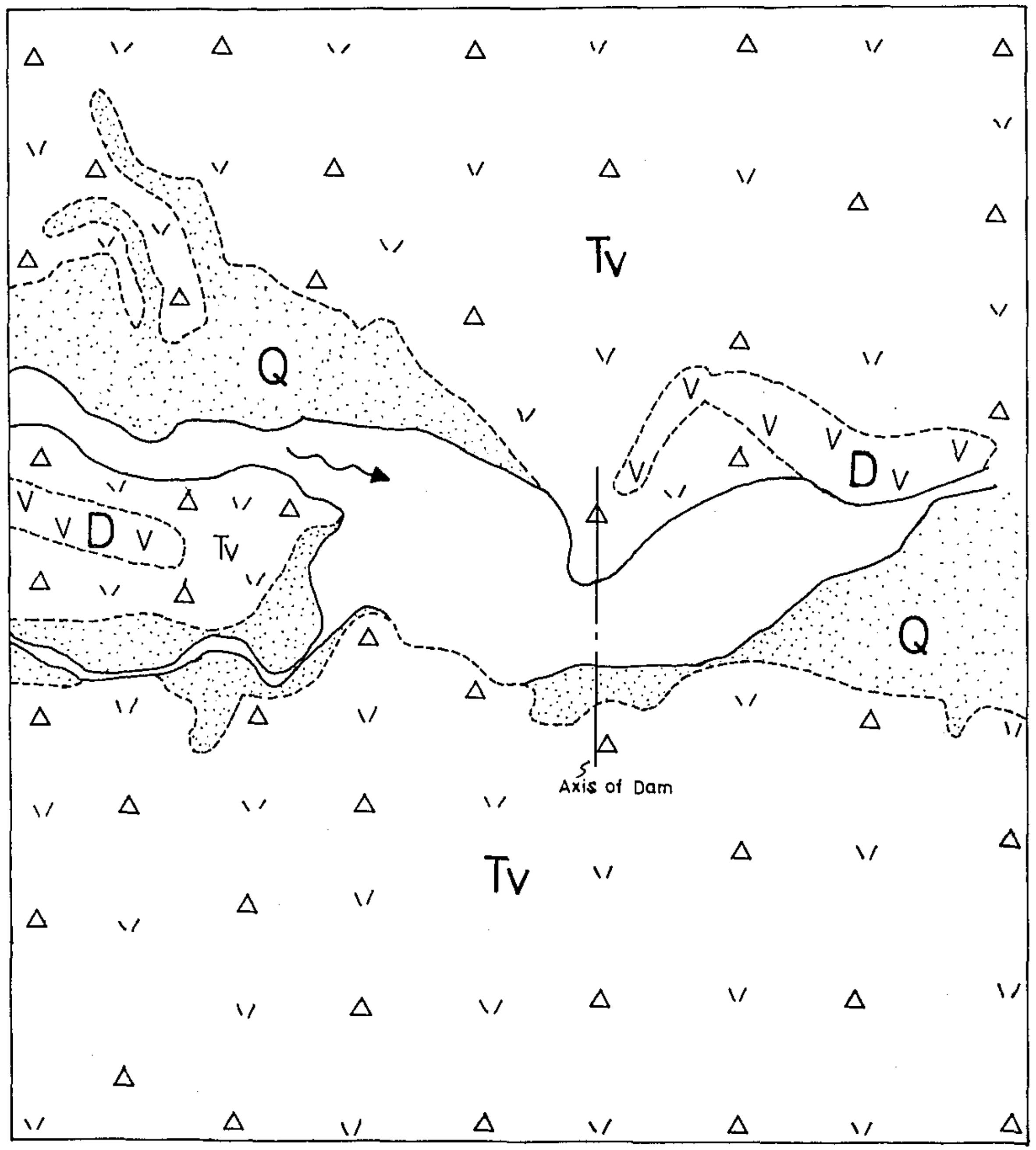
SCALE 1 : 10,000



LEGEND

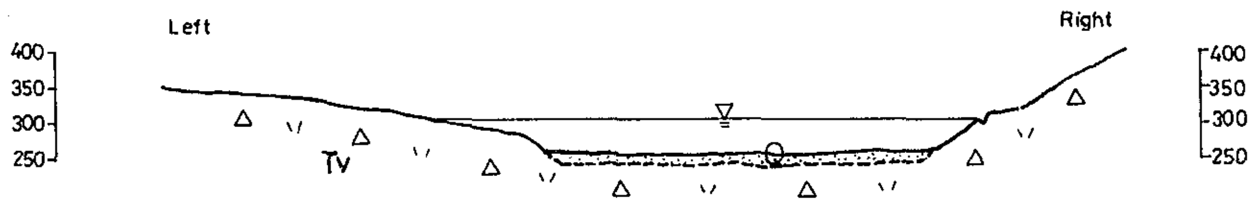
- Q — Quarternary Sediments
- D — Dykes : Diabase
- Tv — Tertiary Sedimentary Rocks : Volcanic Breccia, Tuff
- Axis of Dam
- Inferred Faults
- Geological Boundary

Fig.2-3 GEOLOGIC MAP OF JONGGOA DAM SITE SCALE 1:20,000



Geologic Profile Along Axis of Dam

SCALE 1: 10,000



LEGEND

Q	Quarternary Sedments
D	Dykes : Diabase
Tv	Tertiary Sedimentary Rocks :Volcanic Breccia, Tuff
(dashed line)	Geological Boundary
(solid line)	Axis of Dam
(broken line)	Inferred Faults (Lack in This Map)

III SABO AND SOIL CONSERVATION

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1. SABO

1.1 General

The field investigation and the observation of aerophotographs verified that the grassland (including small denuded areas) occupies about 7% of the Jeneberang river basin (727 km^2), and that the area of collapse which is now producing sediments is less than 0.03%. These areas correspond to 13% and 0.05% of the upper basin of the Bili-Bili dam site. The remaining 93% is the forest and farmland. It seems that the grassland does not produce sediments because the river channels in the grassland appear stable.

1.2 Sediment Discharge

To estimate the annual sediment discharge, the following studies have been done.

1) Estimation by Ezaki's formula

2) Estimation by Tanaka's formula

3) Estimation by Schoklitsch's formula

4) Study of sediments deposited at the Janeberang estuary

5) Comparative study of sedimentation in reservoirs in Indonesia

<u>Ezaki's formula</u>

Ezaki gives the formula on an annual sediment discharge, that is;

$$Vs = E(8.85\ S^2 I + 7.83 \times \frac{Ad}{A} D^2 I)$$

where,

The first term : The volume of bed material load
The second term: The volume of wash load

- Vs : Sediment discharge corresponding to I (m^3)
- S : Average river bed gradient near the reservoir end
- I : The total amount of discharge which satisfies $QS \geqq 1$ (m^3/year)
- Ad : Area of collapses within the catchment area (km^2)
- A : Catchment area (km^2)
- D : Average gradient of collapses
- E : Trap efficiency which should be adopted from Brune's curve (refer to Fig. 1-2)
- Q : Discharge exceeding $\frac{1}{S}$ (m^3/s)

There are the last ten annual records which show the flow regime at Bili-Bili. Discharge data which exceed 150 m^3/s should be picked up, because the gradient of riverbed is 1/150 at the reservoir end. By using the picked-up data, the annual average discharge over 150 m^3/s is calculated as follows. (Refer to Table 1-1.)

$$I = \Sigma Q \times 86{,}400 = 1{,}564.3 \times 86{,}400 = 135.16 \times 10^6\ m^3/yr$$

Active collapses are selected by observing the aerophotographs, and the total area of collapses (Ad) is 0.2 km^2.

The distribution of the collapses is shown in Fig. 1-1. The average gradient of collapses and the catchment area are determined to be 0.96 and 384.4 km^2 respectively.

The trap efficiency is calculated as follows.

The total annual average inflow to the dam (i) is calculated as follows. (Refer to Table 1-1.)

$$\begin{aligned} i &= Q\ \text{mean} \times 86{,}400\ \text{sec} \times 365\ \text{day} \\ &= 31.275\ m^3/s \times 86{,}400\ \text{sec} \times 365\ \text{day} \\ &= 986 \times 10^6\ m^3 \end{aligned}$$

The total storage capacity (c) of proposed Bili-Bili Dam is;

$$320 \times 10^6\ m^3$$

therefore, ratio of capacity to inflow (c/i) is;

$$\frac{c}{i} = \frac{320 \times 10^6}{986 \times 10^6} = 0.32$$

From Fig. 1-2, the trap efficiency (E) is determined to be 98%.

The annual sediment discharge is;

$$\begin{aligned} Vs &= 0.98[8.85 \times (1/150)^2 \times 135.16 \times 10^6 + \\ &\quad 7.83 \times \frac{0.2}{384.4} \times 0.96^2 \times 135.16 \times 10^6 \\ &= 549{,}407\ m^3/year \end{aligned}$$

therefore, the annual specific sediment discharge is;

$$549.407\ m^3/year \div 384.4\ km^2 = 1{,}429\ m^3/km^2/year$$

<u>Tanaka's Formula</u>

Dr. Tanaka proposed the formula by studying the topographic and geologic conditions on 36 reservoirs in Japan. The topographic condition is composed of relief energy and the average of heights.

According to the geological condition, the catchment area is classified into 9 groups. The formula to calculate the annual specific sediment discharge varies by each group.

First, the topographic map of the project area is to be devided into square areas of 16 km^2 each.

The relief energy (X_1) and the average of heights (X_2) in each square area must be measured.

The weighed average $\overline{X}_1$ and $\overline{X}_2$ should be calculated as follows.

$$\overline{X}_1 = \frac{\Sigma fi \times i}{\Sigma fi}$$

where,

Xi : relief energy in each square (expressed in unit of 100 m)

fi : number of squares where same value of Xi is observed

$$\overline{X}_2 = \frac{\Sigma fi' \times Xi'}{\Sigma fi'}$$

where,

Xi': average height in each square (expressed in unit of 100 m)

fi': number of squares where same value of Xi' is observed

Then the coefficient of topographical feature (X) should be calculated as below.

$$X = \overline{X}_1 \times \overline{X}_2$$

The annual specific sediment discharge (Y) is calculated by applying the value of X to the following formulas.

A group : $Y = 6.6 X - 934 \pm 166$

B group : $Y = 11.8 X - 543 \pm 49$

C group : $Y = 4.5 X + 150 \pm 69$

D group : $Y = 10.1 X - 254 \pm 107$

E group : $Y = 9.9 X - 77 \pm 51$

F group : $Y = 9.0 X - 523$

G group : Y = impossible to be calculated

H group : Y = impossible to be calculated

I group : $Y = 13.0 X - 6 \pm 189$

A, B and C groups are the area consisting of plutonic and hypabyssal rocks and also their metamorphic rocks.

D and E groups are the areas composed of paleozoic rocks.

A, B, C, D and E groups are observed in the special regions in Japan.

The features of the remaining groups are explained below.

F group : composed of crystaline schist
G group : composed of effusive rocks
H group : composed of Cenozoic sedimentary rocks
I group : composed of plutonic rocks, dikes, effusive rocks and Cenozoic sedimentary rocks

The project area is classified into I group. $\overline{X}$ and X is calculated as shown in Table 1-2.

$$X = \overline{X}_1 \times \overline{X}_2 = 6.636 \times 8.03 = 53.287$$

$$Y = 13.0 \times 53.289 - 6 \pm 189$$

$$= \underset{\text{(max.)}}{875.7} \sim \underset{\text{(average)}}{(686.7)} \sim \underset{\text{(min.)}}{497.7}$$

This formula which is adopted only in Japan should be modified for application in Indonesia. The difference between Japan and Indonesia is the climate. Especially the annual mean rainfall of the project area is two times as much as that of the mountainous areas in Japan, and intensity of rainfall in Indonesia is generally stronger than that in Japan. Therefore, the results are considered to be too small to adapt to conditions in Indonesia. The value thus obtained may have to be modified by larger values, taking the rainfall pattern and intensity in Indonesia into consideration.

Schoklitsch's Formula

Schoklitsch's formula is expressed as follows.

$$G = \Delta(Q \times A)^{0.2}$$

where,

G : annual sediment discharge (m^3)
Q : annual mean discharge of river (m^3)
A : catchment area (km^2)
Δ : coefficient

Coefficient Δ varies with the following conditions.

100 - 300 : for better condition in the basin
600 - 1000 : for the big basin composed of various complex layers
1,650 - 4,500 : for wild devasted rivers where erosion is prevalent

The Jeneberang river has 986 x 10^6 m^3 of annual mean discharge and 384.4 km^2 of catchment area at Bili-Bili. Considering topographic and geologic conditions of the project area, it is assumed 2,500 for the value of coefficient "Δ" is appropriate.

Applying Δ = 2,500, $Q = (986 \times 10^6 \times 384.4)^{0.2} \times 2{,}500$
$= 517{,}500\ m^3/year$

Therefore, annual specific sediment discharge is;

$517{,}500\ m^3/year \div 384.4\ km^2 = 1{,}346\ m^2/km^2/year$

Study of Sediments Deposited at the Estuary

Sediments deposited at the Jeneberang estuary is calculated by comparing two sea charts surveyed in 1900 and in 1979. The latter is surveyed by this study team. The transition of the shore line and the depth contour lines are shown in Fig. 1-13 in the "River Improvement" section of this Supporting Report.

The depth of seabed is shallowed by sediments transported by the Jeneberang river (refer to Fig. 1-13). The total volume of sediments is calculated to be around 60 x 10^6 m^3. Therefore, the annual specific sediment discharge is;

$60{,}000{,}000\ m^3 \div 79\ year \div 727\ km^2 = 1{,}049\ m^3/km^2/year$

This figure is the average volume of sediments in the whole catchment area including plain lands. Considering only the mountainous area, this figure will increase.

Comparative Study of Sedimentation in Reservoirs

A comparative study of sedimentation in reservoirs in Indonesia has been done and the results are as follows.

Karankates dam (the Brantas R.)	: 980 - 1,460 $m^3/km^2/year$
Selorejo dam (the Kalikonto R.)	: 850 - 1,700 $m^3/km^2/year$
Wonogiri dam (the Sala R.)	: 1,170 $m^3/km^2/year$

1.3 Determination of the Annual Specific Sediment Discharge

Each method provides various results, such as

1) Ezaki's formula : 1,429 $m^3/km^2/year$

2) Tanaka's formula : 497 ∿ 687 ∿ 876 $m^3/km^2/year$
(min.)(average) (max.)

3) Schoklitshe's formula : 1,346 $m^3/km^2/year$

4) Sediments in the estuary : 1,045 $m^3/km^2/year$

5) Existing data of other dams : 850 - 1,700 $m^3/km^2/year$

In Japan Ezaki's formula is said to be the most adoptable. All the results are similar to one another. Therefore, the annual specific sediment discharge is determined to be 1,500 $m^3/km^2/year$ in due consideration of the safety.

2. SOIL CONSERVATION

2.1 General

Lompobatang caldera exists at the uppermost reaches of the Jeneberang river. This caldera was formed by the depression. The sharp edge of the caldera wall indicates that this caldera may have been formed in the Holocene.

Sediments are produced by the wall collapse and by the scarp collapse in terrace and fan along the Jeneberang river. Since most of the caldera wall consist of hard rock, it is not anticipated that collapse will occur within the centuries to come.

The collapses of the wall are observed along the faults and weak rocks.

Riverbed deposits begin to increase from Sirondjong toward the lower reaches. Sediments are also produced from the riverbed. The distribution of collapses and riverbed deposits is shown in Fig. 1-1.

2.2 Sabo

The vast source of sediments is described in Section 2.1. The amount of sediments discharge is studied in Section 1.3. This section deals with the countermeasure against sediment discharge.

Sabo dam has generally three functions.

1) To catch the sediment discharge

2) To regulate the sediment transportation

3) To prevent producing sediments

The third function is arrived at from riverbed rising at the upper reaches of the Sabo dam, because the riverbed raised by sediments plays the role of counterweight to prevent a new occurrence of collapse (refer to Fig. 2-1.)

Then it is effective to install Sabo dam in the area producing sediments.

The Sabo dam sites are selected by following conditions (refer to Fig. 1-1).

1) The lower reaches of the active collapse which is now producing sediments

2) A narrow valley which can save the total volume of a dam (resulting in saving of construction cost)

3) The place which can prevent lateral erosion of terrace and fan scarp by means of raising the riverbed (refer to Fig. 2-1)

4) The place to which the construction materials can be conveyed without difficulties

It is difficult to control thoroughly sediment discharge by the proposed three dams, if repeated collapses occure in the future. The main purpose of the Sabo dams proposed in this study is to prevent the local disasters.

It is recommended that Sabo dams be constructed by concrete gravity type.

The foundation of Sabo dams consist of thick sand and gravels. Therefore, Sabo dams have to be of a floating type.

An example of a Sabo dam is illustrated in Fig. 2-2.

2.3 Soil Conservation

Excluding the area of Lompobatang caldera, most of grassland and denuded land have been formed by human activities. Then, it is not difficult to approach the grassland and denuded land.

As described in Section 1.1, rivers in the grassland appear stable and produce less sediment. Accordingly, a countermeasure is not required urgently from the viewpoint of soil conservation, but it is preferable that afforestation be conducted not only to provide forestry, but also to provide water retention, flood run-off control and better natural scenery.

It is observed that trees are growing in the grassland, especially in and around the places where water accumulates. Afforestation is realizable by selecting the species suitable for the site and by providing trenches which will increase the water content of the ground and is effective against sheet erosion. A water chute crossing the trenches is required to maintain water stream (refer to Fig. 2-3).

Further investigation will be required to provide afforestation successfully.

Table 1-1 TOTAL DISCHARGE VOLUME

YEAR	DISCHARGE (m^3/s)*	Mean Annual Dischage (Qmean) (m^3/s)
1953	2,666.42	31.26
1956	1,242.32	33.91
1957	713.97	29.24
1958	886.46	27.76
1959	1,912.37	35.05
1960	2,016.22	30.03
1975	0.00	30.03
1976	1,400.08	25.24
1077	4,639.41	41.56
1078	166.22	28.67
TOTAL	15,643.47	312.75
Mean	1,564.3	31.28

NOTE: * - Only the discharge data exceeding 150 m^3/s are summed up.

($I = 1{,}564.3\ m^3/s \times 86{,}400\ sec = 135.16 \times 10^6\ m^3$)

($Q\,m^3/year = 31.28 m^3/sec \times 86{,}400 sec \times 365 day = 986 \times 10^6 m^3/year$)

Table 1-2 VALUES OF RELIEF ENERGY AND AVERAGE HEIGHT

Height (m)	Relief Energy (X1)			Average Height (X2)		
	Xi	fi	Xi.fi	Xi	fi	Xi.fi
100 -- 199	2	1	2	2	3	6
200 -- 299	3	4	12	3	4	12
300 -- 399	4	3	12	4	2	8
400 -- 499	5	6	30	5	4	20
500 -- 599	6	2	12	6	1	6
600 -- 699	7	5	35	7	5	35
700 -- 799	8	4	32	8	1	8
800 -- 899	9	4	36	9	2	18
900 -- 999	10	1	10	10	3	30
1000 --1099	11	1	11	11	3	33
1100 --1199	12	0	0	12	0	0
1200 --1299	13	1	13	13	0	0
1300 --1399	14	1	14	14	1	14
1400 --1499				15	1	15
1500 --1599				16	0	0
1600 --1699				17	1	17
1700 --1799				18	0	0
1800 --1899				19	0	0
1900 --1999				20	0	0
2000 --2099				21	1	21
2100 --2199				22	1	22
Total	–	33	219	–	33	265

$$\overline{X1} = \frac{219}{33} = 6.636$$

$$\overline{X2} = \frac{265}{33} = 8.03$$

$$\overline{X1} \times \overline{X2} = 6.636 \times 8.03 = 53.287$$

$$X = 53.287$$

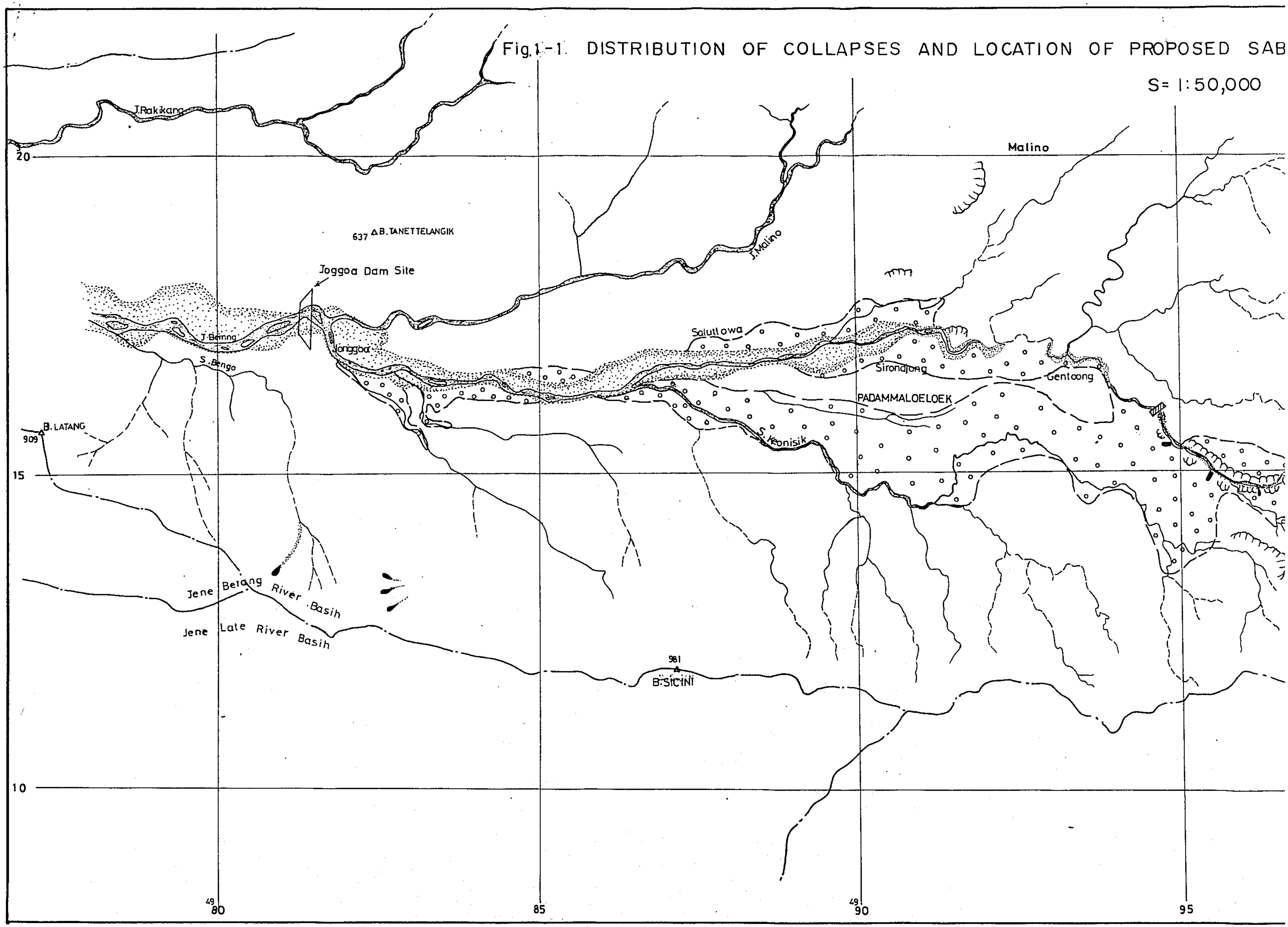


Fig.1-1. DISTRIBUTION OF COLLAPSES AND LOCATION OF PROPOSED SAB

-1. DISTRIBUTION OF COLLAPSES AND LOCATION OF PROPOSED SABO DAMS

S= 1:50,000

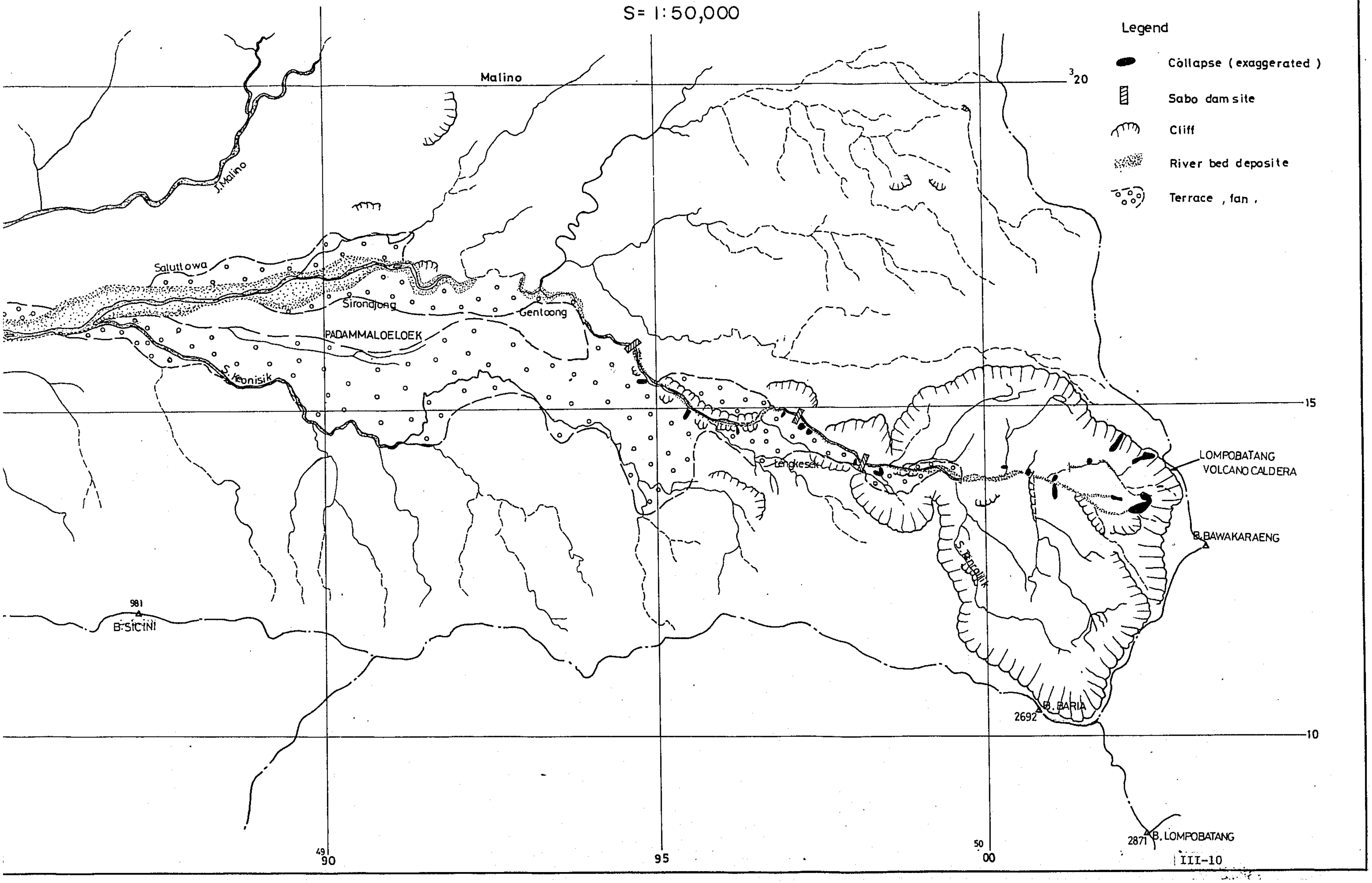


Fig1-2 RESERVOIR TRAP EFFICIENCY AS A FUNCTION OF CAPACITY-INFLOW RATIO

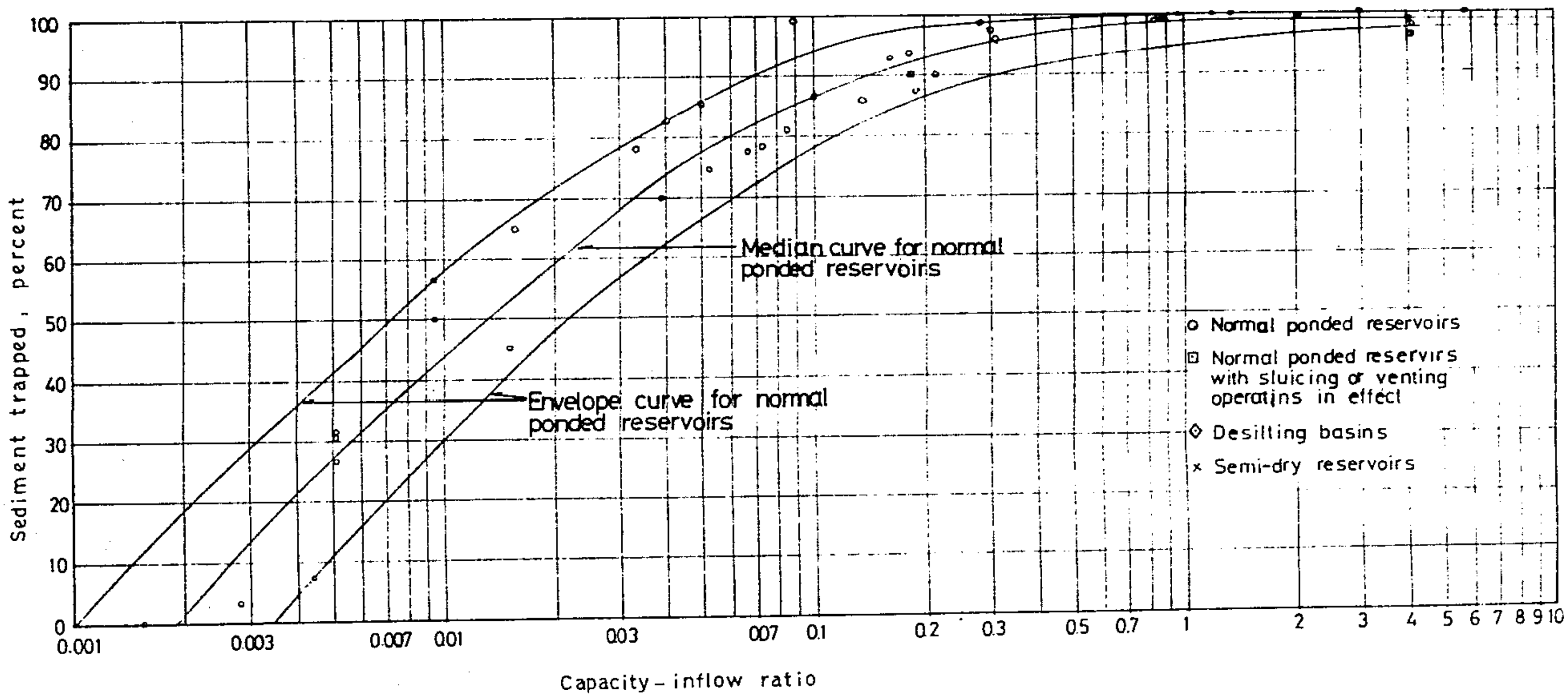
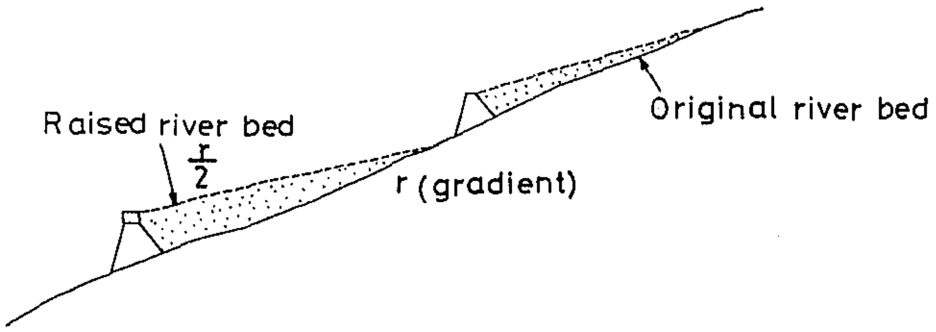


Fig.2-1 FUNCTION OF SABO DAMS

Profile



Cross-Section

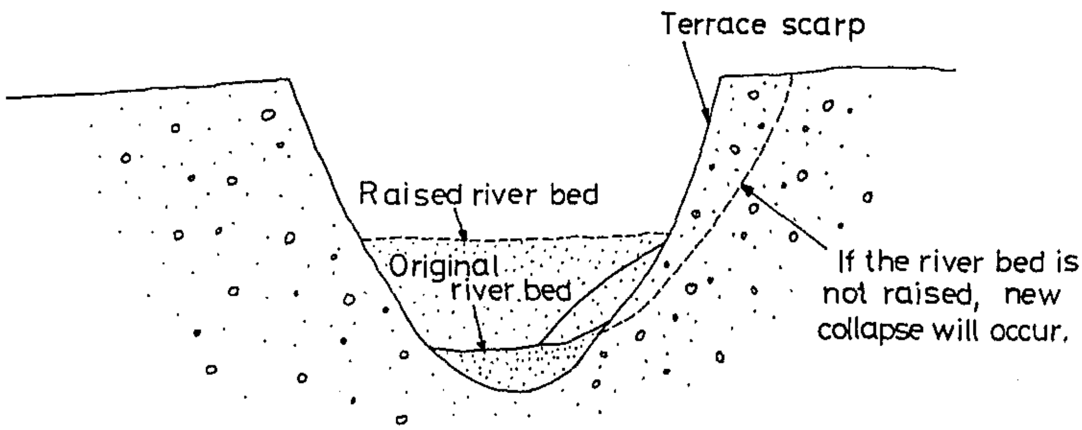


Fig.2-2 STANDARD SABO DAM

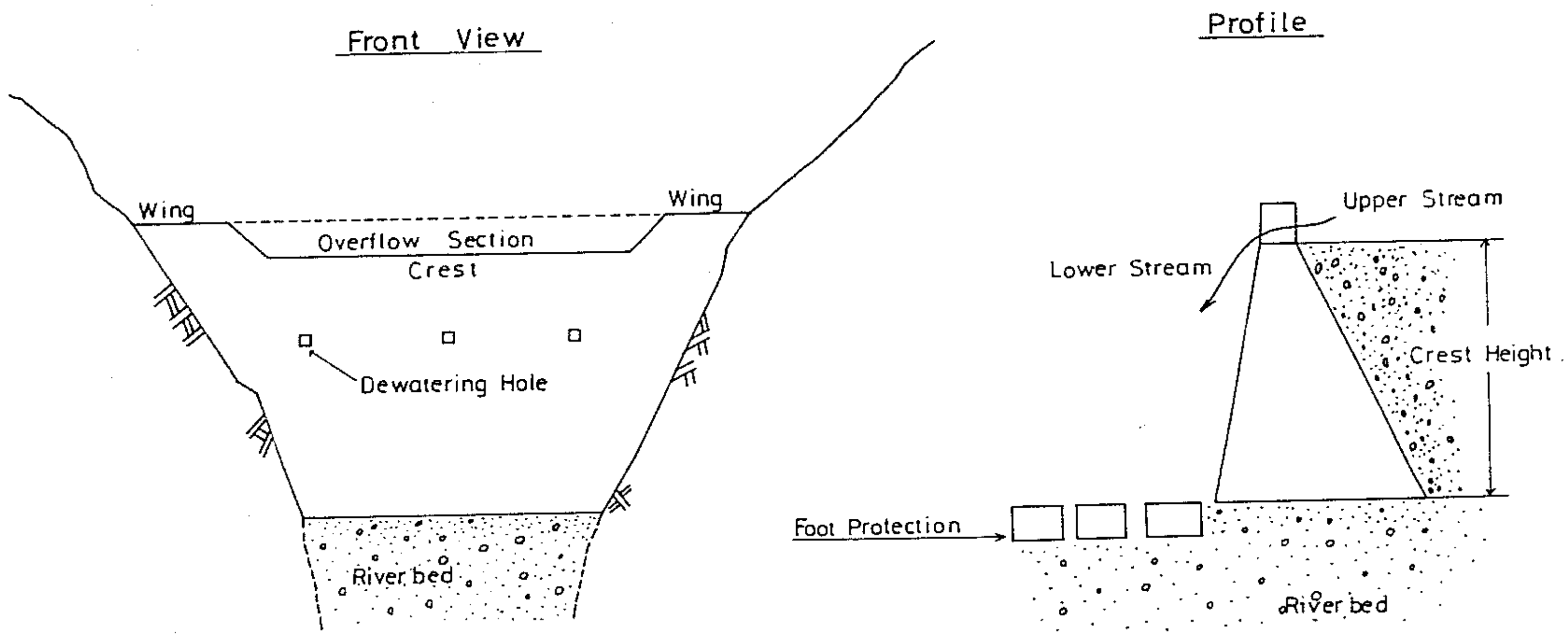
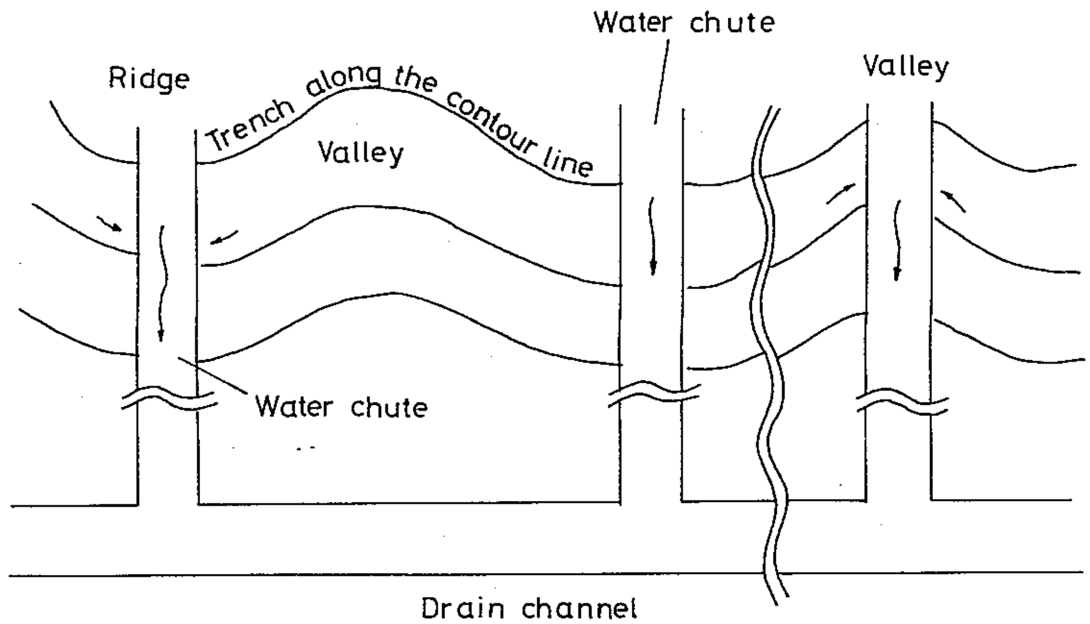


Fig. 2-3 COUNTER TRENCH



IV RIVER IMPROVEMENT

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1 PRESENT CONDITIONS OF RIVERS

1.1 River Basin

The Jeneberang river with a catchment basin of 727 km^2 and a length of 75 km originates in the Bawakaraeng Mountain. The upper reaches show topographically a feather-shaped mountainous land, in which flood run-off concentrates.

The natural vegetation of the mountaineous area is sparse; however, Aran-Aran and a few tall trees do exist. In the top reaches lies a collapse which has produced sediments to raise the riverbed level of the Jenberang river. Below Bili-Bili an alluvial fan has developed on the right and left sides. On the right side of these lower reaches lie an urban area of Ujung Pandang city and agricultural land which is now under urbanization. On the left side extends a fertile agricultural land.

The Tallo river flowing down to the west in parallel with the Jeneberang river has a catchment basin of 417.3 km^2 and a length of 66 km. Flood run-off concentrates in the mountainous upper reaches, and subsequently many places in the lower reaches is inundated, resulting in a swampy area.

Some of these areas in the lower reaches of the Tallo river is used for paddy fields and fish ponds.

1.2 River Conditions

<u>River Channel</u>

The Jeneberang river flows through the hilly land down to the Kampili weir and goes down in the flat-lying land without any heavy meandering and pours into the Makassar Strait. The stretch of 20 km from the estuary to the Kampili weir which is objective stretch for the river improvement works is presented in Fig. 1-1.

The riverbed gradient of the Jeneberang is 1/1,400 and 1/2,100 in the upper and lower reaches of the Sungguminasa bridge respectively (refer to Fig. 1-2).

At present, the Jeneberang river splits at 4.4 K into right and left courses. Judging from the aerophotoes, before 1924 the river used to split into the right course in the north and the old Jeneberang course which is now closed. According to the interview, the left course was originated by the flood in 1967.

Embankment along the Jeneberang river, commenced in 1965, has been completed in the stretch from 2.0 K to 9.0 K in the left and from 2.6 K to 10.9 K on the right. However, there is no dike in the vicinity of 7.2 K of the left and also none in the vicinity of 8.0 K of the right. In addition, the dikes have insufficient safety because of inconsistency of cross-sections and elevation.

The Sungguminasa bridge is located at about 9.0 km from the estuary. The river narrows down considerably at the site of the bridge.

The Tallo river with a mean gradient of 1/10,000 is a so-called tidal river. The stretch under tidal influence extends more than 10.0 km from the estuary. The Tallo river, meandering down into the Makkasar Strait, is a natural river having no dike except for the partial compartment near the estuary (refer to Fig. 1-3 and 1-4).

Present Flow Capacity

The bankful flow capacities of the Jeneberang river in the upper and lower reaches of the Sungguminasa bridge are estimated at 600 m^3/s and 1,000 m^3/s respectively by non-uniform flow calculation. Fig. 1-5 shows the flow capacity of each section along the Jeneberang river. Table 1-1 shows the result of non-uniform flow calculation.

The flow capacity in the upper reaches of the Kampili weir is considered to be rather big because these reaches are surrounded by hilly lands. Therefore, the calculation of this flow capacity is not required in this study.

The section having the minimum flow capacity in the lower reaches of the Sungguminasa bridge is located near 8.0 K. This section has no dike as shown in Fig. 1-6. However, the road lying 1 m above the ground level on the right side serves as a dike. The discharge which inundates the right side and reaches the surface of the road is estimated at 1,800 m^3/s. Water flooding in the right side area will return to the river after flood.

In the upper reaches of the Sungguminasa bridge there exist a road on the right hand side and the Kampili irrigation channel on the left hand side, both of which are higher than the existing ground level. The area surrounded by the road and the channel plays the role of a natural buffer (refer to Fig. 1-7). The discharge, if over 1,800 m^3/s, will overtop the road and flow into the Bili-Bili irrigation channel on the right hand side.

The flow capacity of the Tallo river is so small as 50 m3/s at 18.0 K and 130 m^3/s at 9.0 K (refer to Fig. 1-8). The cross-sections of 9.0 K and 18.0 K are presented in Fig. 1-9.

Condition of Riverbed Fluctuation

The conditions of riverbed fluctuation are not exactly grasped, but based on the field investigation and interview, the following was judged.

The Jeneberang river used to have a narrow and deeper channel, and the riverbed at present is raised by the repeated hillside collapses in bygone years.

However, as mentioned in "Geology" in the foregoing of this report, the collapse are found stable in these years. It is, therefore, assumed that no serious sediment discharge has occured lately though sediments have deposited in some parts along the river course.

At present, riverbed materials consist of gravel at Bili-Bili and Kampili, and fine sand at Sumgguminasa.

Fig. 1-10 shows the 50% grain size of riverbed materials at each cross-section.

The grain size accumulation curve and specific gravity are given in Fig. 1-11.

In the Tallo river, practically no riverbed fluctuation could be observed as there is no appreciable sediment discharge.

Transition of Estuary

A Sand-bar has developed at the Jeneberang esturay due to sediment discharge from the upper reaches. Long before, a great deal of sediments passed through the old Jeneberang course, which is now closed, and deposited in the Makassar port. At present, the main channel of the Jeneberang river shifted to the south (left course) as a result of which no sediment transportation to the Makkasar port area is observed. (refer to Fig. 1-12)

Since the collapses in the upper reaches are found stable, serious sediment deposition is not observed at the estuary.

The total sediment yield since 1900 is estimated at about 60,000,000 m^3 judging from the comparative study between the results of the newly conducted surveying and the chart prepared by the Ministry of Marine, Netherlands in 1900 (refer to Fig. 1-13).

1.3 River Utilization

The Jeneberang river water is utilized for irrigation (Bili-Bili and Kampili intakes), drinking water (point 8.8 K), industry (point 15.2 K) and fish ponds at the estuary. At 3.6 K, people are ferried across the Jeneberang river. Fig. 1-14 shows the existing riparian facilities. Sand collection is briskly conducted in the lower reaches of the Sungguminasa bridge mainly during a dry season by the local populace.

The Jeneberang river water is decreased so much during dry season that irrigation water is available only for about 10% of all the irrigation area.

The Tallo river water is utilized as cooling water for the thermal power station at 13.0 K and for fish ponds at the estuary. The Tallo river, being tidal, has a sluggish gradient, and is used for boat transportation.

1.4 Flood Damage

The biggest flood was experienced in February of 1967. The flood discharge at Kampili weir is estimated at 3,350 m^3/s from the water stage records.

During the flood in 1967, the right bank in the upper reaches of the Sungguminasa bridge was overtopped at 9.5 K and the right bank collapsed at 3.0 K, resulting in a big damage in the land-side area thereof and southern part of the old Ujung Pandang city.

2 OPTIMUM SCALE OF RESERVOIR AND RIVER

2.1 Standard Project Flood

Flood discharge on a 50-year return period basis was employed for the formulation of an overall flood control plan, with due consideration to the improvement scales to other rivers in Indonesia.

Standard project flood at the following places are tabulated below.

Bili-Bili	:	2,750 m^3/s
Kampili	:	3,700 m^3/s
Sungguminasa	:	3,700 m^3/s

Standard project flood of 3,700 m^3/s at Sungguminasa will be shared both by an impounding reservoir and by the improved Jeneberang river channel.

2.2 Comparative Study

The comparative study to know the optimum scale of reservoir and river improvement is given as below.

<u>Comparison by Construction Cost</u>

1) Sole-purpose dam for flood control

   The total construction cost varies according to the share ratio as shown in Fig. 2-1.

   Judging from Fig. 2-1, the more the discharge is allocated to the river channel the higher the total construction cost becomes. This proves that flood control only by river improvement is most economical.

2) Multi-purpose dam

The cost /1 allocated to flood control capacity of a multi-purpose dam was considered for economical evaluation.

The total costs of river improvement and dam construction are almost the same at any allocation as illustrated in Fig. 2-2.

In the economic viewpoint, it is recommended that a multi-purpose dam be constructed rather than one for the sole-purpose of flood control.

Comparison by Land Acquisition

Broadening of the river width will be required to flow down more than 2,500 m^3/s by Jeneberang river, which results in a great deal of house evacuation and land acquisition (refer to Fig. 2-3 /2).

2.3 Determination of Optimum Scale

The total construction cost is almost the same as mentioned above.

Accordingly, to avoid the serious social problems which would result from land acquisition and house evacuation due to the river improvement. It is recommended that the discharge shared by the river channel to limit to 2,500 m^3/s. Accordingly, the overall flood control plan is formulated on the following conditions.

- Standard project flood at Kampili:	3,700 m^3/s
- Discharge controlled by a dam:	1,200 m^3/s
- Design discharge for river improvement:	2,500 m^3/s
- River improvement stretch:	Estuary to Kampili weir/3

/1 : The cost was calculated by the ratio of flood control capactiy to the total effective capacity.

/2 : Number of houses to be evacuated and area of land acquired for the construction of the proposed dam is in the cases of a dam solely for flood control.

/3 : The upper channel of Kampili weir has such a sufficient flow capacity that there is no need to facilitate river improvement.

3 RIVER IMPROVEMENT OF OVERALL FLOOD CONTROL

3.1 General Conception

The river improvement work has been planned with full consideration to the technical, social and economic aspects as mentioned below.

1) To control 50-year return period flood by means of the river improvement and the impounding reservoir.

2) To smooth the river course alignment in order to stabilize the proposed river channel.

3) To minimize land acquisition and house evacuation.

4) In principle, to keep the design high-water level as low as possible to reduce the damage potential.

5) To plan the flood control facilities to the extent possible to mitigate the damage caused by extraordinary flood.

6) To take the stability of the river channel into full consideration.

3.2 Design High-Water Level

It is advisable that the design high water level of the river channel be planned as low as possible to reduce damage potential. However, a completely-excavated channel requires much excavation and high construction cost. In principle, the design high water level in the target stretch from the estuary to Kampili weir will be fixed within 2 m above the ground level (refer to Fig. 3-1).

3.3 Alignment

A new alignment is proposed in the stretch of 20 km from the estuary to Kampili. In principle, the present river course will remain since the meandering is slight. The river width is proposed to be 265 m and 325 m in the upper and lower reaches of Sungguminasa respectively. Bends of the course will be widened more than the standard width to flow safely down the design flood.

It is considerably difficult to distribute the design flood as designed to the right and left course diverted at 4.4 K. The left course, the main channel of the Jeneberang at present, will be improved, while the right course will be closed. Fig. 3-2 shows the proposed alignment.

3.4 Longitudinal Profile

The longitudinal profile is proposed as shown in Fig. 3-1 to secure the channel stability, flow capacity of each cross-section, and maintenance of the estuary.

The proposed longitudinal gradients in the upper and lower reaches of the Sungguminasa bridge are 1/1,270 and 1/1,900 respectively. The ratio of river bed gradient between the upper and lower reaches is planned to be less than 1 : 2 to assure the stability of river channel. The average excavation depth is about 80 cm and 60 cm in the upper and lower reaches of the Sungguminasa bridge respectively. As the results, the water supply pumping station at 8.8 K and paper manufacturing pumping station at 15.2 K will be modified according to the river improvement works.

3.5 Cross-Section

Compound corss-section is superior to single cross-section, though more costly, in the viewpoint of channel stability. Normal discharge can flow down in the low-water channel, and flood discharge will pass in the whole cross-sectional area.

The standard cross-section shown in Fig. 3-3 can confine 950 m^3/s of 1.5-year return period discharge in the low-water channel and 2,500 m^3 in the whole cross-section area.

In principle, height of dike is fixed within 3.2 m above the ground level.

However, where the height of dike is required to be over 3.2 m above the ground level due to topographic conditions, a berm may have to be constructed to secure the safety of the dikes.

The section near the Sungguminasa bridge has an insufficient cross-section area to flow the design flood. To increase its flow capacity, the river width will be broadened. The Sungguminasa bridge will be extended in length according to the newly proposed river width.

The proposed corss-sections overlapping the existing cross-sections are shown in Fig. 3-4.

3.6 Stability of River Channel

Study of the river channel stability is made through calculation of tractive force at each cross-section of existing river channels and of the one proposed in the overall plan. (Refer to Fig. 3-5.)

In making a comparison of tractive force between the existing and the proposed river channels, as can be seen in Fig. 3-5, it is obvious that there are wide variations in tractive force over the entire length of the existing river channel. On the other hand, tractive force is practically steady all over the entire length of the channel except a slight difference between the upper and lower reaches of the Sumgguminasa bridge and a gradual decrease along the bends.

For keeping better stability of river channels, however, it is indispensable to manage and maintain the river as much as practically possible.

3.7 Required Earthwork

The required earthwork consisting of embankment as well as excavation of the channel and drainage ditches are given in Table 3-1.

Since the riverbed materials are not appropriate for embankment, the materials shall be conveyed from a borrow pit.

Table 3-1 EARTHWORK VOLUME

	Upper reaches of Sungguminasa Bridge	Lower reaches of Sungguminasa Bridge	Drainage Ditches
Embankment	850,000 m^3	480,000 m^3	
Excavation	2,900,000 m^3	1,600,000 m^3	150,000 m^3

The right course of Jeneberang is proposed as a spoil bank. In other words, riverbed materials to be dredged can be used for reclamation, but the course would still leave a rooom for drainage ditch. The reclamation volume of the right course is estimated at 1,200,000 m^3. The rest, 3,450,000 m^3 will be utilized for the reclamation of the lower land of project area and the lower-land along the Tallo river.

3.8 Land Acquisition and House Evacuation

The land acquisition and house evacuation for the implementation of the river improvement are tabulated below.

1) Land Acquisition

	Right side land	Left side land
Upper reaches of the sungguminasa bridge	22 ha	36 ha
Lower reaches of the Sungguminasa bridge	4 ha	8 ha

2) House Evacuation

	Right side land	Left side land
Upper reaches of the Sungguminasa bridge	50 houses	80 houses
Lower reaches of the Sungguminasa bridge	50 houses	40 houses

3.9 Riparian Structures

Dike

In principle, it is proposed that the height of the dike be no greater than 3.2 m, including a freeboard of 1.2 m. The proposed dikes should have a crest width of 3.0 m so that vehicles can pass along it for river management. The side-slope gradients of river-side and land-side are to be 1:2 to assure the dike stability. (Refer to Fig. 3-6.)

Where the crest height exceeds 3.2 m above the land-side ground level, berms will be provided 3.0 m in width for the following purposes; 1) protection of the dike against seepage water, 2) securing the slope stability, 3) better maintenance of the dikes and 4) easier flood defense activities. The berms are to be located every 2.0 m down from the crest.

Leakage of seepage water in the dike causes scouring along the toe of the land-side slope of the dike. Scouring due to this leakage is likely to bring about the breaking of dikes.

The causes of seepage water leakage are itemized as follows:

1) Paths of infiltration have developed.

2) Dike bodies which consist of coarse grain non-weathered soil, sand, and gravels do not have a watertight facing on the river-side slope or at the center wall.

3) Compaction of the dike body is insufficient.

4) Cracks in the dike body caused by earthquake.

5) Cross-section area of the dike is too small.

Clauses 1) to 4) in the above are related somehow to execution of construction works. Clause 5) is related to structural design works.

The stability of standard cross-section shown in Fig. 3-6 should be judged after observing the soil features at the borrow pit.

Drainage Ditch

The drainage ditch, now existing along the land-side toe of the dike, is to drain seepage water and land-side water to secure stability of the dike and also to protect the toe.

The major features of the drainage ditch are determined as below. However, due to lack of detailed topographic maps of the vicinity, the area of the basin and slope gradient of the channel are based on the less-detailed topographic maps (S=1/50,000). The cross-section of drainage ditches is shown in Fig. 3-6.

Basin area	:	3.0 km^2
Run-off	:	9.0 m^3/sec
Channel slope gradient	:	1:4,000
Roughness coefficient	:	0.025
Channel cross-section	:	12.2 m^2
Flow velocity	:	0.7 m/sec

Revetment

Wet masonry will be employed for revetment using cobbles which can be easily obtained at the project site. The revetment structure consists of high-water revetment, low-water revetment and foot protection. The foot protection will be placed at the base of the low-water revetment to prevent riverbed erosion (refer to Fig. 3-7).

Revetment will be constructed in the vicinity of the bridge and along the concave side of bends of the river totalling 11,100 m in length, as shown in Fig. 3-2.

The slopes of the dike which are not specially protected by revetment are sodded against erosion caused by rainfall and river water.

Groyne

The purpose of groyne is to prevent erosion at the foot of side-slope by slowing down flow velocity and by accelerating sediment deposit. Groyne will be provided at the concave side of a bend, which is less proof against erosion due to high flow velocity. Wooden pile permeable groyne will be employed to the project.

The possible directions of groyne include up-stream, down-stream and right-angle to the flow direction. The right-angled groyne can be recommended to achieve acceleration of sediment deposit.

Dimensions of the groyne are based on the empirical formulas, shown below, which are widely accepted in Japan.

$$\frac{L}{B} = 0.1$$

$$\frac{D}{L} = 1.5$$

$$\frac{D}{H} = 20$$

where;

B : width of river
L : length of groyne
D : interval of groynes
H : height of groyne

The structural dimensions and placement intervals of the groyne are determined by the formulas as shown above. However, the intervals may be modified after considering the river conditions such as degree of bend, flow velocity and so on. The structural details are shown in Fig. 3-8. The total length of sections where the groynes are to be placed is 4,300 m, and number of groynes are 86. The standard interval is determined by the formula above.

Groundsill

The groundsill is proposed to prevent erosion of the riverbed. Most of the sediment will be deposited in the Bili-Bili reservoir after construction of the Bili-Bili dam. The groundsill will be installed at 30 m downstream of the Sungguminasa bridge to protect its foundations. The groundsill, as shown in Fig. 3-9, is placed over the full width of the channel to maintain the required elevation and stability of the riverbed.

The elevation of the crest of groundsill is to be the same as the elevation of the proposed riverbed. Land-side ends of groundsills are to be embedded deep enough into the dike for security.

Aprons are to be constructed on both the up and the down ends of the concrete main body in order to protect the riverbed from scouring at these ends.

Concrete blocks are to be placed, also for protection against scouring, at the immediate down-stream end of the apron.

Sluice

As to the number of sluices, it is not desirable to install too many sluices from the viewpoint of safety of the dike especially at the time of flood.

Sluices will be required at the interval of 2 km on an average to drain the land-side water. Since detailed topographic maps of the vicinity are not readily available, the location of the sluices is determined from the less detailed topographic map (S = 1/50,000). The proposed location of the sluices are shown in Fig. 3-2.

The principal features of the proposed sluices are as follows.

Design conditions

Drainage area : $A = 3.0 \text{ km}^2$
Specific discharge : $q = 3.0 \text{ m}^3\text{/sec/km}^2$ (see Fig. 3-10)
Peak discharge : $Q = A \cdot q = 9 \text{ m}^3\text{/sec}$
Flow velocity of inside sluice : $V = 1.3 \text{ m/sec}$
Cross sectional area of sluice :

$$a = \frac{Q}{V} = \frac{9}{1.3} = 7 \text{ m}^2$$

Based on the above-mentioned design conditions, the structural details of the sluice are as shown below.

Dimensions of sluice

Height, inside :	1.9 m
Width, inside :	1.9 m
Type of sluice :	double-channel
Length :	16.0 m

The proposed sluice of reinforced concrete has a manually operated steel sluice gate. A cut-off wall will be provided at the center to interrupt the seepage water. Wooden sheet piles will also be driven to prevent infiltration from under the foundation. Fig. 3-11 illustrates the structure of the sluice.

<u>Sungguminasa Bridge</u>

The section of channel near the Sungguminasa bridge has an insufficient cross-section area to confine the design flood. To increase this flow capacity, the river width will be broadened. The Sungguminasa bridge will be extended in length according to the newly proposed river width.

Since the bridge is still quite new and allows a sufficient clearance between the girder and the design high water level, the bridge is to be modified in such a manner that the exisiting bridge ends will be extended to the both banks of the river in the same longitudinal graditent as that of the existing bridge surface. Fig. 3-12 shows the proposed longitudinal section of the proposed bridge.

4. RIVER IMPROVEMENT OF URGENT FLOOD CONTROL

4.1 General Conception

The river improvement work has been planned with full consideration to the technical, social and economic aspects as mentioned below.

1) This urgent plan has adapted the overall plan to avoid extreme an amendment.

2) To avoid the social problems arising from land acquisition and house evacuation due to the river improvement.

3) In shaping the standard cross-section, the cross-section of the existing river cahnnel will be used provided the section has a sufficient flow capacity for design flood.

4) In principle, to keep the design high-water level as low as possible to reduce the damage potential.

5) To plan the flood control facilities to the extent effective to mitigate the damage caused by extraordinary flood.

4.2 Scale and Stretch of Urgent River Improvement

To determine the scale and the stretch of river improvement a comparative study was conducted as shown in Table 4-1. The relation between flow capacity and earthwork volume was also studied for some combinations of dike and riverbed elevation (refer to Fig. 4-1).

Table 4-1 COMPARATIVE STUDY OF RIVER IMPROVEMENT

Improvment Scale (return period)	Improvement Stretch	Dike Height	Longitudinal Profile *
5-year	Estuary to Kampili weir	2.8 m	Case A
5-year	Estuary to the bridge	2.2 m	Case A
7-year	Estuary to Kampili weir	3.0 m	Case B
7-year	Estuary to the bridge	2.5 m	Case A
10-year	Estuary to Kampili weir	3.0 m	Case C
10-year	Estuary to the bridge	3.0 m	Case A

* Refer to Fig. 4-1

Based on the comparative study, the improvement scale and stretch are proposed as shown below:

River improvement scale	:	10-year return period
Design flood	:	2,100 m^3/s at the Sungguminasa bridge
River improvement stretch	:	Estuary to the Sungguminasa bridge

The following is the results of comparative study.

1) The value of damage potential to property of all types in the lower reaches of the bridge is far greater than that in its upper reaches. (The primary purpose of flood control is to protect Ujung Pandang city located in the lower reaches.)

2) The each work volume of the river improvement from the estuary to Kampili weir is extremely big, compared with that in the lower reaches. Earthwork volume of river improvement from the estuary to the bridge between 5- and 10-year return periods do not show much difference (refer to Table 4-2).

3) The reaches between the Sungguminasa bridge and Kampili weir have a function as a natural buffer, which is effective to control a flood.

4.3 Design High-Water Level

It is advisable that the design high-water level of the river channel be planned as low as possible to reduce damage potential.

Moreover, the design high-water level proposed in the plan better be set at lower evaluation, from the economic viewpoint.

In principle, the design high-water level is kept within 2 m above the ground level in order to lower the damage potential (refer to Fig. 4-2).

4.4 Alignment

The stretch that requires alignment extends for about 7 km from the down reaches of the Sungguminasa bridge to the lower end of the existing dike at 2.0 K.

Although inundation will occur in the lower reaches below 2.0 K by any flood over 800 m^3/s, new dikes are not proposed in this stretch since 1) the overtopped water does not flow into Ujung Pandang city and 2) the reaches have few assets.

The stretch of 7 km has been almost embanked with little meandering. Though the inconsistent width between the existing dikes influence adversely on the channel stability, the present alignment remains to prevent a social problem derived from house evacuation and land aquisition. The proposed alignment is shown in Fig. 4-3.

4.5 Longitudinal Profile

The longitudinal profile is designed at 1/1,900 based on the general conception described in page 12 and by taking into consideration the stability of the low water channel. The proposed longitudinal profile is shown in Fig. 4-2.

4.6 Cross-Section

It is economical that the cross-section having a sufficient flow capacity remains. However, to stabilize the channel against the low-water discharge, only the low-water channel will be prepared for the urgent plan. In other words, only the riverbed will be excavated down to the proposed riverbed stage.

The Jeneberang river splits at 4.4 K into the right and left courses. In the lower reaches of 4.4 K only the left course regarded as a main course of the Jeneberang will be excavated. The right course will be left intact.

The proposed cross-sections overlapping the existing cross-sections are shown in Fig. 4-4.

4.7 Natural Buffer

The minimum bankful discharge between Sungguminasa and Kampili is estimated at 600 m^3/s. Under the existing conditions, the discharge over 600 m^3/s overflows the channel at various points. The area surrounding the Jl. Malino and the Kampili irrigation channel can confine a discharge of 2,100 m^3/s, which decreases to 1,800 m^3/s at the Sungguminasa due to overflowing on the way. The discharge over 2,100 m^3/s will flow into the Bili-Bili irrigation channel over Jl. Malino and inflict damage on Ujung Pandang city.

Jl. Malino lying between the Bili-Bili irrigation channel and the river channel will be raised so that the 10-year return period flood of 2,500 m^3/s will not flow into the irrigation channel. Accordingly, the discharge in the natural buffer is regulated to be 2,100 m^3/s.

The capacity of the area is 13.0 x 10^6 m^3. Figs. 4-5, 4-6 and 4-7 show the location of the inundation area, the stretch, the longitudinal profile and the cross-section of the road to be raised.

4.8 Study on Adverse Influence due to River Improvement

<u>Inundaion Area above the Sungguminasa Bridge</u>

By raising the road, a negative benefit will occur during a big flood because the inundation water stage in this area will be increased. On the other hand, by excavating the low-water channel in the lower reaches of the Sungguminasa bridge, a positive benefit will occur during small floods. These negative and positive benefits counter-balance each other.

<u>The Lower Reaches of the Jeneberang River</u>

Flood discharge will increase from 1,800 m^3/s to 2,100 m^3/s after river improvement, which may influence the water stage in the lower reaches of 2.0 K.

However, the riverbed will be excavated to maintain the channel stability. As a result, the water stage in the lower reaches will not exert any harmful influence on it as shown in Fig. 4-8 and Table 4-3.

Table 4-3 VARIATION OF WATER LEVEL AT 2.0 K

		Discharge (m^3/s)	Water Level (m)
Left Jeneberang river	Before Improvement	950	3.13
	After Improvement	1,330	3.11
Right Jeneberang river	Before Improvement	850	3.20
	After Improvement	770	3.05

Land-side Water

The area has suffered from land-side water damage. The proposed raising of the existing dikes will not causes new land-side damage. On the other hand, new drainage will be provided along the dikes. After the completion of the plan, therefore, the land-side damage will be mitigated due to the improved drainage ditch system.

4.9 Required Earthwork

The excavation volume of the river channel is estimated at 800,000 m^3. At present, sand is collected from the Jeneberang riverbed in a range of 200,000 m^3/year to 300,000 m^3/year.

Any sand excavated during the project construction period can therefore be sold in a similar way. The earthwork necessary to improve the Jeneberang river are summarized in Table 4-4.

Table 4-4 EARTHWORK VOLUME

Works	Volume (m^3)
Embankment	95,000
Road Raising	16,000
River Channel Excavation	794,000
Drainage Ditch Excavation	12,000

4.10 Land Acquisition and House Evacuation

The following shows number of houses to be evacuated and areas of land to be acquired in accordance with the progress of the proposed river improvement project. Since houses do not lie densely along the channel of the Jeneberang river, number of houses required to be evacuated is only 60 in total, and also because of no widening of the river channel is involved, land to be aquired is about 5 ha in total.

1) Land Acquisition

	Right side land	Left side land
Lower reaches of the Sungguminasa bridge	2 ha	3 ha

2) House Evacuation

	Right side land	Left side land
Lower reaches of the Sungguminasa bridge	40 houses	20 houses

4.11 Riparian Structures

Dike

The existing dikes in the lower reaches of the Sungguminasa bridge are to be raised in the urgent flood control plan.

The structure of embankment is the same as that of the overall plan; i.e., freeboard is 1.2 m, width of the crest of the dike is 3.0 m, and slope gradient of both river- and land-sides is 1:2. Where the crest height exceeds 3.2 m above the land-side ground level, berms with a width of 3.0 m will be constructed at intervals of 2.0 m (refer to Fig. 3-6).

Drainage Ditch

In complying with the policy of the overall plan, two drainge ditches with the total length of 2,200 m will be constructed between 5.4 K and 7.6 K along the left bank (refer to Fig. 4-9).

The major features of the above-mentioned ditches are given below.

Drainage area :	0.32 km^2
Run-off :	2.81 m^3/sec
Channels slope gradient :	1:4,000
Roughness coefficient :	0.025
Channel corss section :	5.44 m^2
Flow velocity :	0.6 m/sec

The existing sluices Nos. 1 and 2 will be removed in this project on the ground mentioned in "Sluice" below. Inundation water of the area will be drained through the simply excavated ditch into the newly constructed Jongaya channel. The cross-secction of the ditch is shown in Fig. 4-9. The location of the channel and ditch mentioned above is shown in Fig. 4-10. Excavation of the drainage ditch will be in accordance with the cross-section shown in Fig. 4-10.

The major features of this ditch are as follows.

Drainage area :	1.0 km^2
Run-off :	4.0 m^3/sec
Channel slope gradient :	1:2,000
Roughness coefficient :	0.025
Channel cross-section :	5.44 m^2
Flow velocity :	0.8 m/s

Revetment

Following the policy of the overall plan, revetment will be provided at the concave side of bends in a stretch of 5,700 m in total. The location and structure are presented in Figs. 4-3 and 3-7.

In the stretch of the dike where no revetment is applied, sodding will be provided to protect the slopes from erosion caused by rainfall and river water.

Groyne

The purpose and structure of the groyne are the same as these of the overall plan.

Twenty-three groynes will be provided at the concave side of bends in a total stretch of 1,000 m to protect the revetment. Log pile permeable groyne which prevents erosion is proposed in the improvement plan (refer to Fig. 4-3).

Demensions of the proposed groyne are 30 m in length, 2.3 m in height and 45 m in interval (refer to Fig. 4-11).

Sluice

Some of the existing sluices will be left intact, and others will be renewed or removed for the following reasons.

1) The existing sluices have flap gates, which open by the pressure of the inundation water. In other words, inundation is unavoidable to a certain extent.

2) Sluices installed at short intervals have an adverse influence on the safety of dike.

Along the Jeneberang river down-stream of the Sunguminasa bridge, there are 5 sluices right and left each (refer to Fig. 1-14).

Out of the above, the sluices Nos. 1 and 2 on the right bank will be removed because the proposed Jongaya drainage channel, can fulfill their function.

The sluice No. 3 existing on the right side has a flap gate. This will be replaced by a new one with a manual control gate to drain the land-side water.

The sluices Nos. 4 and 5 constructed recently on the right side need no gate, since the elevation of the gatesills is high. These sluices will be used in the future.

On the left bank, five sluices exist at short intervals in the section from 5.0 K to 7.5 K, and are not equipped with gates. These sluices will be removed and replaced by two new sluices with gates.

The location and structure of the proposed sluices are shown in Figs. 4-3, 4-12 and 4-13 respectively. The structure of the sluices are the same as that in the overall plan. The design conditions of the newly proposed sluices are presented in Table 4-5.

Road Raising

The structure of the road to be raised consists of subgrade, sub-base course, base course and pavement. The subgrade is an embankment with a slope gradient of 1:2, and consists of soil from the borrow-pit. Materials for the sub-base course will be crushed stones and gravels, both of which are available in the neighborhood of the construction site. For the base course, unscreened and crushed stones will be used (refer to Fig. 4-14). Asphalt-concrete will be employed for pavement.

Table I-1 THE RESULT OF NON-UNIFORM FLOW CALCULATION

(Existing channel)

NO	H	A	R	V	N	Q	DX	FROUD	IE
4.400	2.900	430.4	0.894	1.394	0.035	600.0	366.00	0.4711E 00	0.2767E-02
4.800	3.621	823.2	2.399	0.729	0.035	600.0	432.00	0.1503E 00	0.2026E-03
5.200	3.720	753.1	2.175	0.797	0.035	600.0	440.00	0.1726E 00	0.2759E-03
5.600	3.884	560.1	2.026	1.071	0.035	600.0	468.00	0.2404E 00	0.5483E-03
6.000	4.103	603.1	2.234	0.995	0.035	600.0	435.00	0.2126E 00	0.4151E-03
6.400	4.337	546.6	2.099	1.098	0.035	600.0	512.00	0.2421E 00	0.5494E-03
6.800	4.608	598.1	2.272	1.003	0.035	600.0	539.00	0.2126E 00	0.4126E-03
7.200	4.834	469.6	2.277	1.278	0.035	600.0	483.00	0.2705E 00	0.6677E-03
7.600	5.115	577.8	2.485	1.038	0.035	600.0	472.00	0.2104E 00	0.3924E-03
8.000	5.321	565.9	2.305	1.060	0.035	600.0	495.00	0.2231E 00	0.4522E-03
8.300	5.450	443.6	2.225	1.353	0.035	600.0	275.00	0.2896E 00	0.7714E-03
8.400	5.573	461.3	2.219	1.301	0.035	600.0	155.00	0.2789E 00	0.7162E-03
8.800	5.898	504.0	2.309	1.191	0.035	600.0	481.00	0.2503E 00	0.5689E-03
9.000	6.109	323.6	2.268	1.854	0.035	600.0	328.00	0.3933E 00	0.1413E-02
9.100	6.261	435.6	3.398	1.377	0.035	600.0	73.00	0.2229E 00	0.3789E-03
9.400	6.417	259.8	2.287	2.310	0.035	600.0	274.00	0.4878E 00	0.2168E-02
9.600	6.934	358.7	2.935	1.673	0.035	600.0	251.00	0.3119E 00	0.8158E-03
9.800	7.146	425.9	2.964	1.409	0.035	600.0	236.00	0.2659E 00	0.5978E-03
10.000	7.270	378.0	2.915	1.587	0.035	600.0	230.00	0.2970E 00	0.7411E-03
10.400	7.562	458.9	2.730	1.307	0.035	600.0	381.00	0.2528E 00	0.5489E-03
10.800	7.805	400.6	2.609	1.498	0.035	600.0	416.00	0.2962E 00	0.7654E-03
11.200	8.202	381.2	2.314	1.571	0.035	600.0	467.00	0.3299E 00	0.9881E-03
11.600	8.504	584.9	2.625	1.026	0.035	600.0	332.00	0.2023E 00	0.3561E-03
12.000	8.621	515.3	1.989	1.164	0.035	600.0	263.00	0.2637E 00	0.6640E-03
12.400	8.879	673.8	2.126	0.890	0.035	600.0	444.00	0.1951E 00	0.3552E-03
12.800	9.043	574.6	2.011	1.044	0.035	600.0	409.00	0.2352E 00	0.5263E-03
13.200	9.338	497.6	1.710	1.206	0.035	600.0	452.00	0.2945E 00	0.8706E-03
13.600	9.686	508.2	2.036	1.181	0.035	600.0	450.00	0.2643E 00	0.6617E-03
14.000	10.020	479.3	1.733	1.252	0.035	600.0	433.00	0.3038E 00	0.9225E-03
14.400	10.310	519.2	1.973	1.156	0.035	600.0	350.00	0.2628E 00	0.6613E-03
14.800	10.612	372.8	2.129	1.609	0.035	600.0	409.00	0.3524E 00	0.1159E-02
15.200	11.050	403.2	2.335	1.488	0.035	600.0	410.00	0.3111E 00	0.8757E-03
15.300	11.030	340.9	2.262	1.760	0.035	600.0	28.00	0.3738E 00	0.1278E-02
15.600	11.449	480.9	2.564	1.248	0.035	600.0	365.00	0.2489E 00	0.5435E-03
16.000	11.815	422.1	1.561	1.422	0.035	600.0	410.00	0.3634E 00	0.1367E-02
16.400	12.186	552.4	1.797	1.086	0.035	600.0	320.00	0.2588E 00	0.6614E-03
16.800	12.397	625.9	2.247	0.959	0.035	600.0	375.00	0.2043E 00	0.3826E-03
17.200	12.613	529.2	1.522	1.134	0.035	600.0	370.00	0.2935E 00	0.8990E-03
17.600	12.930	499.1	1.900	1.202	0.035	600.0	395.00	0.2786E 00	0.7526E-03
18.000	13.219	452.2	2.311	1.327	0.035	600.0	420.00	0.2788E 00	0.7060E-03
18.400	13.498	351.0	2.556	1.709	0.035	600.0	410.00	0.3350E 00	0.9779E-03
18.800	13.855	334.2	3.290	1.795	0.035	600.0	420.00	0.3161E 00	0.8067E-03
19.200	14.241	374.7	2.839	1.601	0.035	600.0	440.00	0.3036E 00	0.7815E-03
19.600	14.602	426.0	1.756	1.409	0.035	600.0	340.00	0.3396E 00	0.1148E-02
20.000	15.061	432.1	1.380	1.389	0.035	600.0	340.00	0.3775E 00	0.1537E-02
20.400	18.411	180.7	1.757	3.320	0.035	600.0	411.00	0.8000E 00	0.6366E-02

NO	H	A	R	V	N	Q	DX	FROUD	IE
4.400	3.380	665.8	1.328	1.502	0.035	1000.0	366.00	0.4164E 00	0.1894E-02
4.800	3.932	937.8	2.686	1.066	0.035	1000.0	432.00	0.2078E 00	0.3731E-03
5.200	4.106	886.6	2.542	1.128	0.035	1000.0	440.00	0.2259E 00	0.4491E-03
5.600	4.357	700.2	2.329	1.427	0.035	1000.0	468.00	0.2986E 00	0.8076E-03
6.000	4.669	772.5	2.738	1.294	0.035	1000.0	435.00	0.2499E 00	0.5358E-03
6.400	4.958	732.2	2.624	1.366	0.035	1000.0	512.00	0.2694E 00	0.6316E-03
6.800	5.272	778.5	2.900	1.285	0.035	1000.0	539.00	0.2410E 00	0.4888E-03
7.200	5.527	630.1	2.327	1.587	0.035	1000.0	483.00	0.3015E 00	0.7720E-03
7.600	5.861	756.2	3.169	1.322	0.035	1000.0	472.00	0.2373E 00	0.4602E-03
8.000	6.095	754.9	3.055	1.325	0.035	1000.0	495.00	0.2421E 00	0.4849E-03
8.300	6.216	601.4	2.921	1.663	0.035	1000.0	275.00	0.3108E 00	0.8110E-03
8.400	6.351	625.5	2.856	1.599	0.035	1000.0	155.00	0.3022E 00	0.7726E-03
8.800	6.703	680.4	3.041	1.470	0.035	1000.0	481.00	0.2693E 00	0.6008E-03
9.000	6.899	417.4	2.762	2.396	0.035	1000.0	328.00	0.4605E 00	0.1814E-02
9.100	7.102	521.6	4.600	1.917	0.035	1000.0	73.00	0.2855E 00	0.5884E-03
9.400	7.254	355.3	2.985	2.814	0.035	1000.0	274.00	0.5203E 00	0.2257E-02
9.600	7.900	548.0	3.282	1.825	0.035	1000.0	251.00	0.3218E 00	0.8365E-03
9.800	8.113	625.5	3.429	1.599	0.035	1000.0	236.00	0.2758E 00	0.6055E-03
10.000	8.252	598.5	3.229	1.671	0.035	1000.0	230.00	0.2970E 00	0.7166E-03
10.400	8.535	673.4	3.099	1.485	0.035	1000.0	381.00	0.2694E 00	0.5977E-03
10.800	8.777	602.6	3.208	1.659	0.035	1000.0	416.00	0.2960E 00	0.7131E-03
11.200	9.158	561.3	2.730	1.782	0.035	1000.0	467.00	0.3444E 00	0.1019E-02
11.600	9.479	812.8	3.472	1.230	0.035	1000.0	332.00	0.2109E 00	0.3526E-03
12.000	9.580	760.7	2.929	1.315	0.035	1000.0	263.00	0.2453E 00	0.5050E-03
12.400	9.796	963.8	3.007	1.038	0.035	1000.0	444.00	0.1911E 00	0.3036E-03
12.800	9.926	825.9	2.867	1.211	0.035	1000.0	409.00	0.2284E 00	0.4410E-03
13.200	10.156	736.4	2.490	1.358	0.035	1000.0	452.00	0.2749E 00	0.6692E-03
13.600	10.447	700.8	2.669	1.427	0.035	1000.0	450.00	0.2790E 00	0.6738E-03
14.000	10.759	682.3	2.453	1.466	0.035	1000.0	433.00	0.2990E 00	0.7956E-03
14.400	11.023	707.0	2.661	1.414	0.035	1000.0	350.00	0.2770E 00	0.6646E-03
14.800	11.309	507.6	2.724	1.970	0.035	1000.0	409.00	0.3813E 00	0.1250E-02
15.200	11.807	544.2	2.812	1.838	0.035	1000.0	410.00	0.3500E 00	0.1042E-02
15.300	11.758	452.2	2.854	2.211	0.035	1000.0	28.00	0.4181E 00	0.1480E-02
15.600	12.275	637.3	3.318	1.569	0.035	1000.0	365.00	0.2752E 00	0.6096E-03
16.000	12.608	651.7	2.237	1.534	0.035	1000.0	410.00	0.3278E 00	0.9861E-03
16.400	12.904	778.8	2.433	1.284	0.035	1000.0	320.00	0.2630E 00	0.6173E-03
16.800	13.110	823.3	2.943	1.215	0.035	1000.0	375.00	0.2262E 00	0.4286E-03
17.200	13.311	772.2	2.203	1.295	0.035	1000.0	370.00	0.2787E 00	0.7165E-03
17.600	13.587	740.6	2.334	1.350	0.035	1000.0	395.00	0.2823E 00	0.7215E-03
18.000	13.877	601.8	2.668	1.662	0.035	1000.0	420.00	0.3250E 00	0.9141E-03
18.400	14.212	445.7	3.143	2.243	0.035	1000.0	410.00	0.4042E 00	0.1339E-02
18.800	14.692	421.0	3.946	2.375	0.035	1000.0	420.00	0.3820E 00	0.1109E-02
19.200	15.229	508.3	3.526	1.967	0.035	1000.0	440.00	0.3347E 00	0.8835E-03
19.600	15.610	699.8	2.463	1.429	0.035	1000.0	340.00	0.2908E 00	0.7520E-03
20.000	15.900	705.3	2.051	1.418	0.035	1000.0	340.00	0.3162E 00	0.9447E-03
20.400	19.156	255.6	2.442	3.913	0.035	1000.0	411.00	0.8000E 00	0.5705E-02

LEGEND NO: Station No. H: M.S.L.m A: Area(m^2) R: Hydraulic Radius(m) V: Velocity(m/s) N: Roughness Coefficient Q: Discharge(m^3/s) DX: Distance(m) FROUD: Froud No. IE: Energy Gradient

Table 4 2 EARTHWORK VOLUME OF RIVER IMPROVEMENT

i) 5 - year return period

Improvement reaches	Design discarge	Height of dike	Lower reaches Excavetion	Lower reaches Embankment	Upper reaches Excavation	Upper reaches Embankment	Total Excavation	Total Embankment	* Excavation *
Estuary to the Kampili weir	2,000 m^3/s	2.8 m	1,200,000 m^3	80,000 m^3	2,200,000 m^3	750,000 m^3	3,400,000 m^3	830,000 m^3	Case A
Estuary to the bridge	1,700 m^3/s	2.2 m	800,000 m^3	60,000 m^3	-	-	800,000 m^3	60,000 m^3	Case A

* Note : Refer to Fig. 4-1

ii) 7 - year return period

Improvement reaches	Design discarge	Height of dike	Lower reaches Excavetion	Lower reaches Embankment	Upper reaches Excavation	Upper reaches Embankment	Total Excavation	Total Embankment	* Excavation *
Estuary to the Kampili weir	2,300 m^3/s	3.0 m	1,400,000 m^3	100,000 m^3	2,600,000 m^3	850,000 m^3	4,000,000 m^3	950,000 m^3	Case B
Estuary to the bridge	1,900 m^3/s	2.5 m	800,000 m^3	80,000 m^3	-	-	800,000 m^3	80,000 m^3	Case A

* Note : refer to Fig. 4-1

iii) 10 - year return period

Improvement reaches	Design discarge	Height of dike	Lower reaches Excavetion	Lower reaches Embankment	Upper reaches Excavation	Upper reaches Embankment	Total Excavation	Total Embankment	* Excavation *
Estuary to the Kampili weir	2,500 m^3/s	3.0 m	1,600,000 m^3	100,000 m^3	2,900,000 m^3	850,000 m^3	4,500,000 m^3	950,000 m^3	Case C
Estuary to the bridge	2,100 m^3/s	3.0 m	800,000 m^3	100,000 m^3	-	-	800,000 m^3	100,000 m^3	Case A

* Note : refer to Fig. 4-1

Table 4-5 DESIGN CONDITIONS OF NEW SLUICES

	Right Bank	Left Bank
Catchment Area	0.15 km^2	0.32 km^2
Specific Run-off	10.0 $m^3/s/km$	8.5 $m^3/s/km$
Discharge	1.5 m^3/s	2.8 m^3/s
Flow Velocity	1.3 m/s	1.3 m/s
Cross-sectional Area	1.15 m^2	2.16 m^2

Fig. 1-1 GENERAL MAP OF JENEBERANG RIVER COURSE

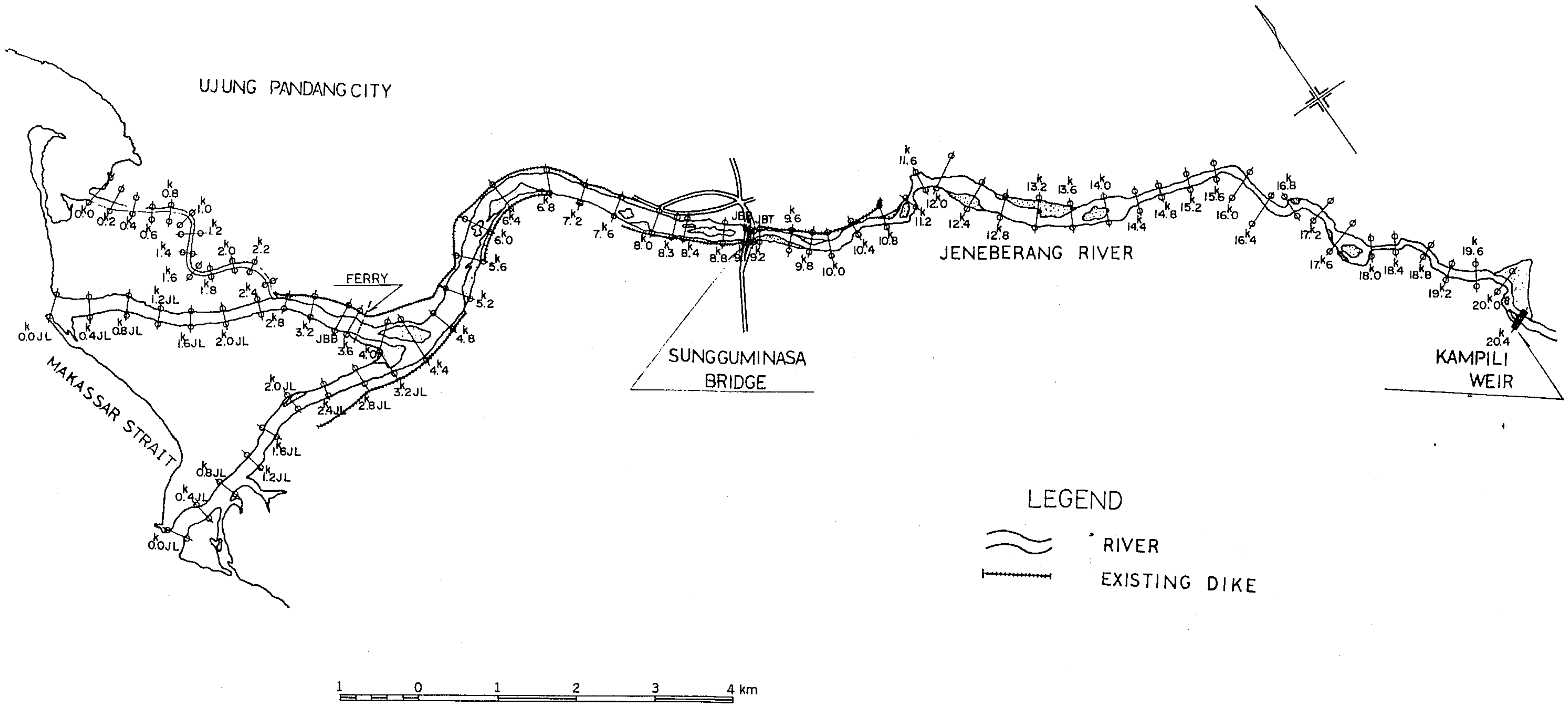


Fig. 1-2 LONGITUDINAL PROFILE OF JENEBERANG RIVER
(EXISTING CONDITION)

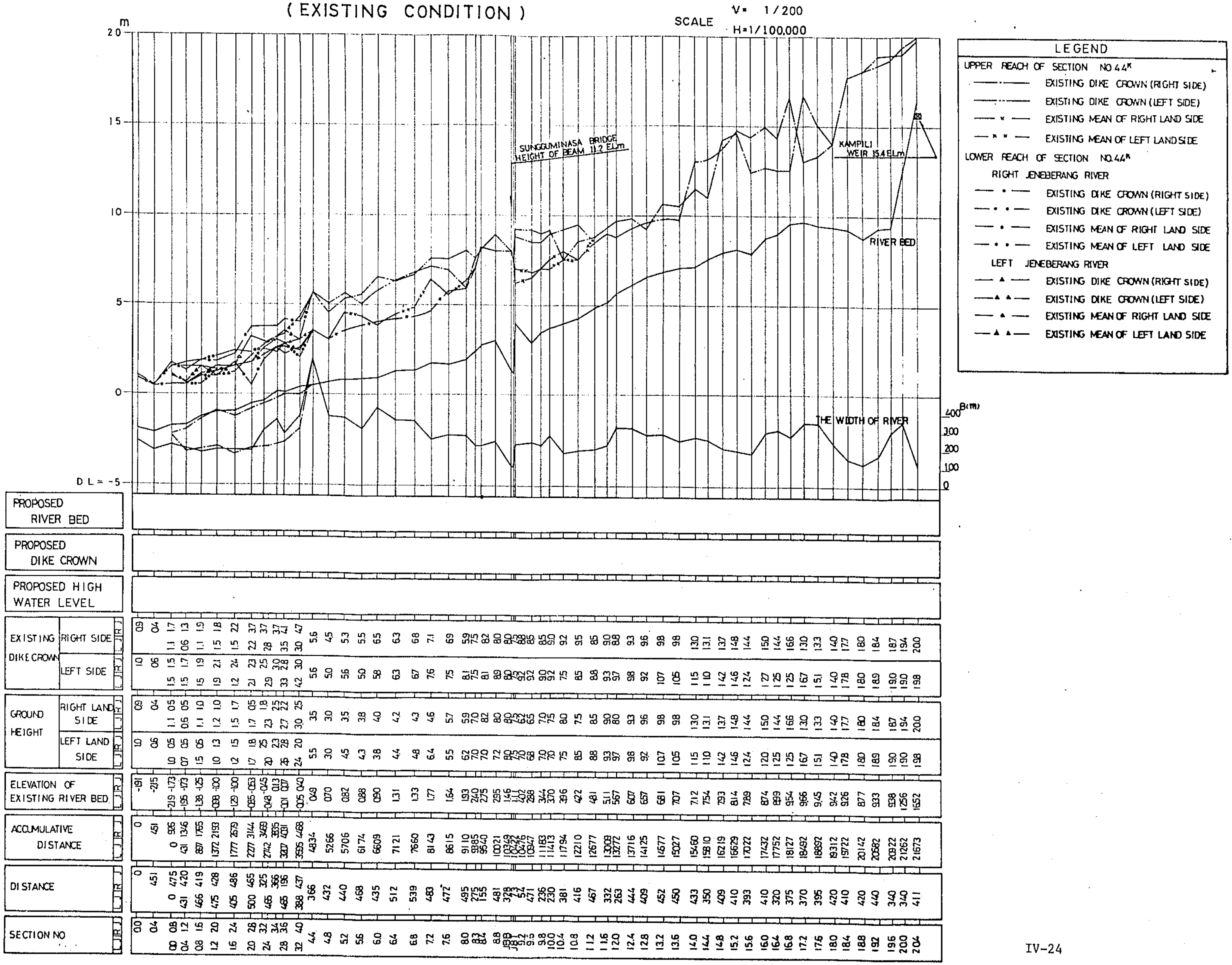


Fig. 1-3 GENERAL MAP OF TALLO RIVER COURSE

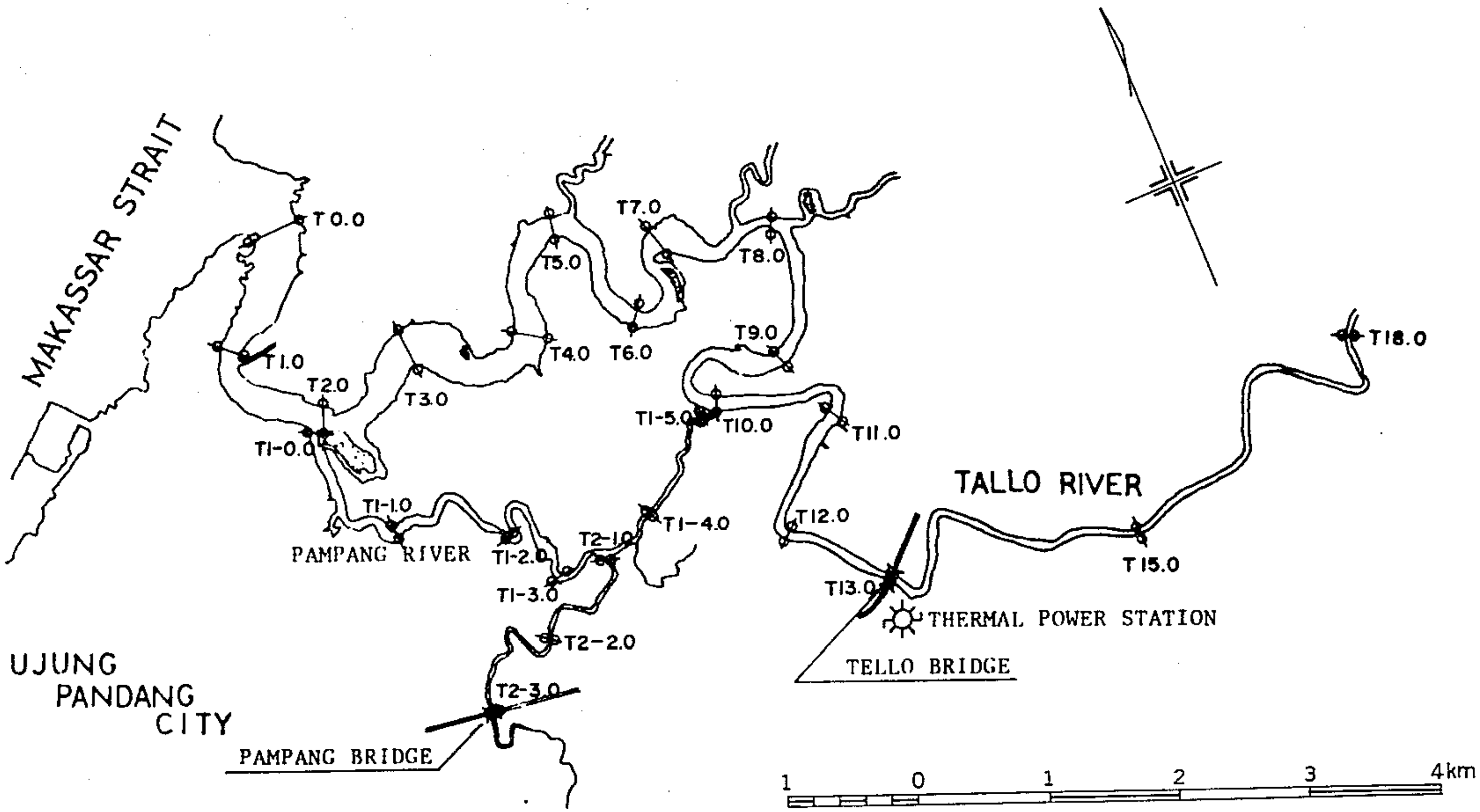
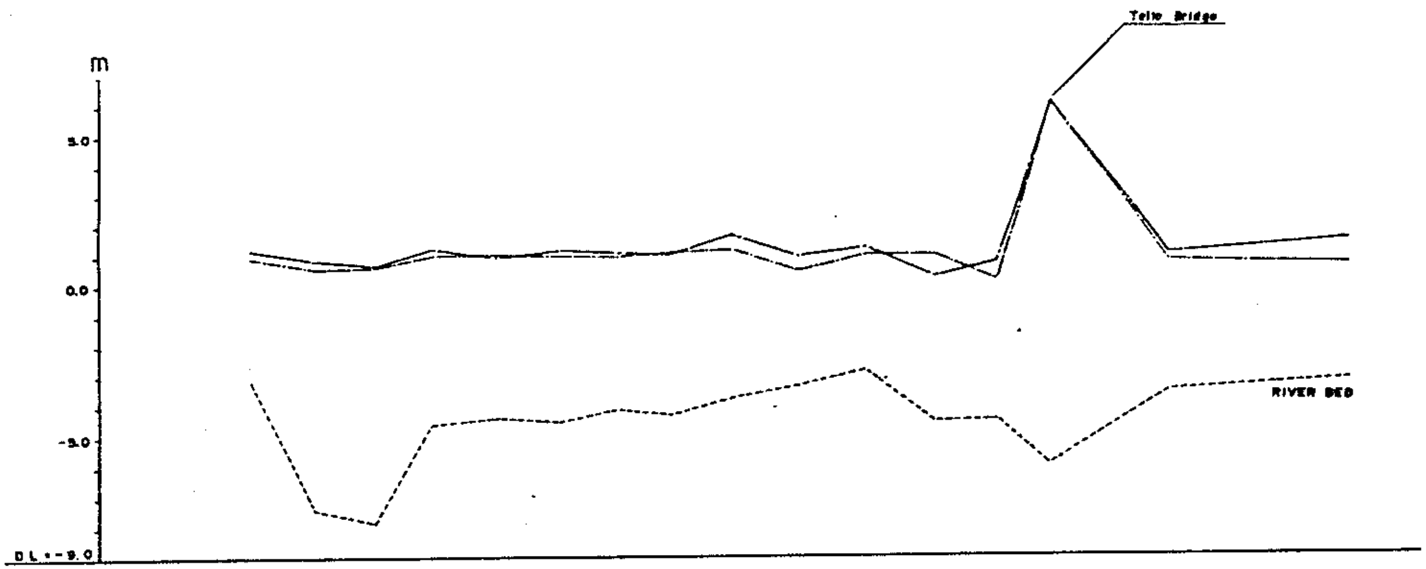


Fig.1-4 LONGITUDINAL PROFILE OF TALLO RIVER

(EXISING CONDITION)



Station NO	Dist	Accum Dist	GH Right	GH Left	Height of River bed
0.0	0.00	0.00	1.19	0.81	-3.14
1.0	[illegible]	[illegible]	0.52	0.56	-7.40
2.0	[illegible]	[illegible]	0.66	0.60	-7.67
3.0	[illegible]	[illegible]	1.20	1.00	-4.42
4.0	[illegible]	[illegible]	0.82	1.00	-4.36
5.0	[illegible]	[illegible]	1.14	0.98	-4.51
6.0	[illegible]	[illegible]	1.08	0.83	-4.10
7.0	[illegible]	[illegible]	1.00	1.06	-4.30
8.0	[illegible]	[illegible]	1.65	1.19	-3.78
9.0	[illegible]	[illegible]	0.53	0.46	-3.37
10.0	[illegible]	[illegible]	1.22	1.00	-2.81
11.0	[illegible]	[illegible]	0.36	1.00	-4.32
12.0	[illegible]	[illegible]	0.74	0.12	-4.49
13.0	[illegible]	[illegible]	6.09	6.13	-5.97
15.0	[illegible]	[illegible]	1.00	0.78	-3.52
18.0	[illegible]	[illegible]	1.90	0.63	-3.16

Fig. 1-5 PRESENT FLOW CAPACITY OF JENEBERANG RIVER

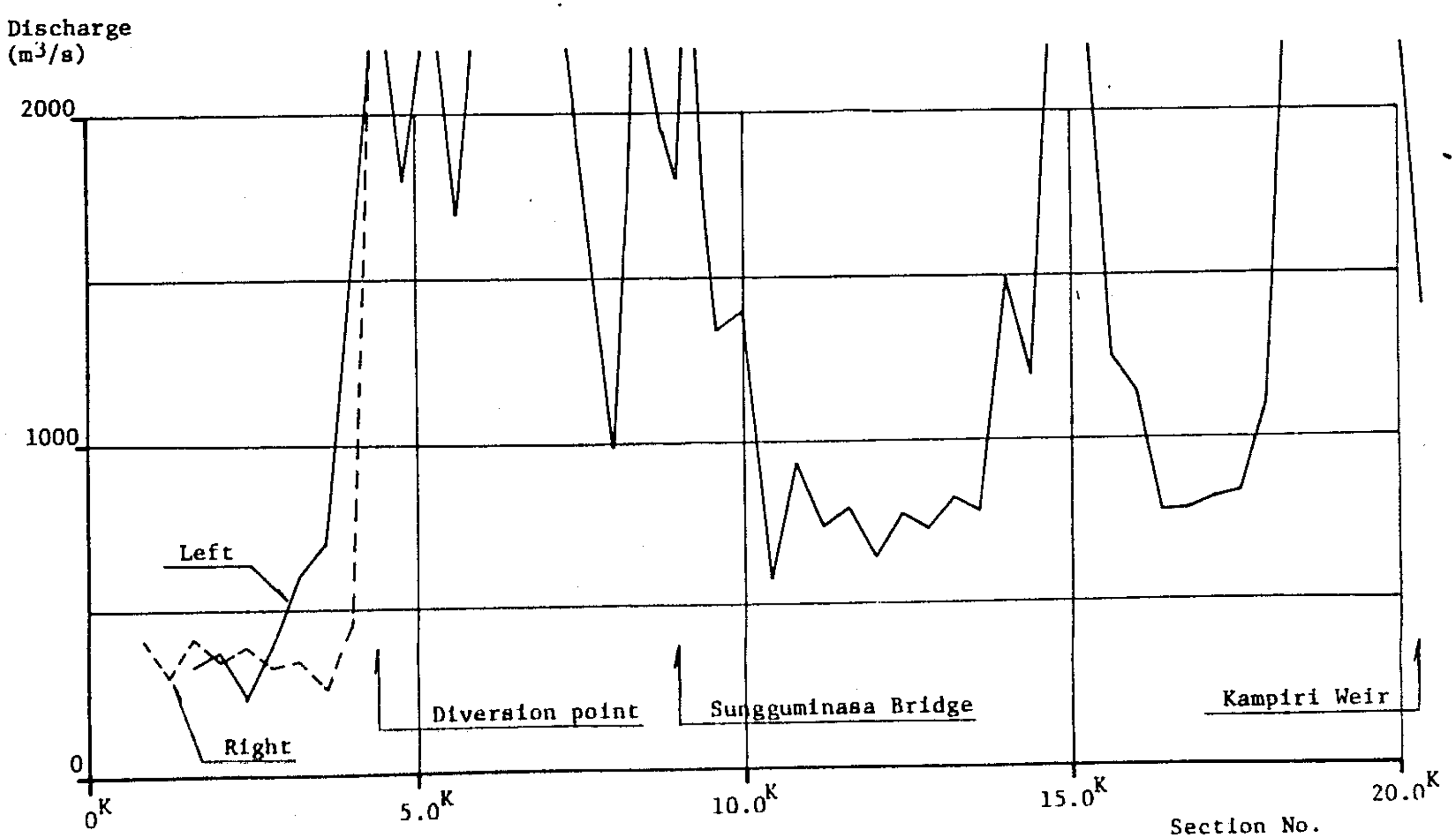


Fig. I-6 CROSS-SECTION OF JENEBERANG RIVER

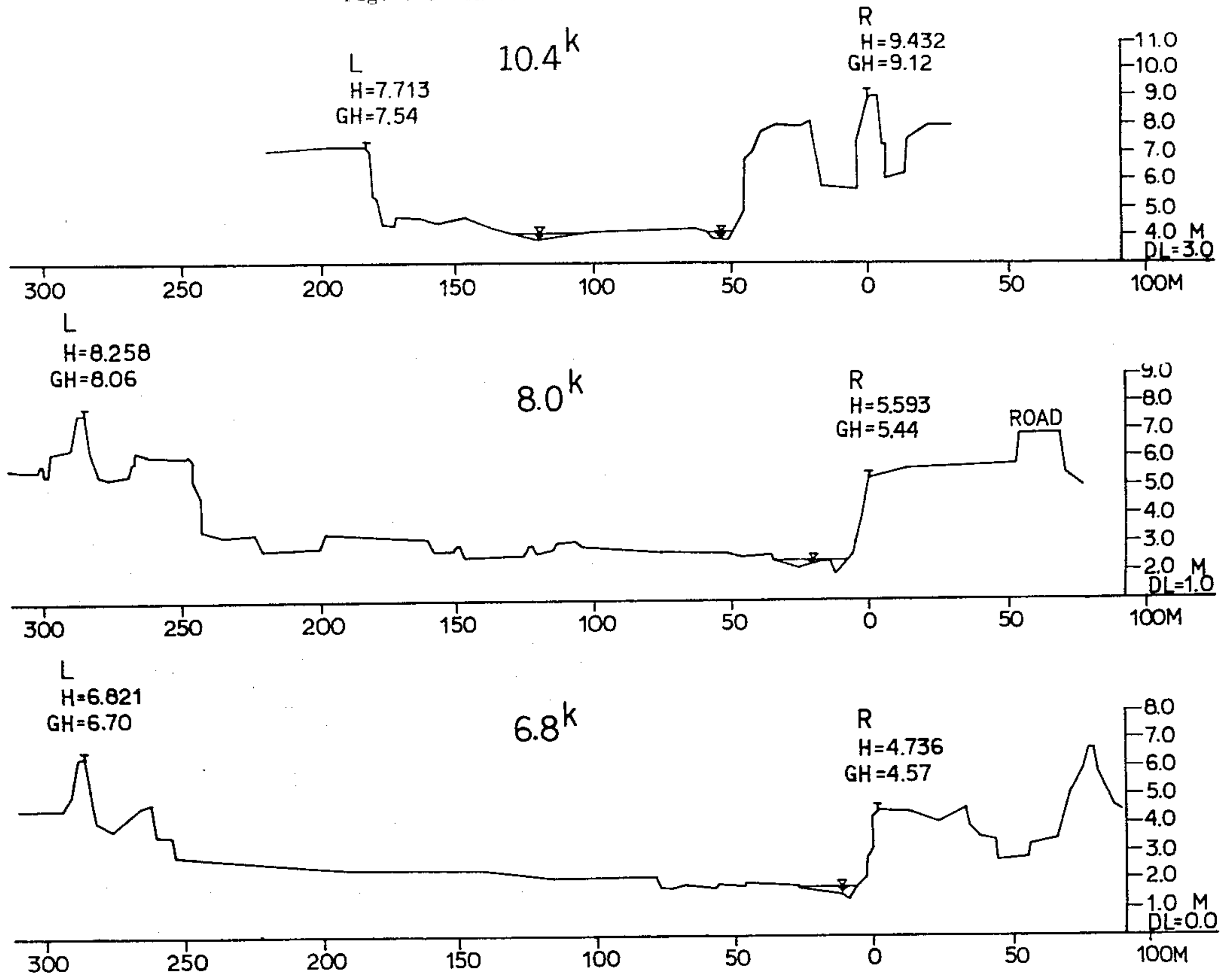


Fig. 1-7 NATURAL BUFFER

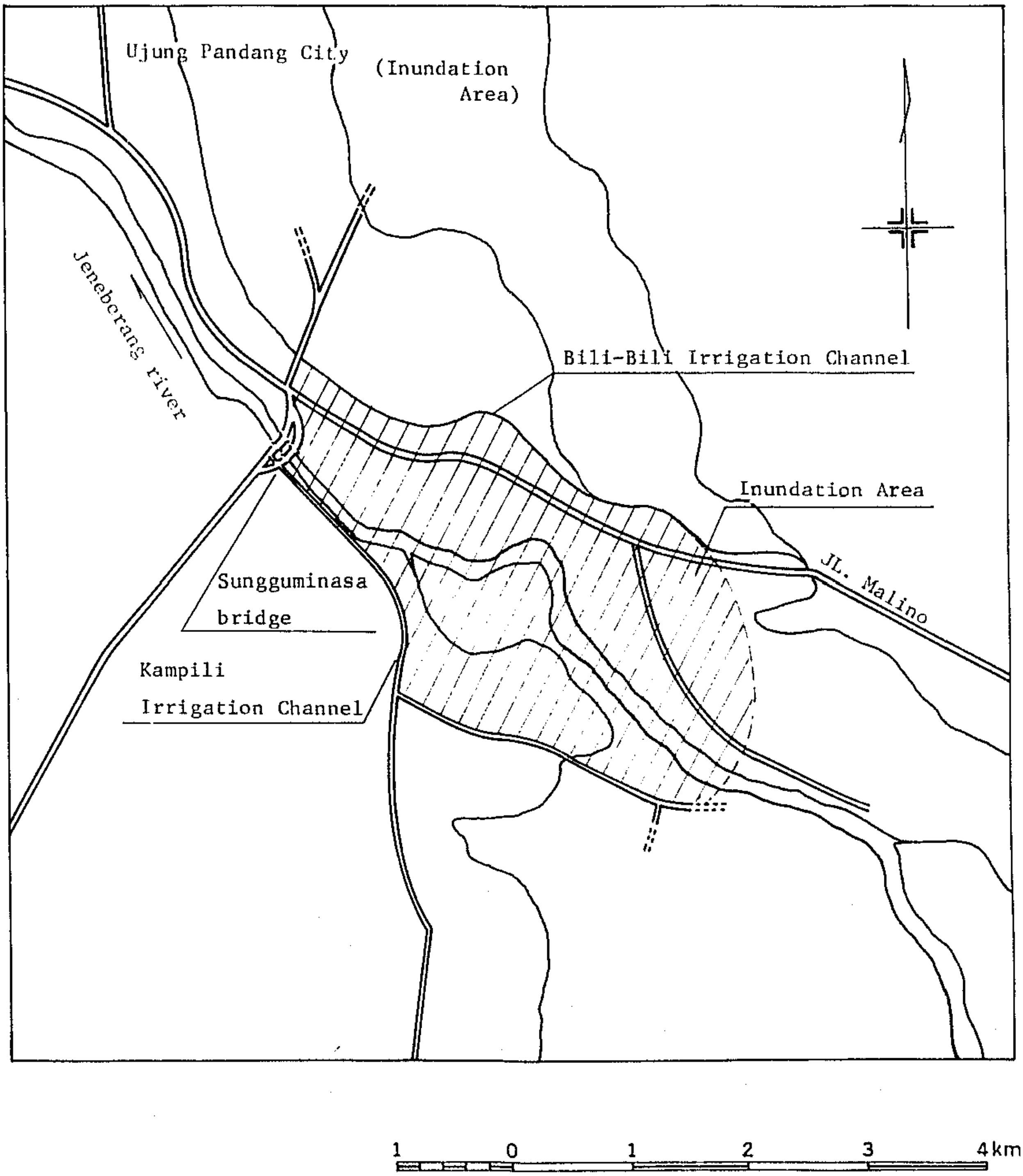
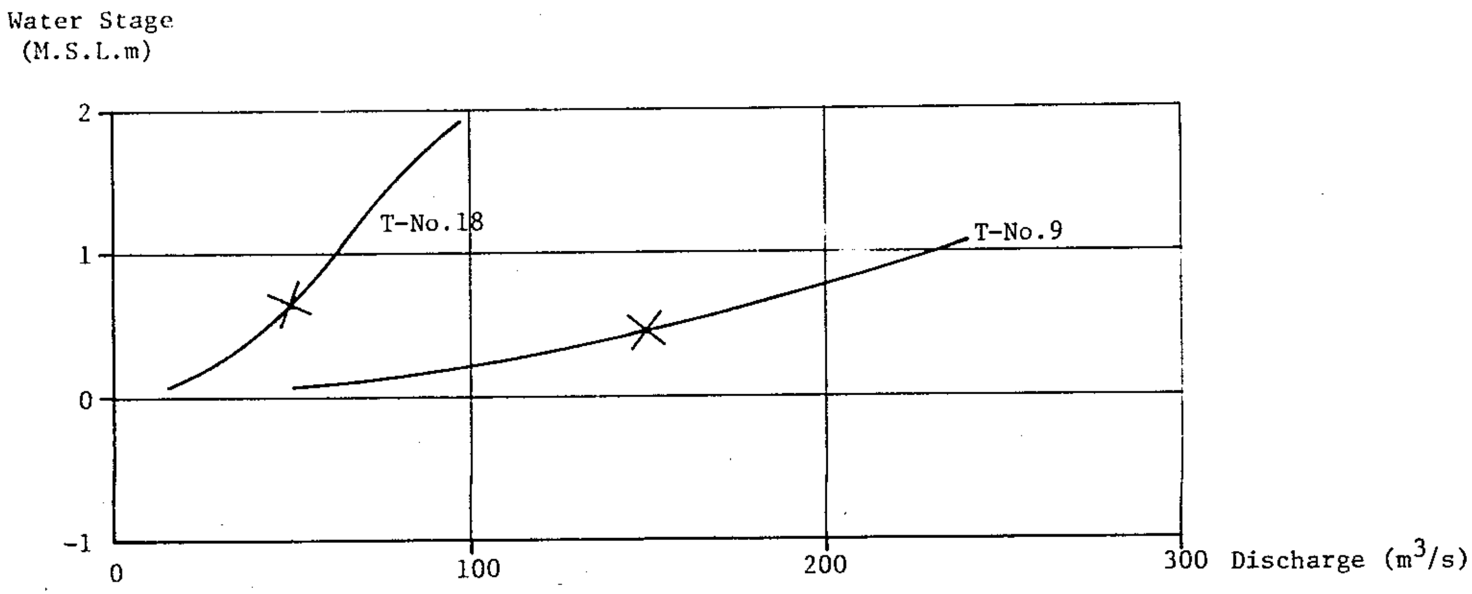


Fig. 1-8 RATING CURVE OF TALLO RIVER

(Section No.9 and No.18)



Note: X Overflow level

Fig.1-9 CROSS-SECTION OF TALLO RIVER

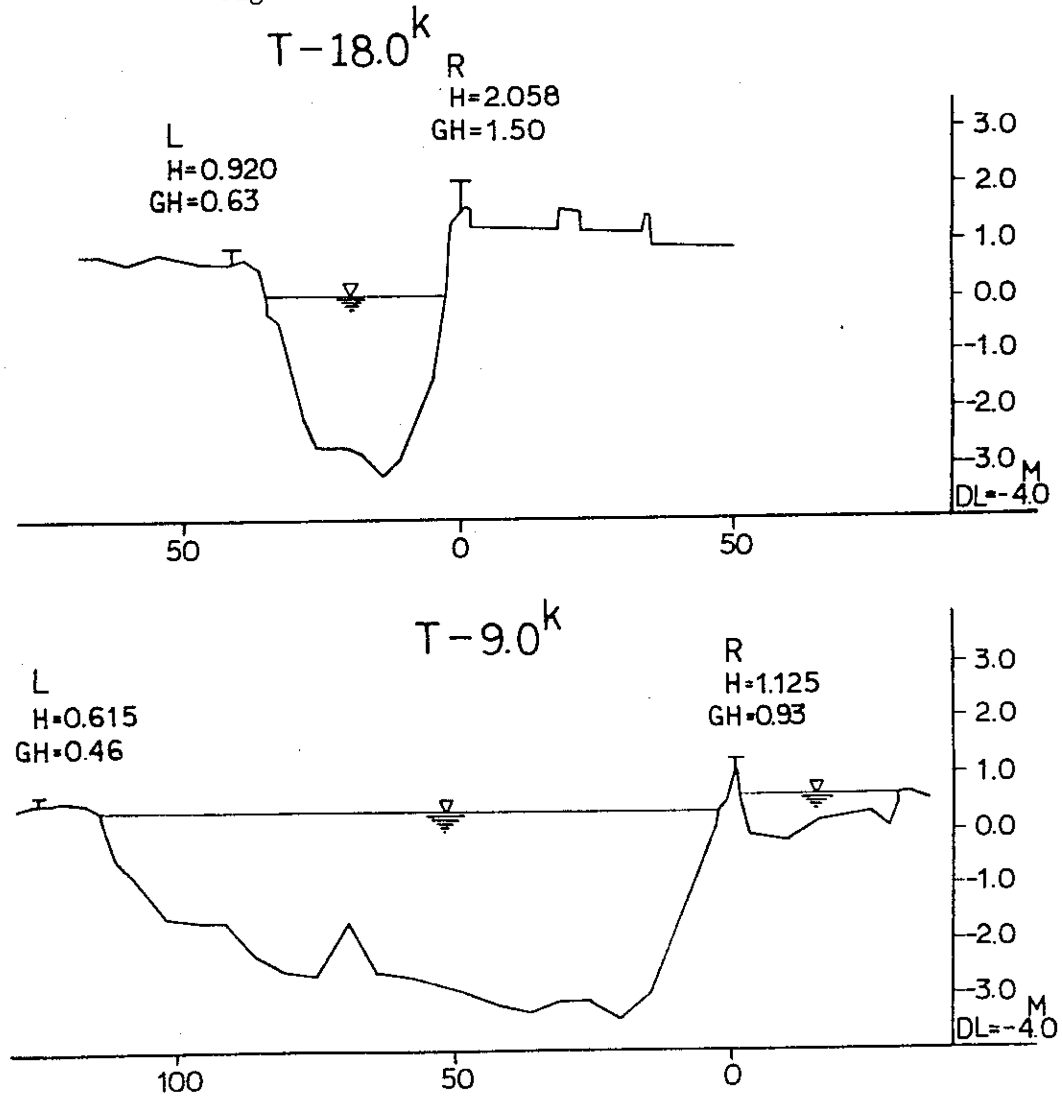
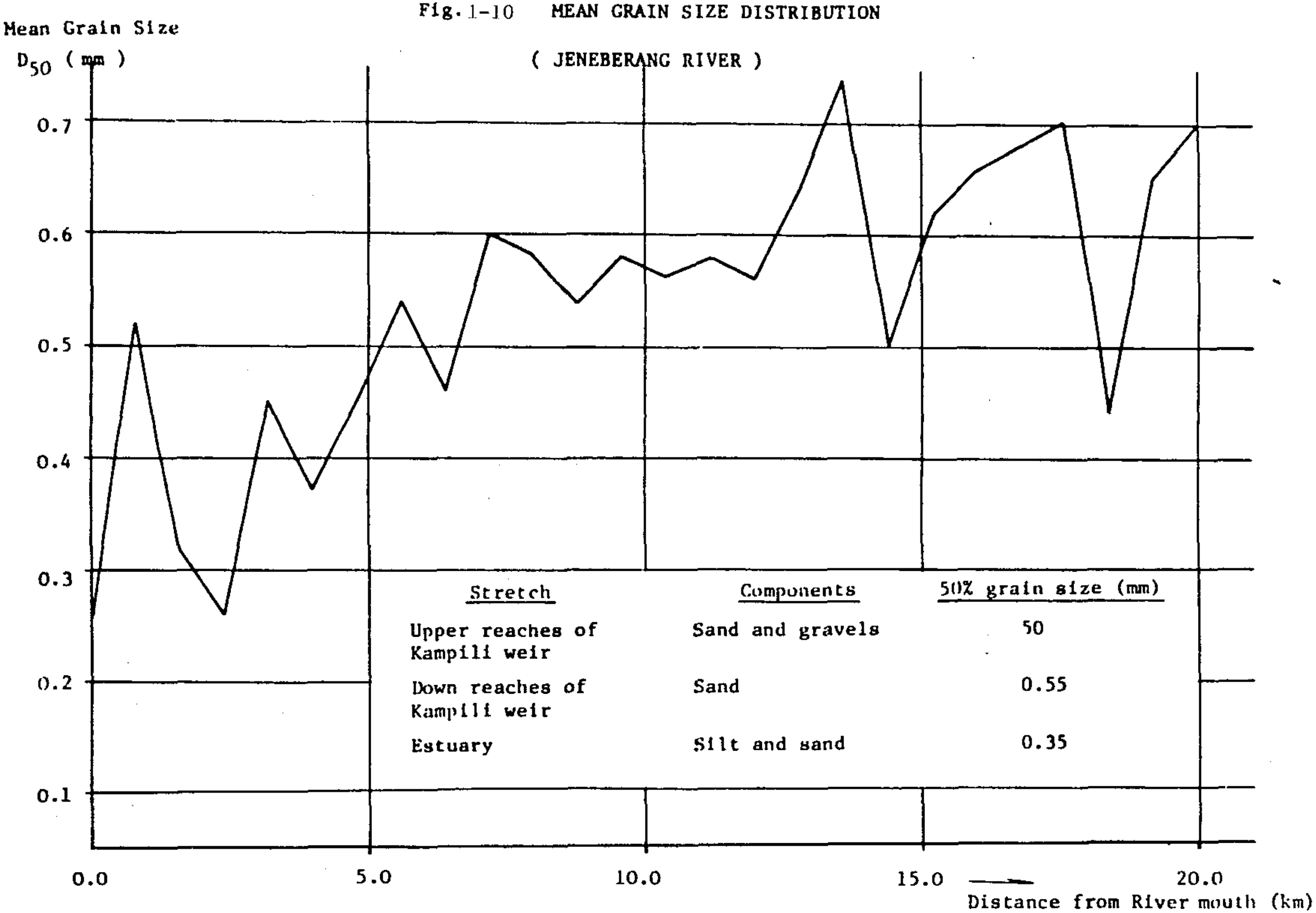


Fig. 1-10 MEAN GRAIN SIZE DISTRIBUTION

(JENEBERANG RIVER)



Stretch	Components	50% grain size (mm)
Upper reaches of Kampili weir	Sand and gravels	50
Down reaches of Kampili weir	Sand	0.55
Estuary	Silt and sand	0.35

Fig. 1-11(1)   GRAIN-SIZE ACCUMULATION CURVE ALONG JENEBERANG RIVER (0.0k)

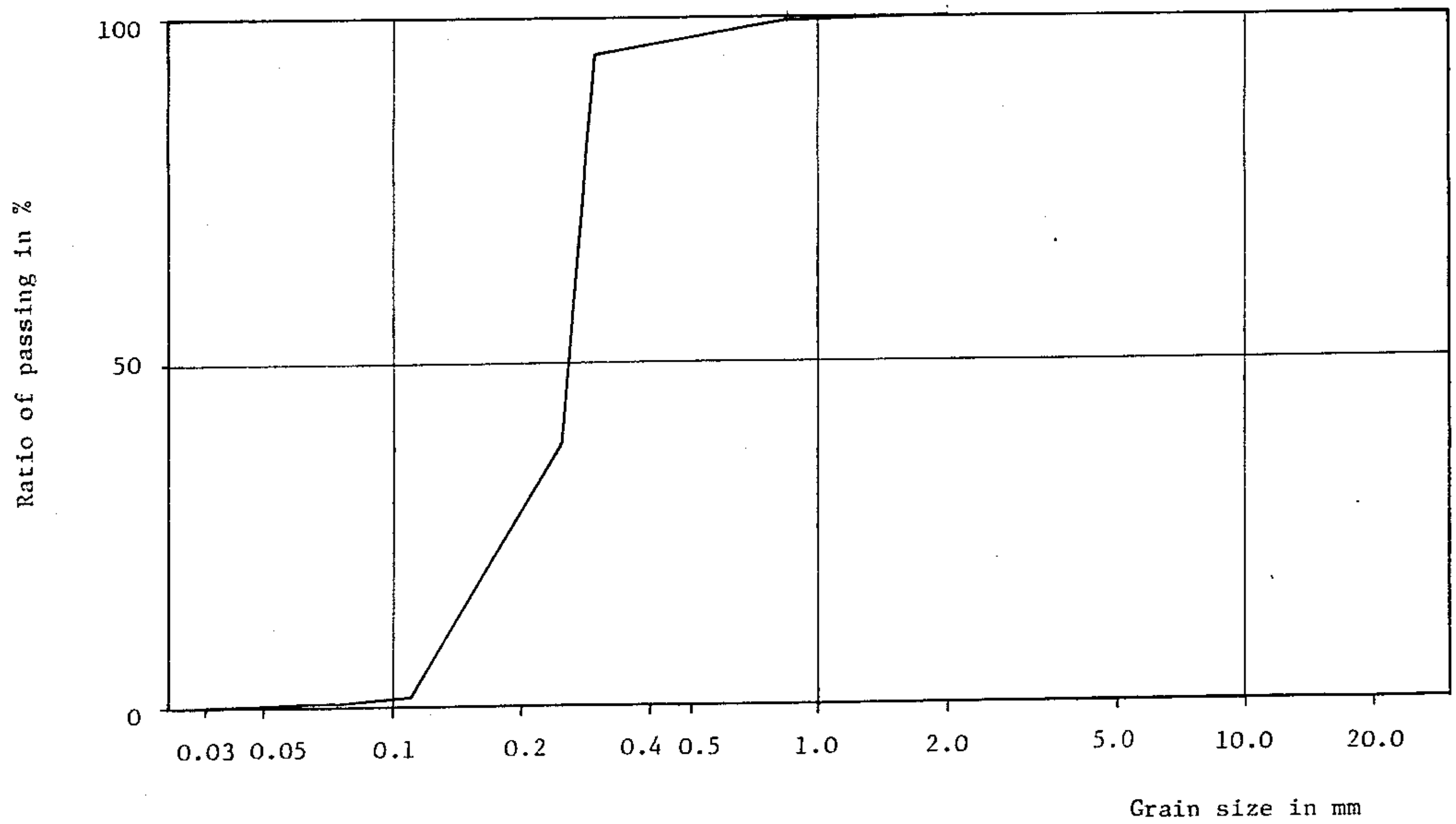


Fig.1-11(2) GRAIN-SIZE ACCUMULATION CURVE ALONG JENEBERANG RIVER (8.3^k)

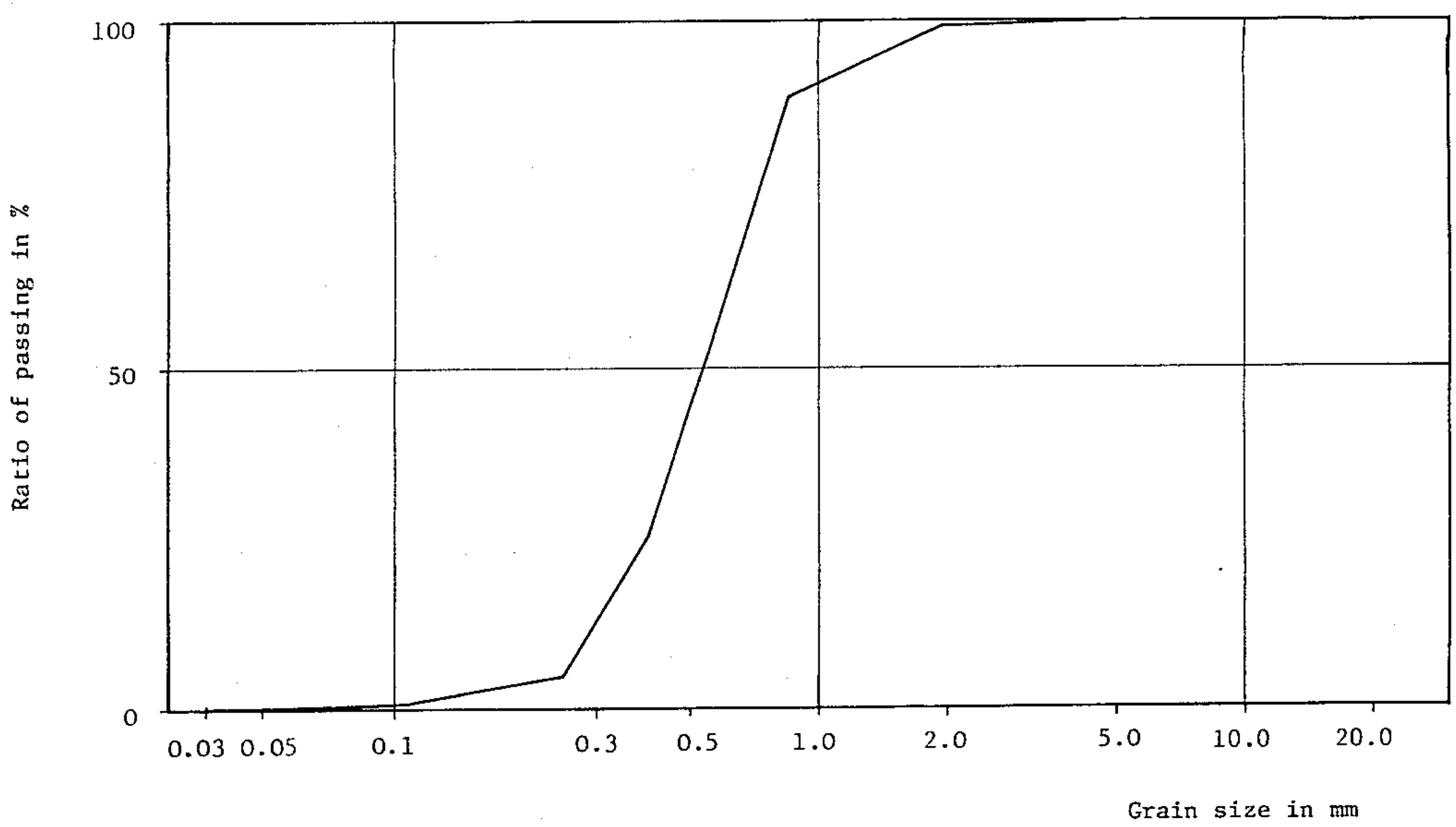


Fig. I-11(3) GRAIN-SIZE ACCUMULATION CURVE ALONG JENEBERANG RIVER (20.0^k)

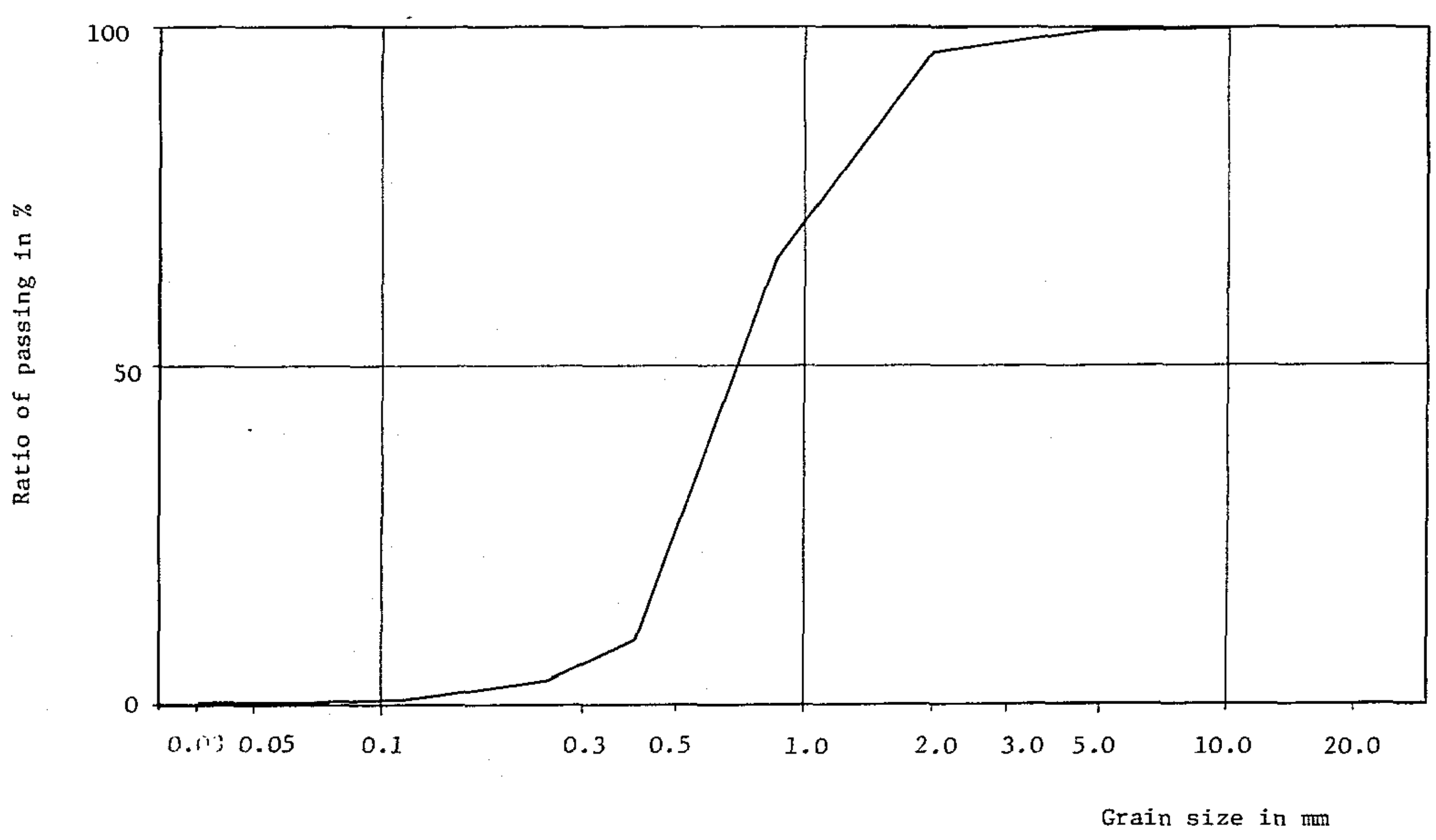


Fig. 1-12 TRANSITION OF ESTUARY SAND BAR

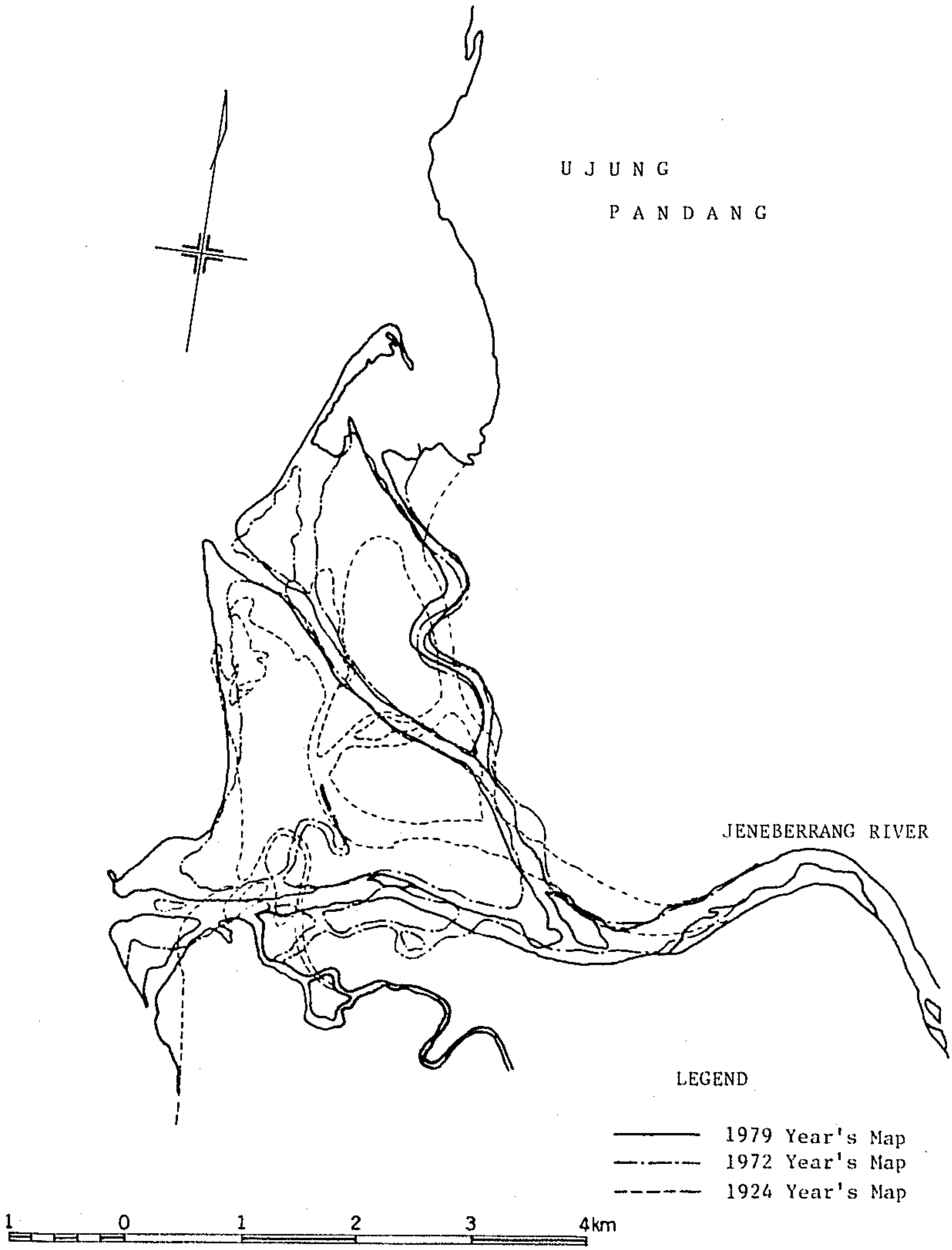


Fig. 1-13 TRANSITION OF ESTUARY SAND BAR

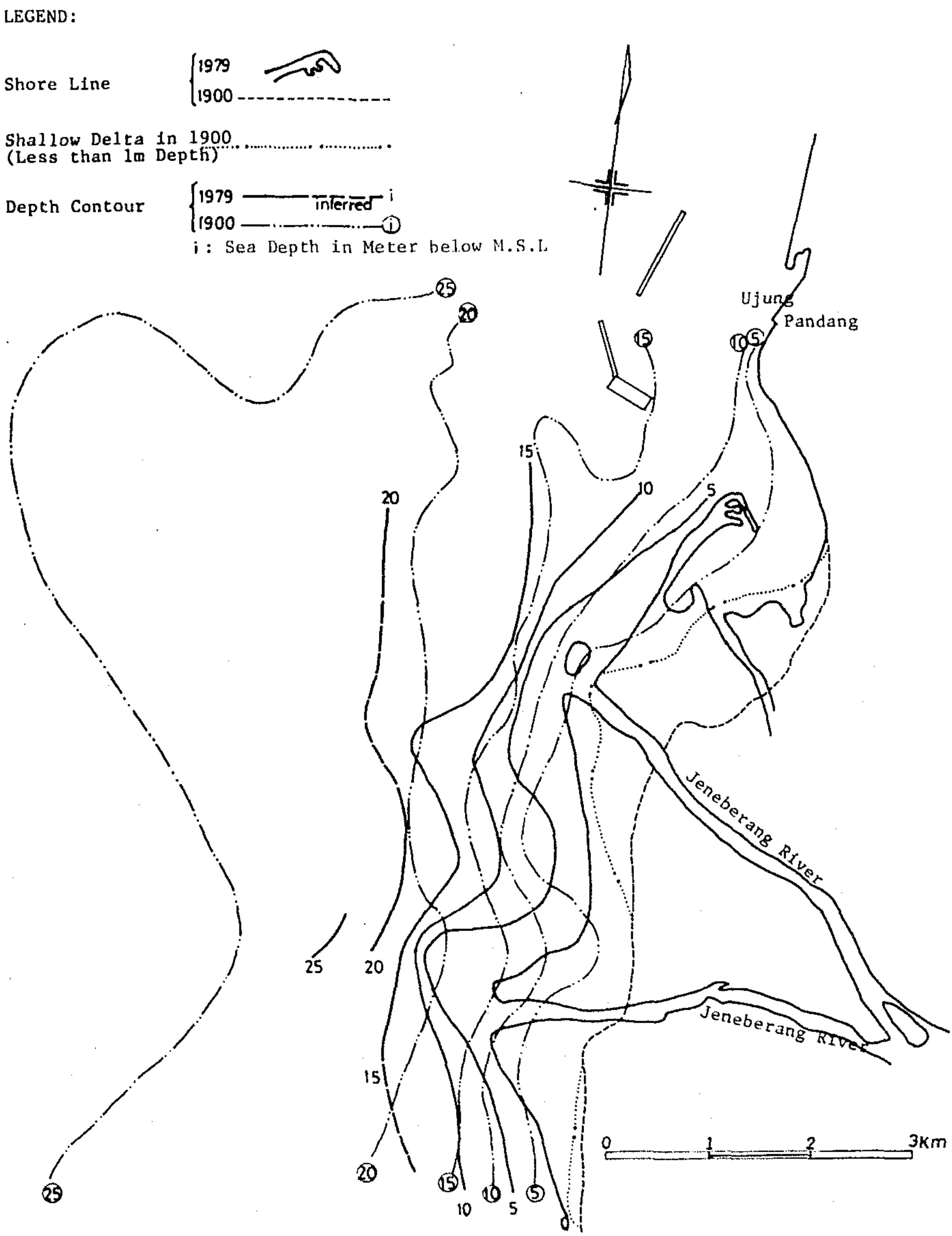
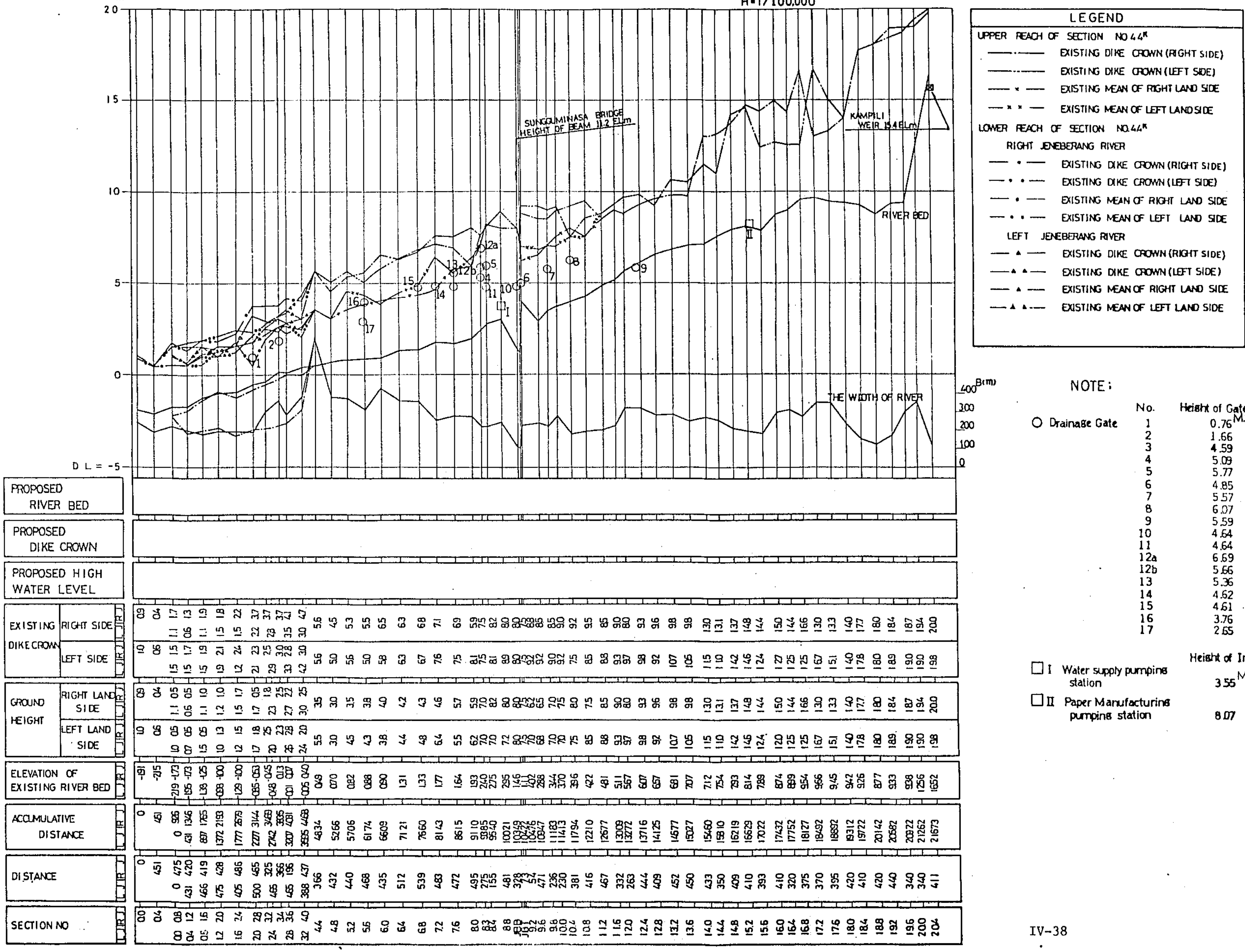


Fig. I-14(1) EXISTING RIPARIAN FACILITIES OF JENEBERANG RIVER



NOTE:

○ Drainage Gate

No.	Height of Gate M.S.L.m
1	0.76
2	1.66
3	4.59
4	5.09
5	5.77
6	4.85
7	5.57
8	6.07
9	5.59
10	4.64
11	4.64
12a	6.69
12b	5.66
13	5.36
14	4.62
15	4.61
16	3.76
17	2.65

	Height of Intake M.S.L.m
□ I Water supply pumping station	3.56
□ II Paper Manufacturing pumping station	8.07

Fig. I-14(2) EXISTING RIPARIAN FACILITIES OF JENEBERANG RIVER

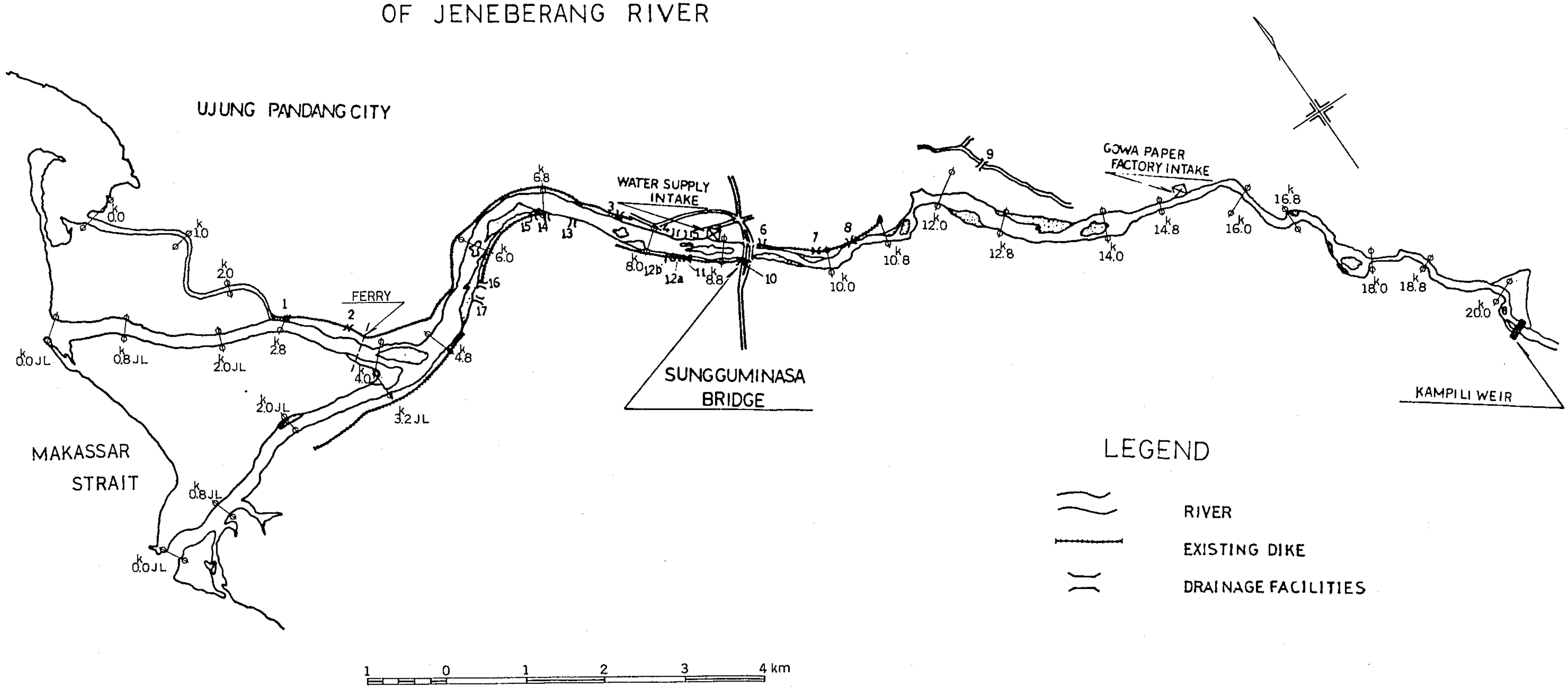


Fig. 2-1 RELATION BETWEEN COST AND DISCHARGE
(FLOOD CONTROL DAM)

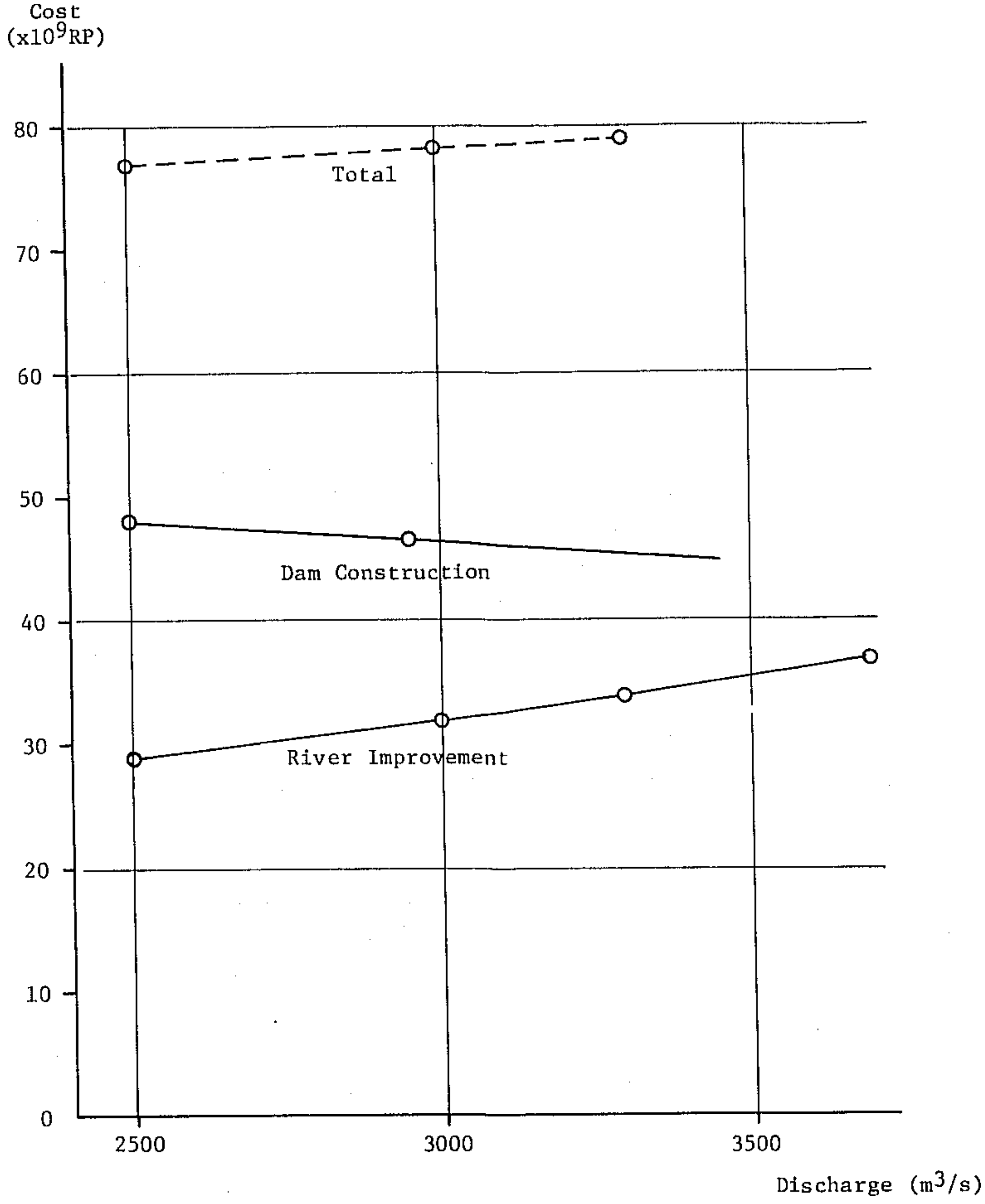
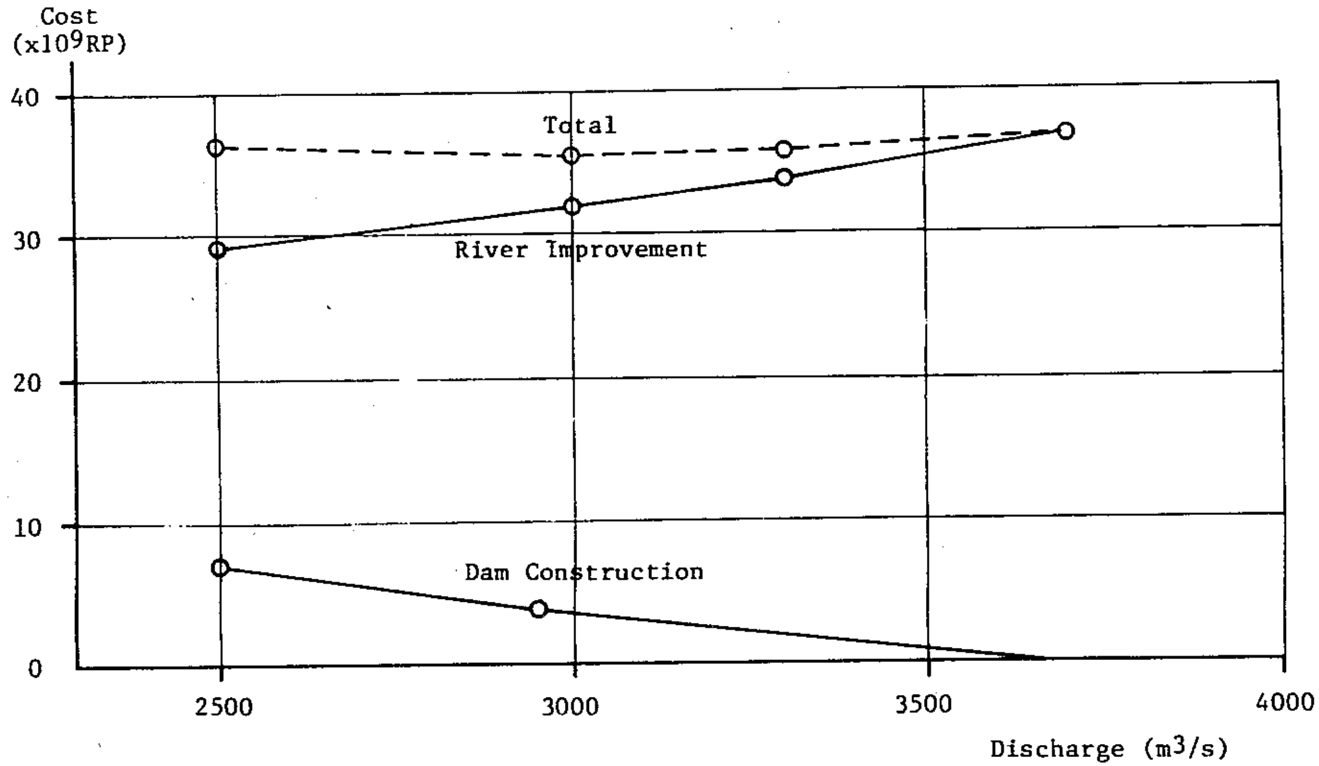


Fig. 2-2 RELATION BETWEEN COST AND DISCHARGE
(MULTI-PURPOSE DAM)



Note: The cost allocated to flood control capacity of a multipurpose dam is counted for the dam construction cost.

Fig.2-3 DISCHARGE-LAND ACQUISITION AND DISCHARGE-HOUSE EVACUATION CURVES

(River Improvement and Dam Construction)

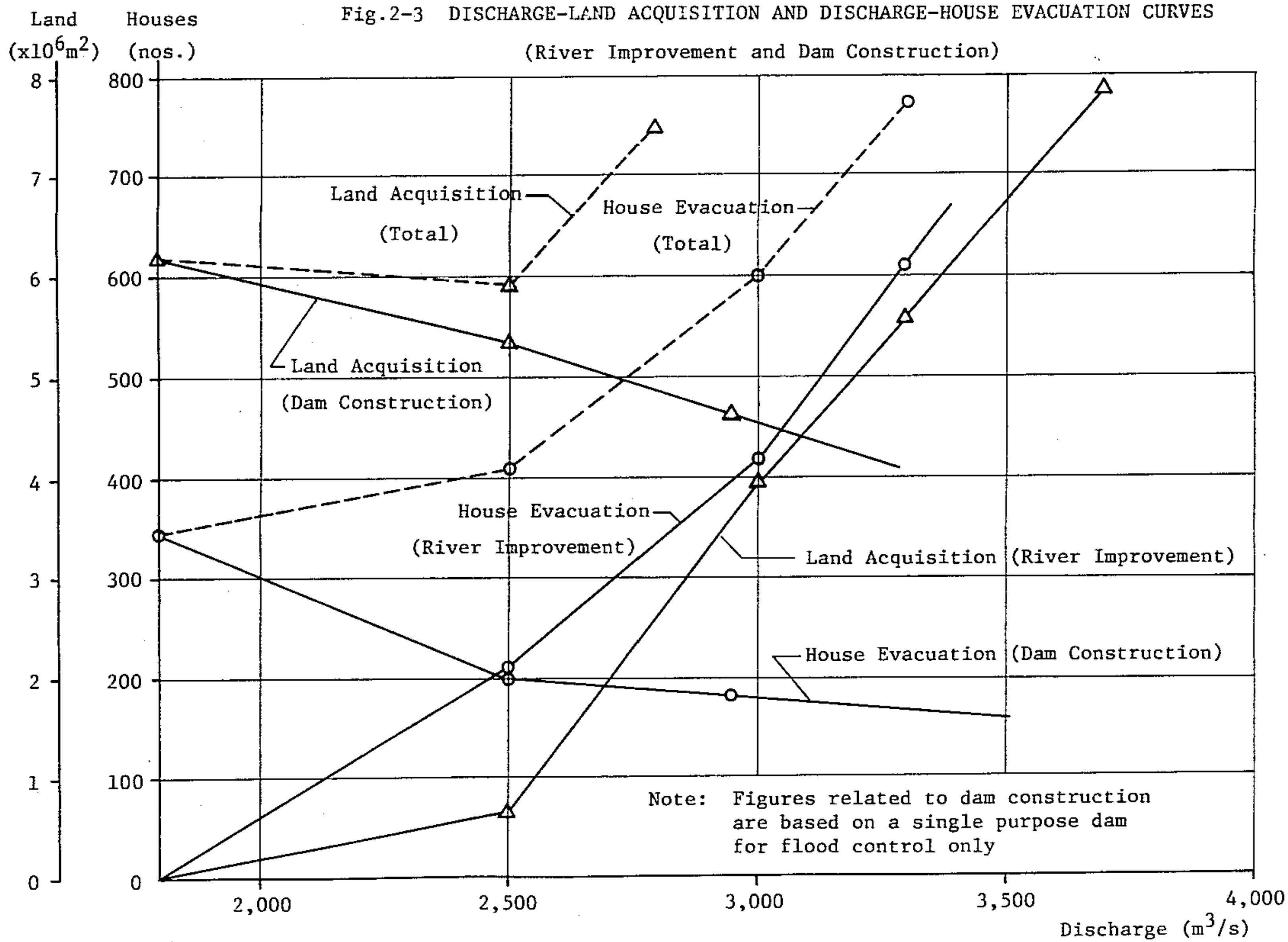
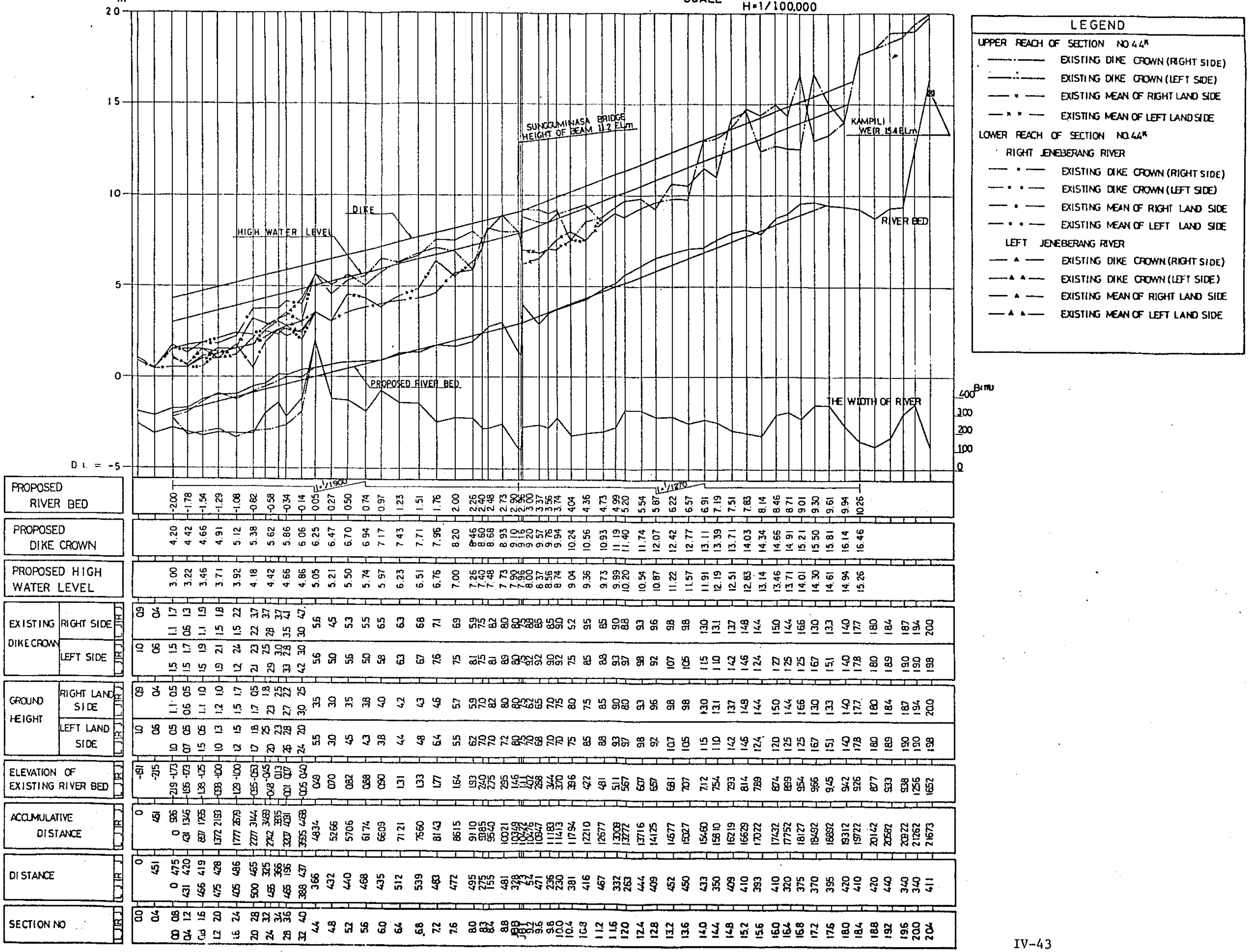


Fig. 3-1 PROPOSED LONGITUDINAL PROFILE OF JENEBERANG RIVER

(OVERALL PLAN)

SCALE V= 1/200 H=1/100,000



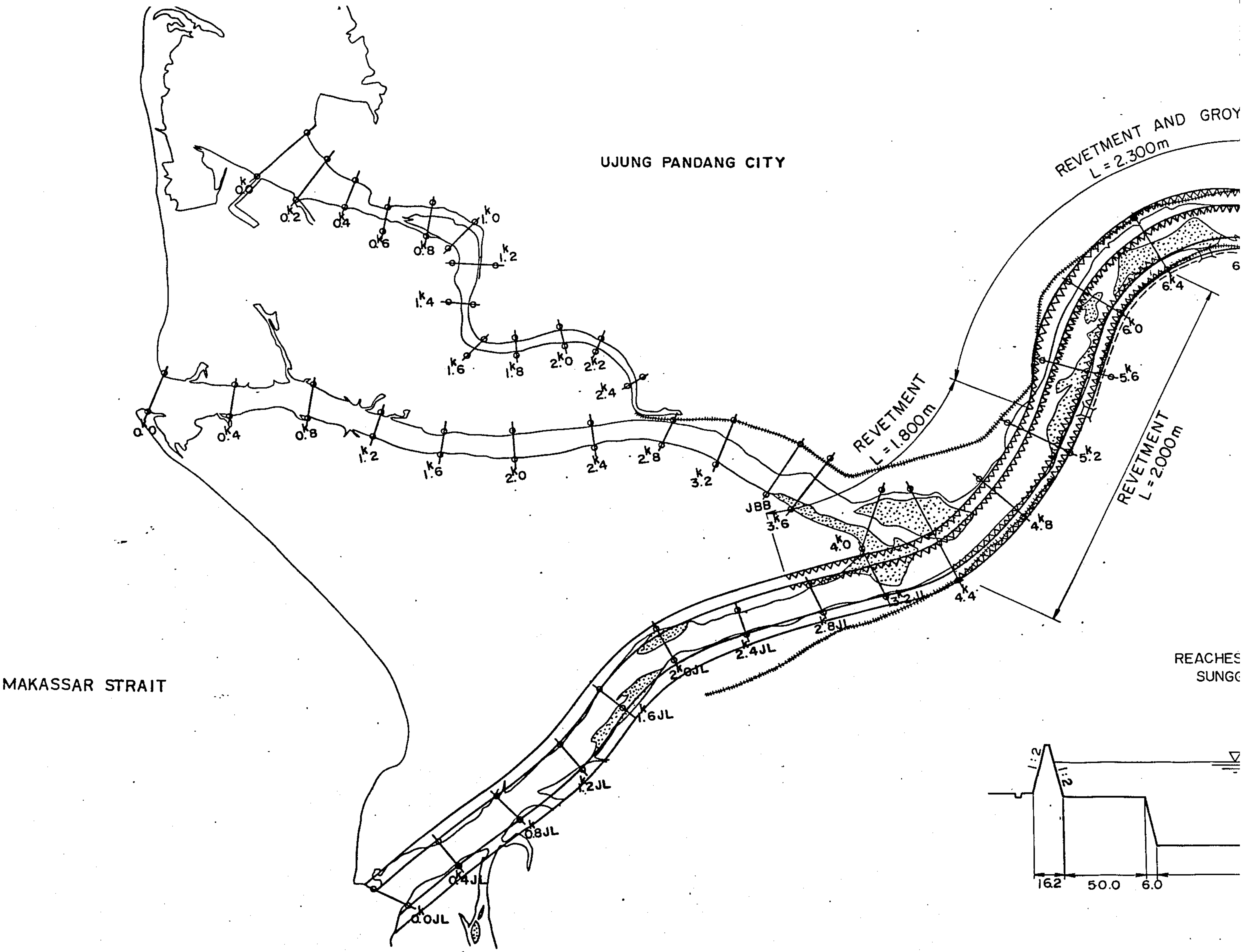
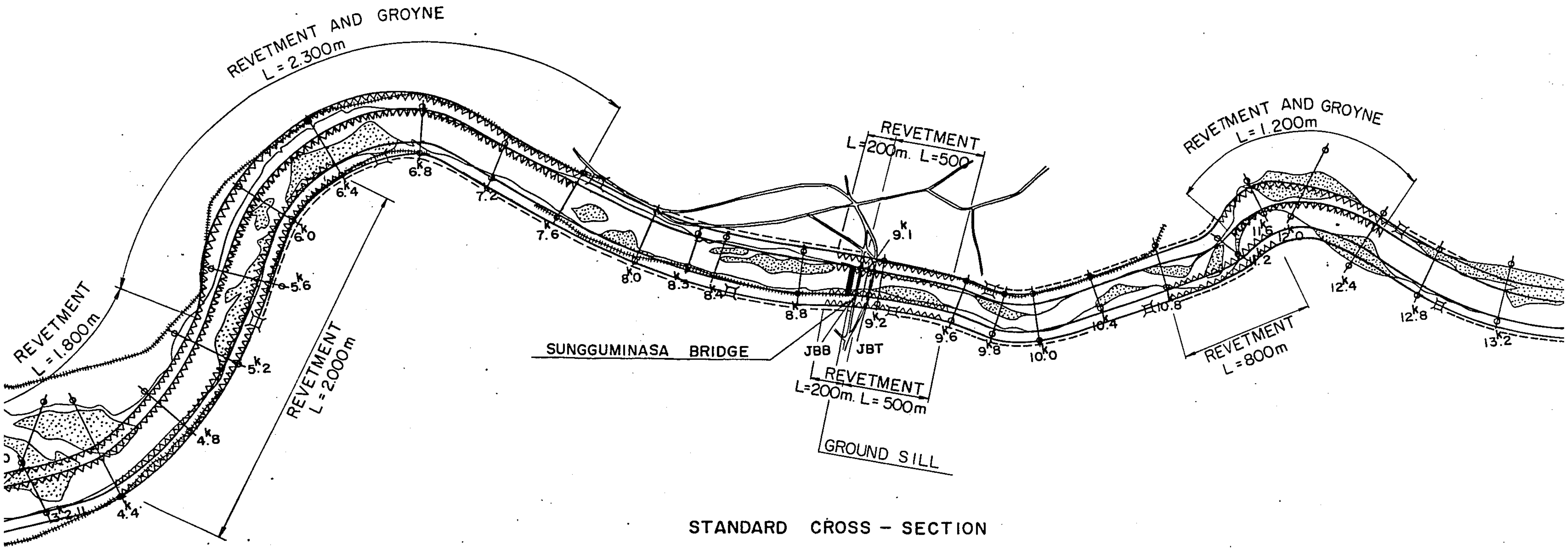
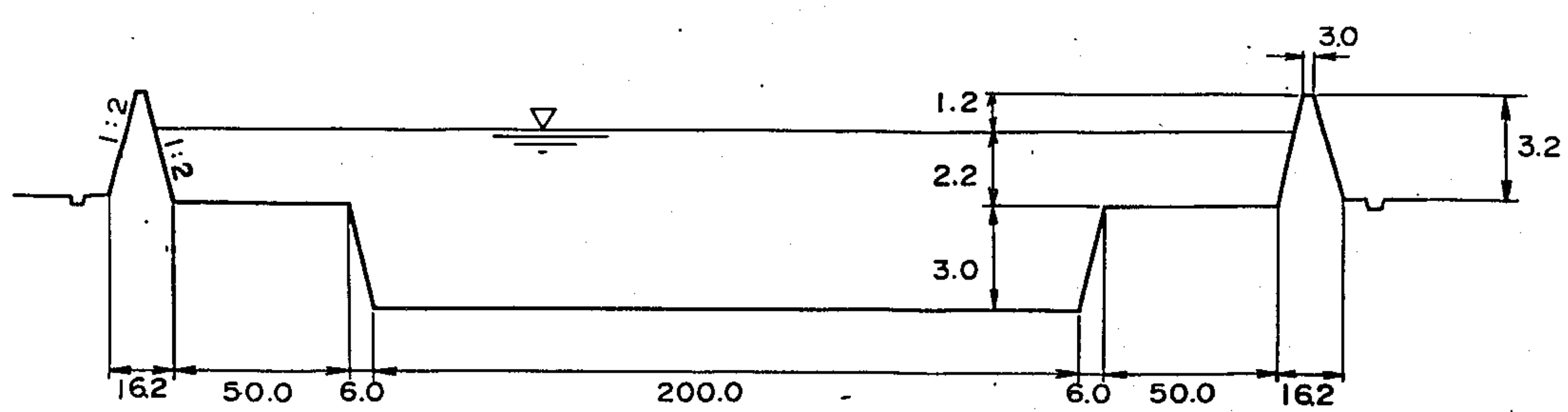

UJUNG PANDANG CITY
MAKASSAR STRAIT
REVETMENT AND GROY
L = 2.300m
REVETMENT
L = 1.800m
REVETMENT
L = 2.000m
REACHES
SUNGG
JBB
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
2.8
3.2
3.6
4.0
4.4
4.8
5.2
5.6
6.0
6.4
0.0JL
0.4JL
0.8JL
1.2JL
1.6JL
2.0JL
2.4JL
2.8JL
3.2JL
1:2
2:1
16.2
50.0
6.0

Fig. 3-2 PROPOSED ALIGNMENT OF OVERALL FLOOD CONTROL

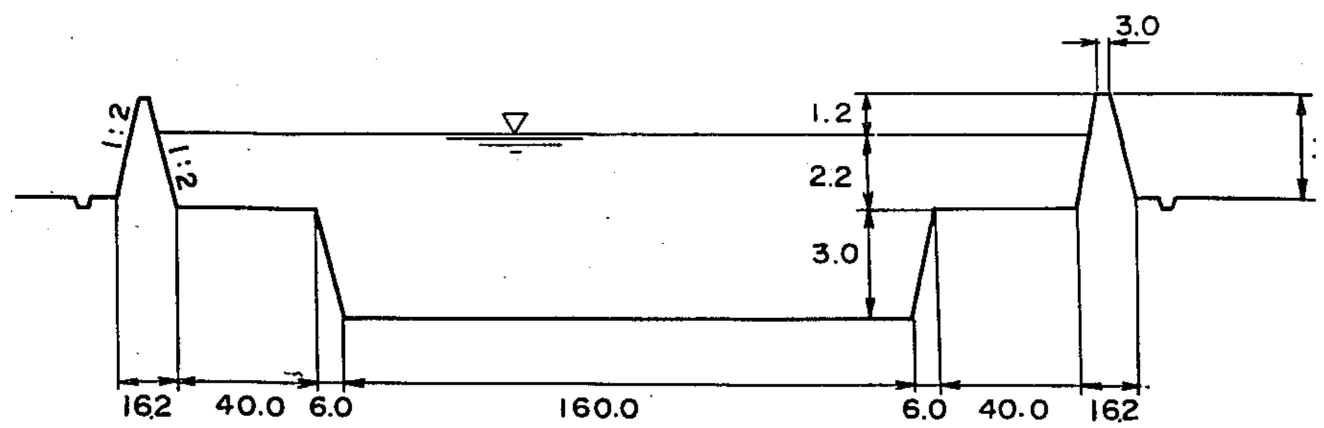


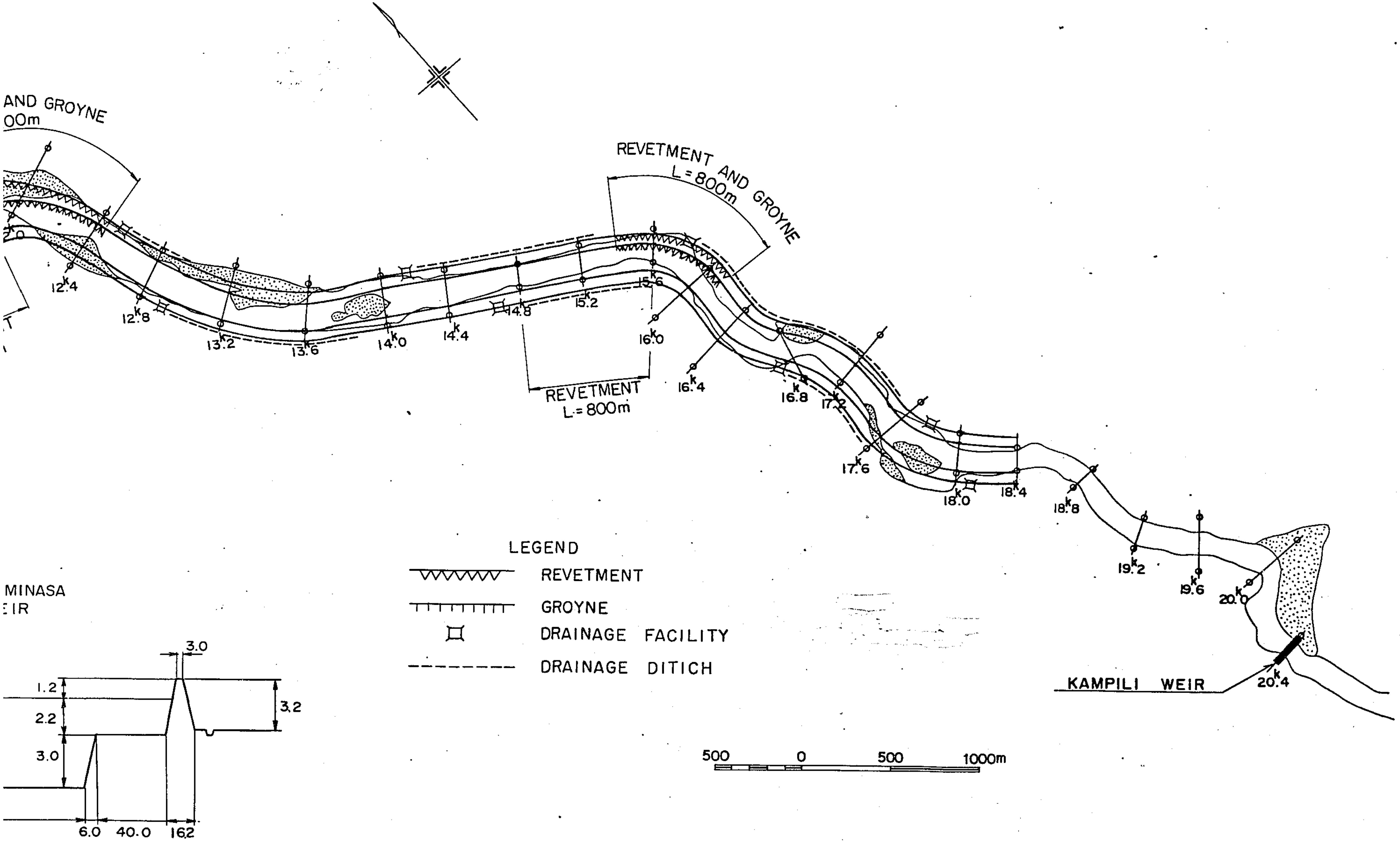
STANDARD CROSS - SECTION

REACHES BETWEEN ESTUARY AND SUNGGUMINASA BRIDGE



REACHES BETWEEN SUNGGUMINASA BRIDGE AND KAMPILI WEIR





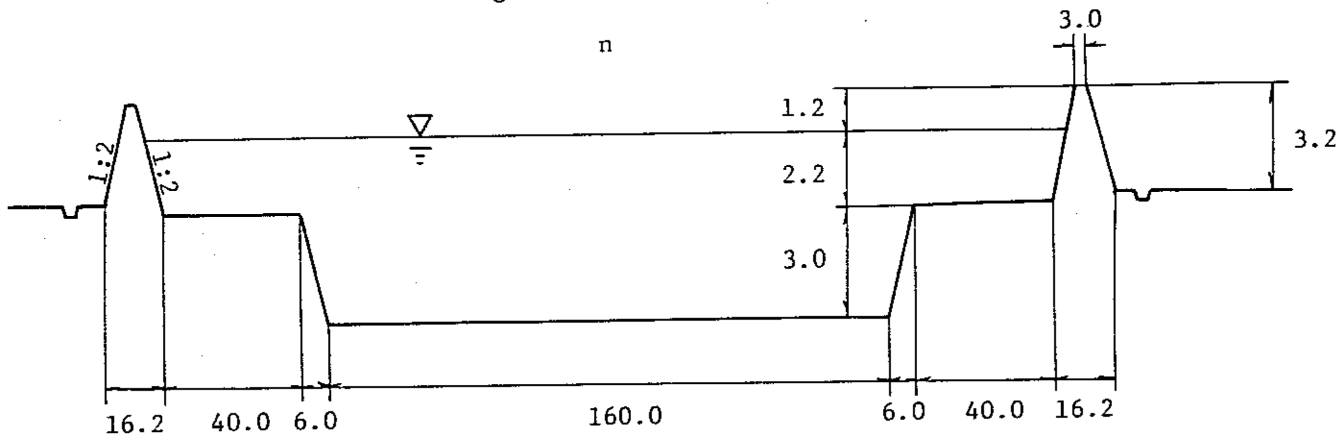
AND GROYNE
00m
REVETMENT AND GROYNE
L=800m
REVETMENT
L=800m
12.4
12.8
13.2
13.6
14.0
14.4
14.8
15.2
15.6
16.0
16.4
16.8
17.2
17.6
18.0
18.4
18.8
19.2
19.6
20.0
20.4
KAMPILI WEIR
MINASA
EIR
LEGEND
REVETMENT
GROYNE
DRAINAGE FACILITY
DRAINAGE DITICH
500
0
500
1000m
3.0
1.2
2.2
3.0
3.2
6.0
40.0
16.2

Fig. 3-3 STANDARD CROSS-SECTION (OVERALL PLAN)

Unit : m

Reaches between Sungguminasa Bridge and Kampili Weir

n



Reaches between estuary and Sungguminasa Bridge

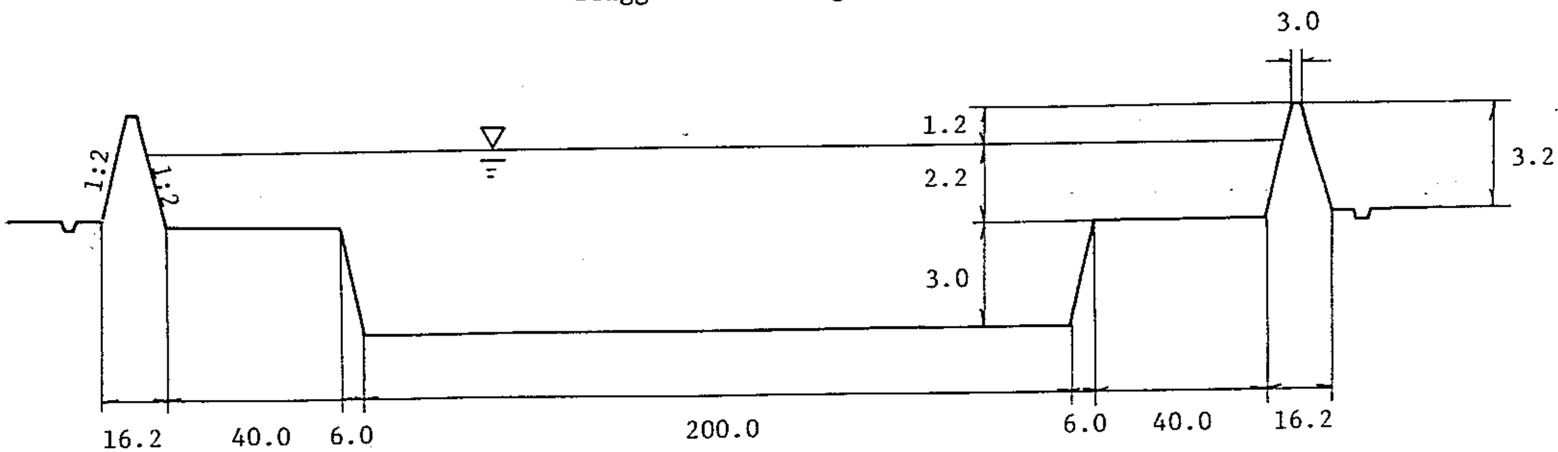


Fig.3-4 (1) PROPOSED CROSS-SECTIONS FOR OVERALL PLAN

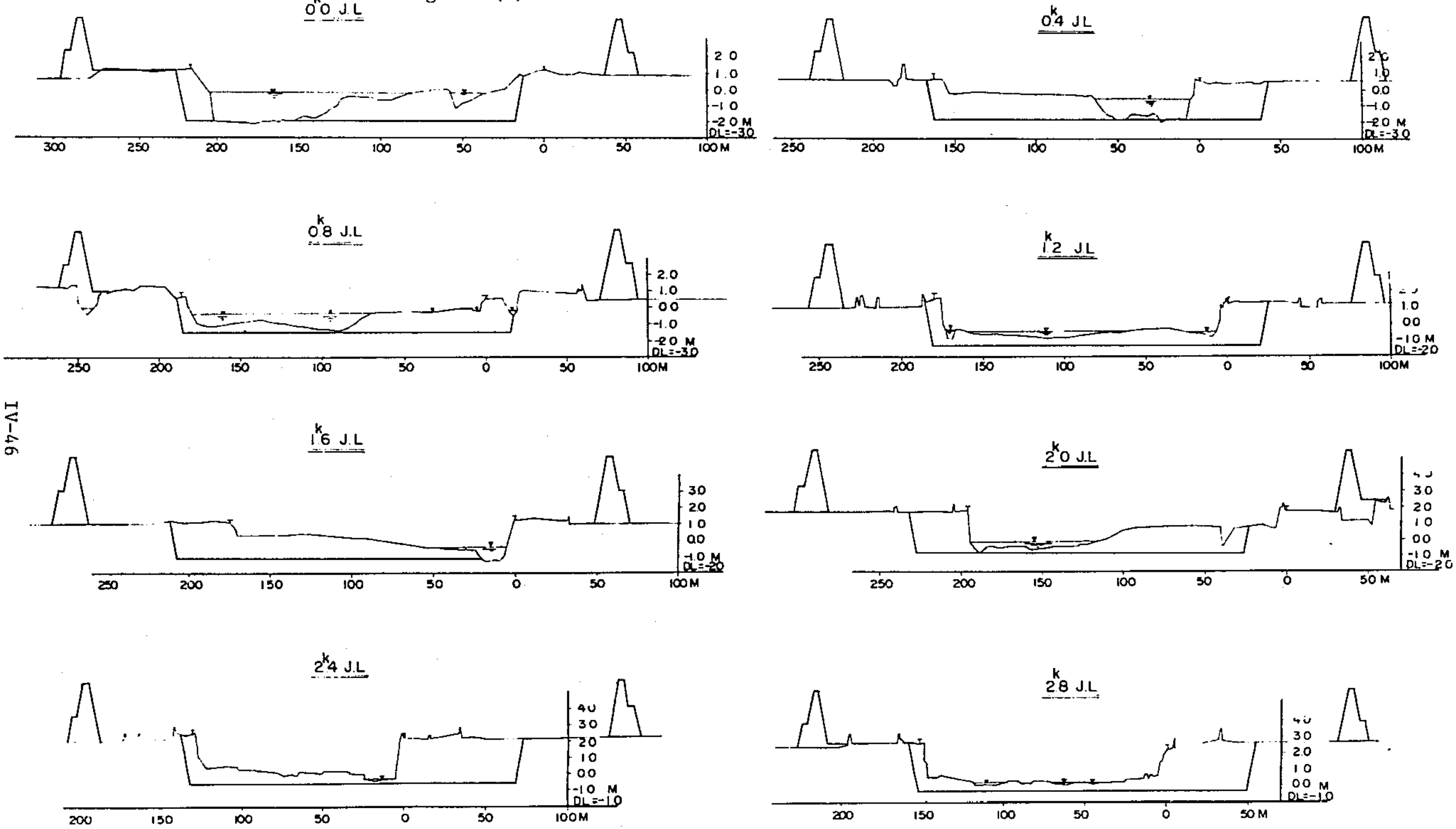


Fig.3-4 (2) PROPOSED CROSS-SECTIONS FOR OVERALL PLAN

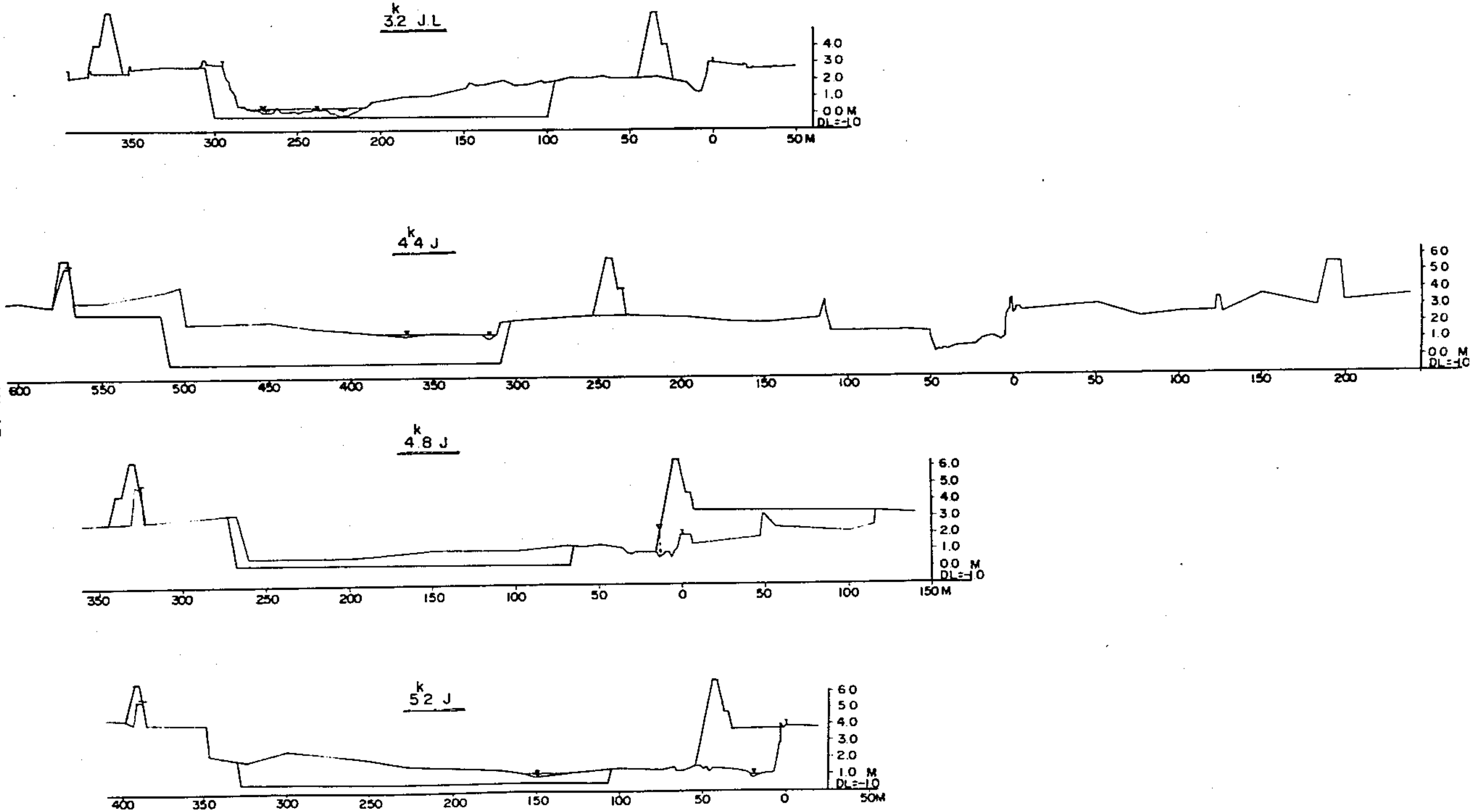


Fig.3-4 (3)   PROPOSED CROSS-SECTIONS FOR OVERALL PLAN

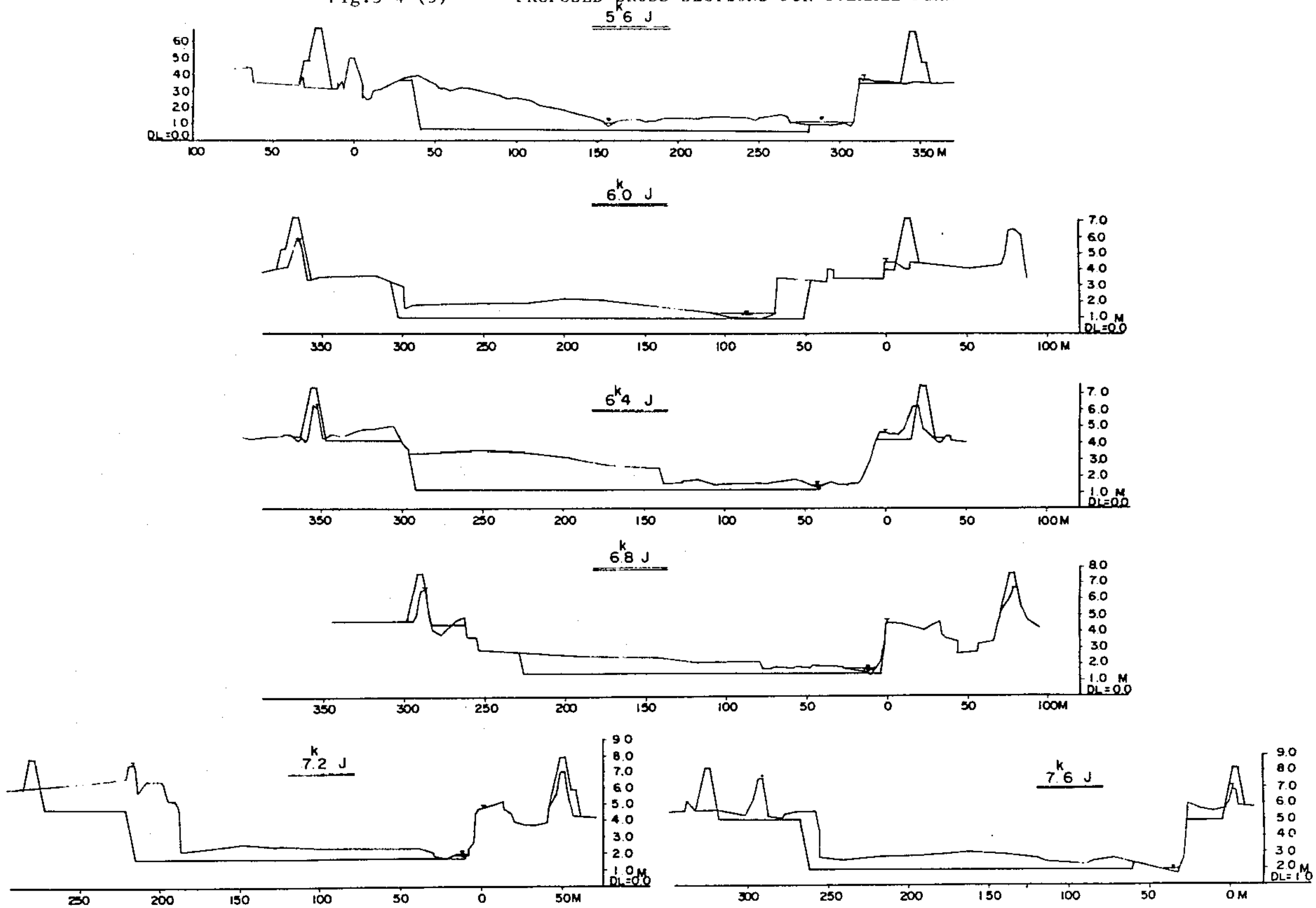


Fig.3-4 (4)   PROPOSED CROSS-SECTIONS FOR OVERALL PLAN

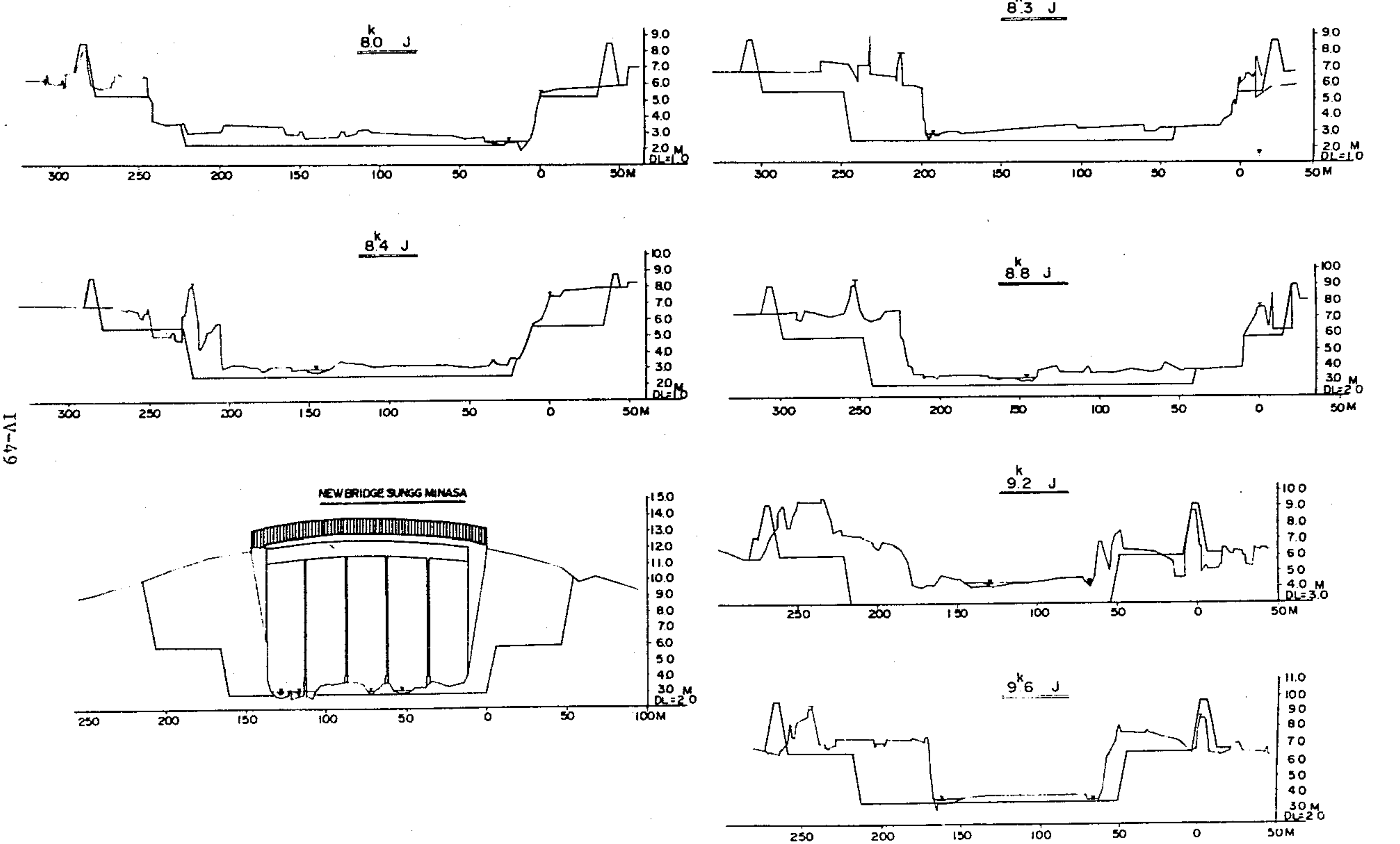


Fig.3-4 (5)   PROPOSED CROSS-SECTIONS FOR OVERALL PLAN

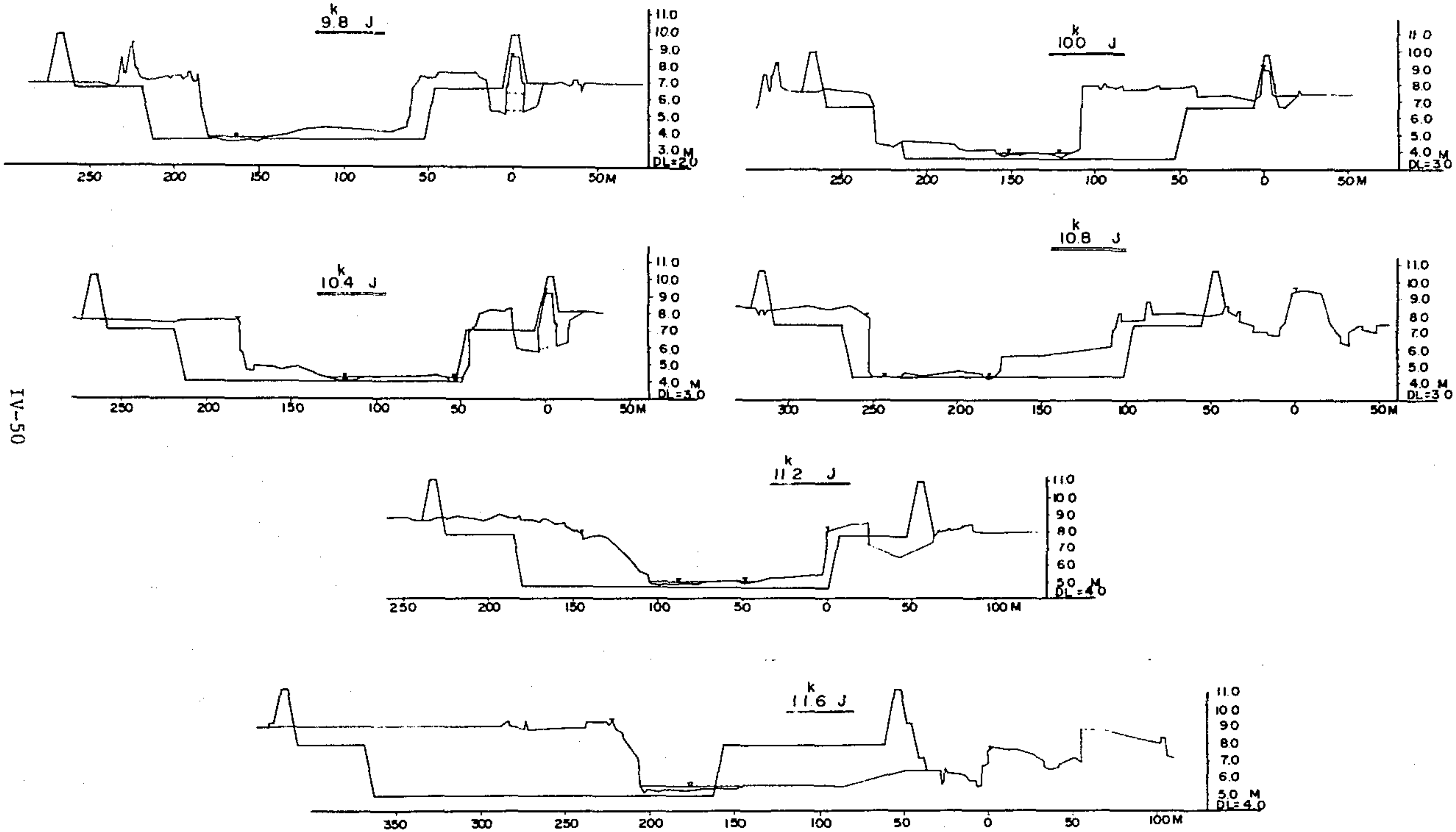


Fig.3-4 (6)　　PROPOSED CROSS-SECTIONS FOR OVERALL PLAN

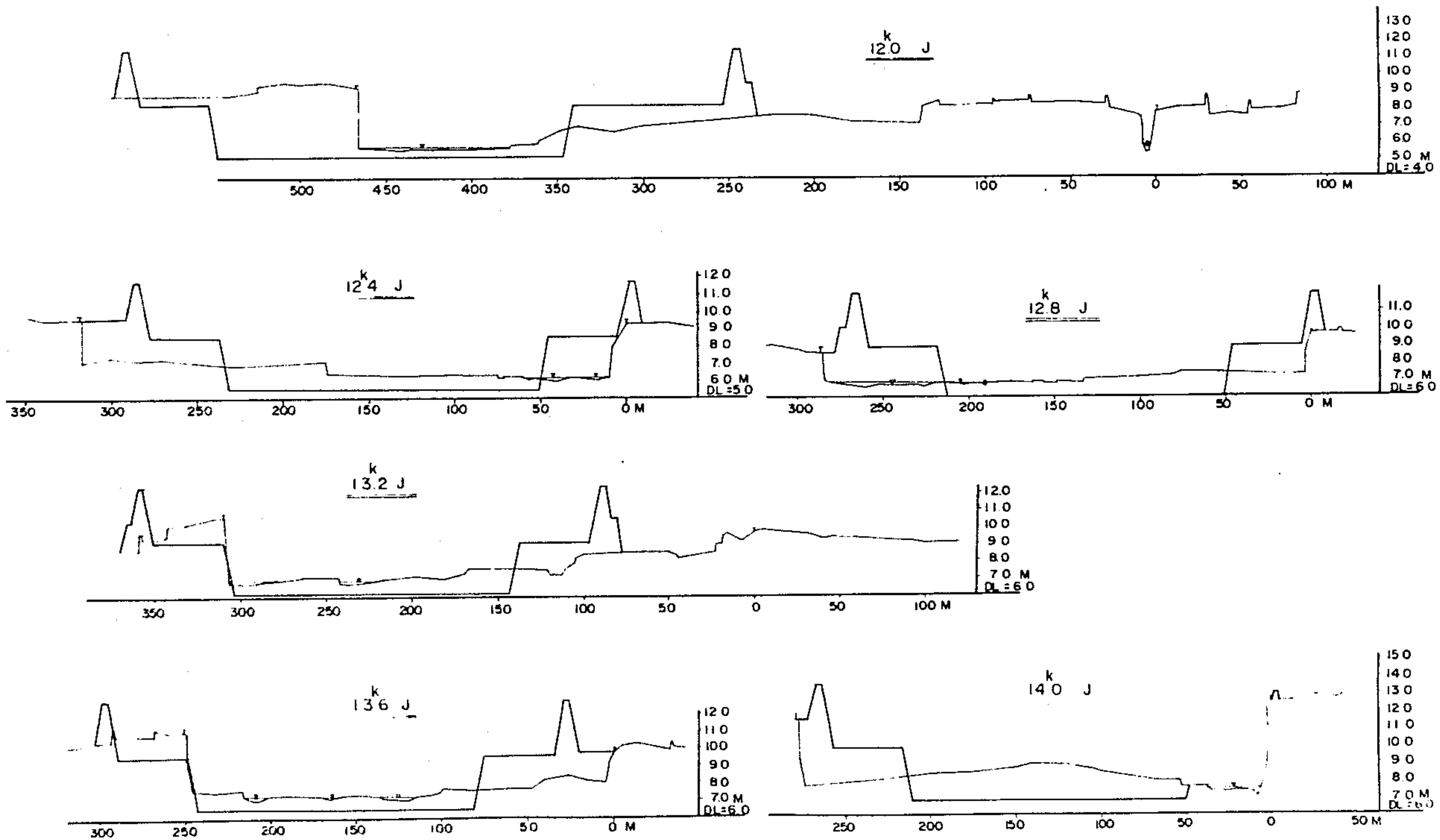


Fig.3-4 (7) PROPOSED CROSS-SECTIONS FOR OVERALL PLAN

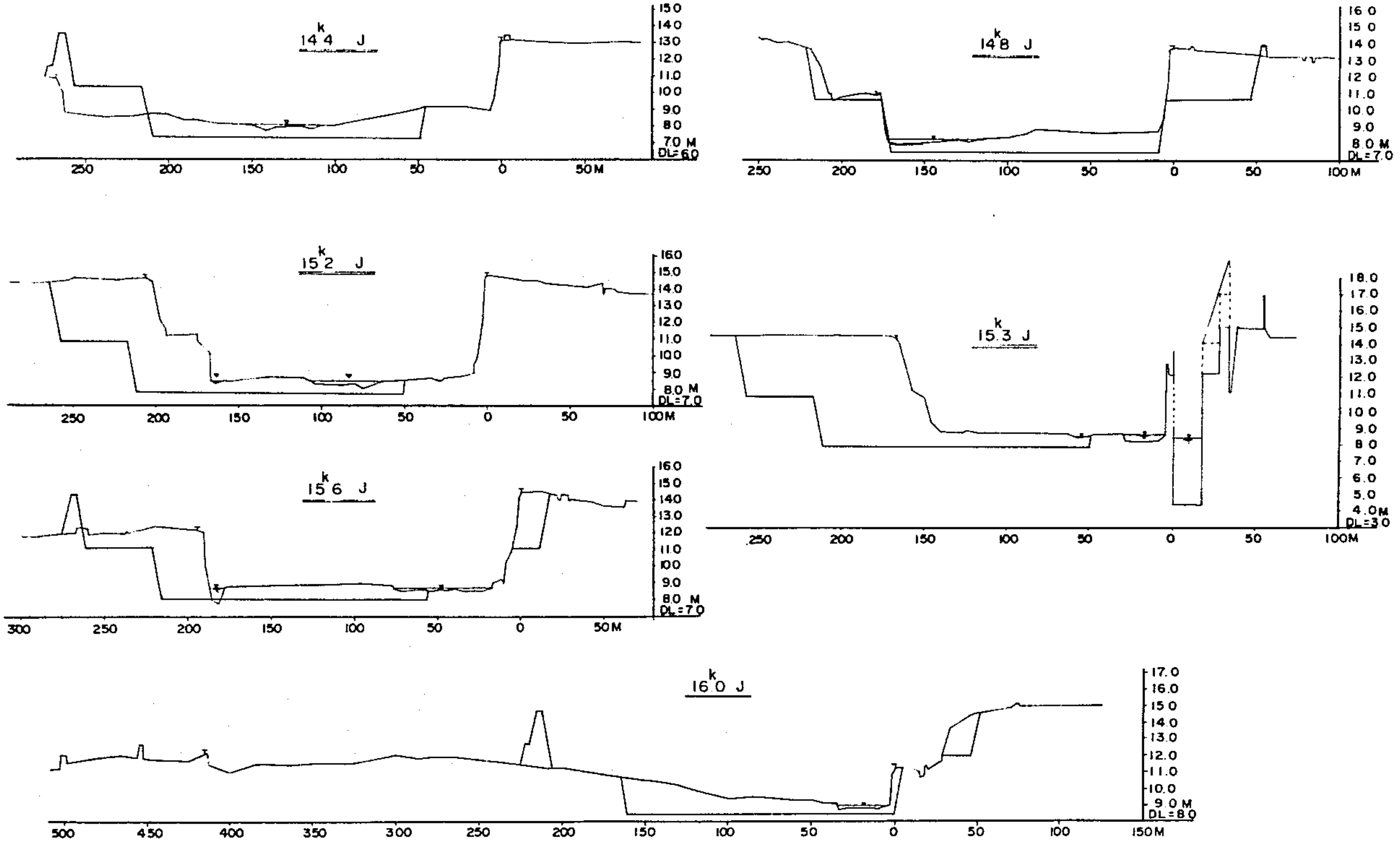


Fig.3-4 (8) PROPOSED CROSS-SECTIONS FOR OVERALL PLAN

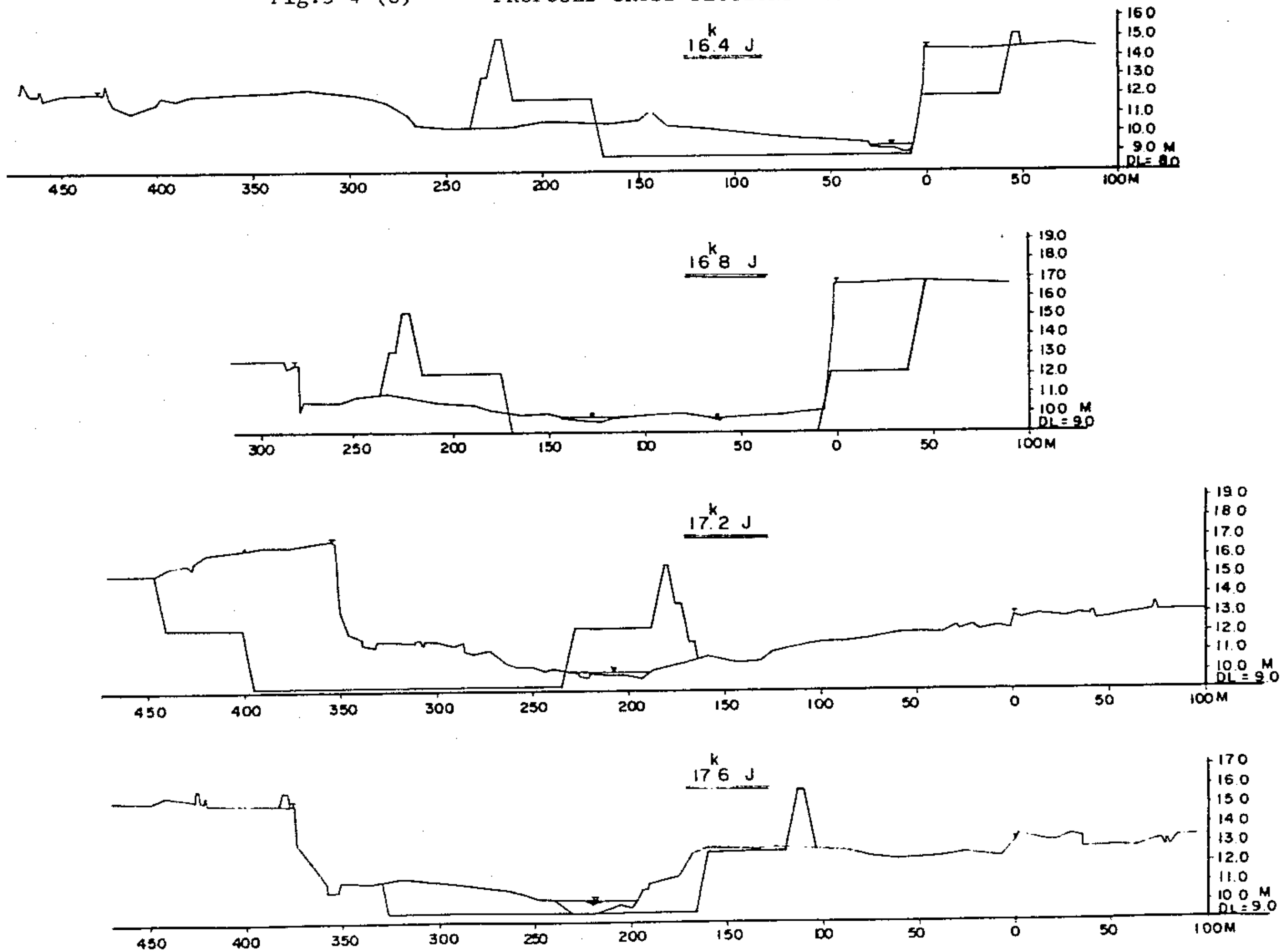


Fig.3-4 (9)　　PROPOSED CROSS-SECTIONS FOR OVERALL PLAN

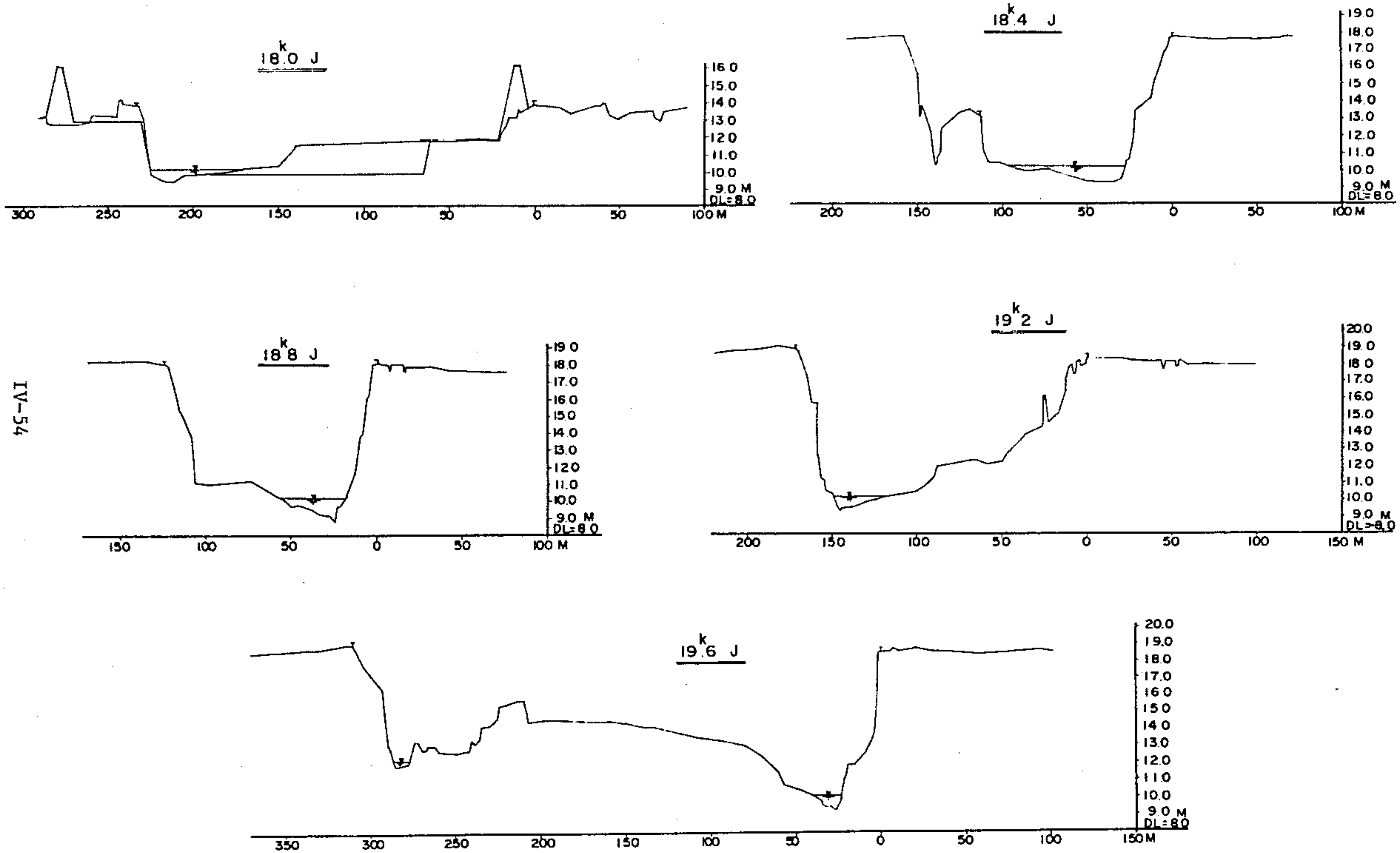
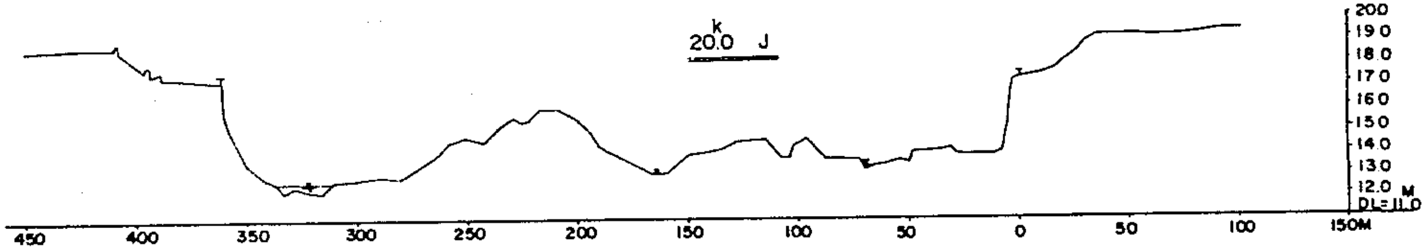


Fig.3-4 (10) PROPOSED CROSS-SECTIONS FOR OVERALL PLAN



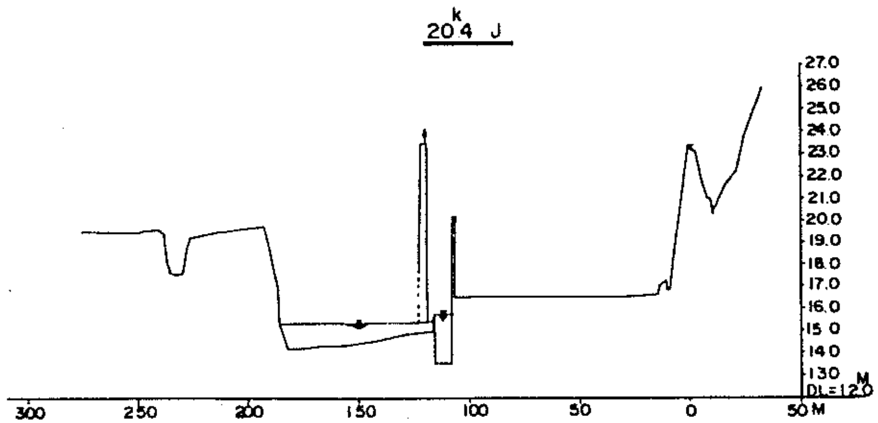


Fig. 3-5 COMPARISON OF TRACTIVE BETWEEN EXISITNG CHANNEL AND PROPOSED CHANNEL

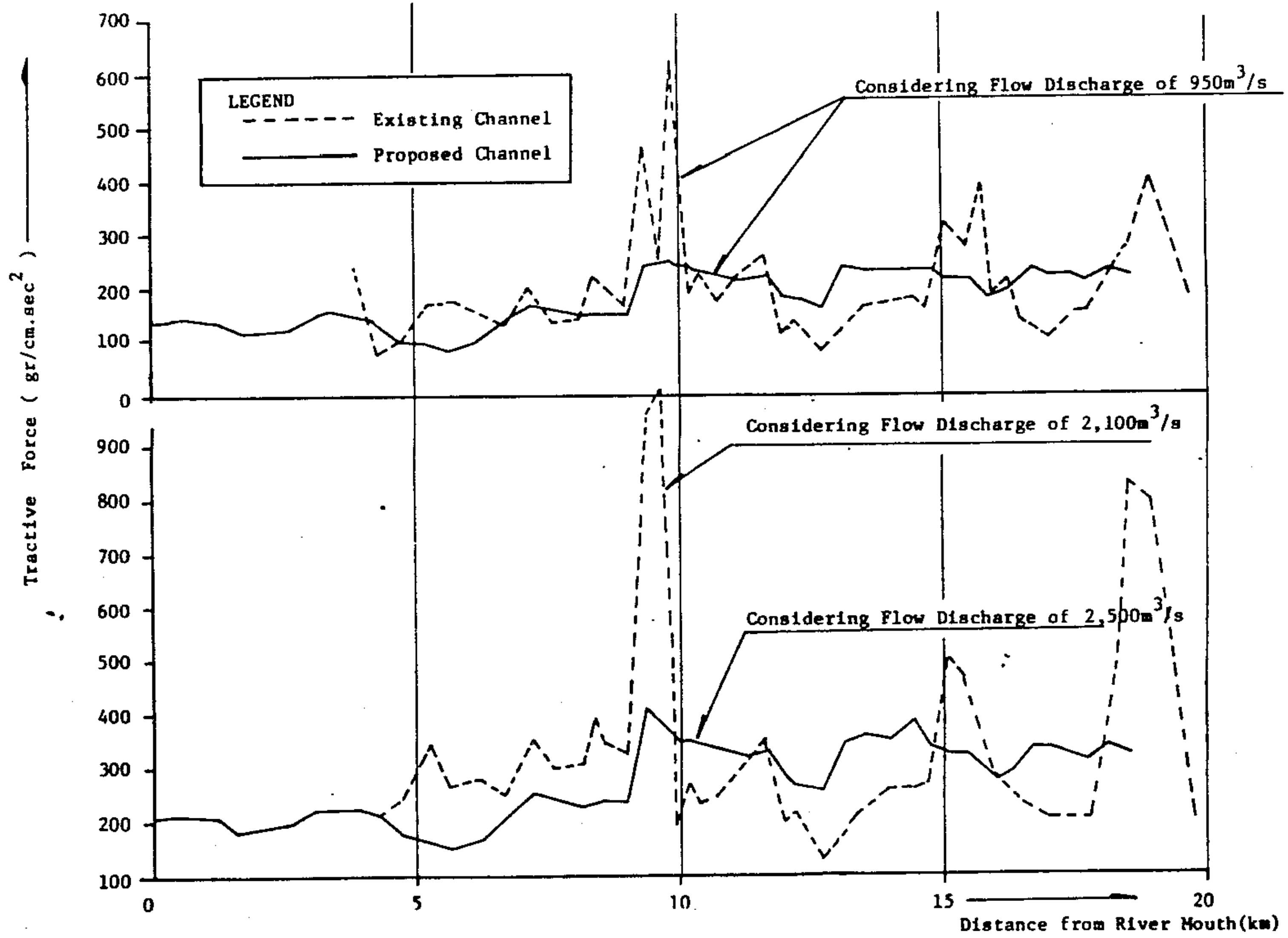


Fig. 3-6 STANDARD CROSS-SECTION OF DIKE (OVERALL PLAN)

Unit : m

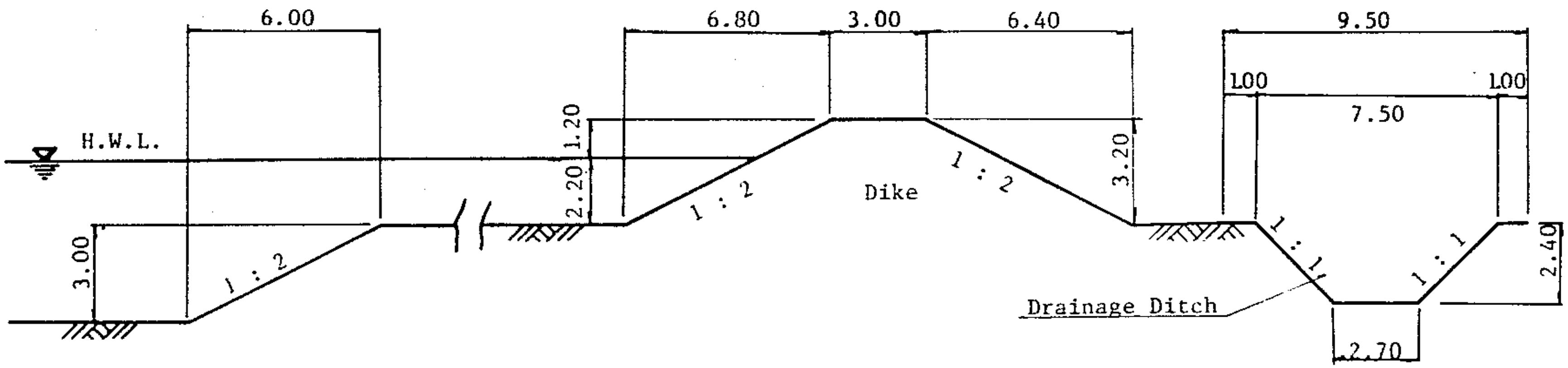


Fig. 3-7 STANDARD CROSS-SECTION OF REVETMENT (OVERALL PLAN)

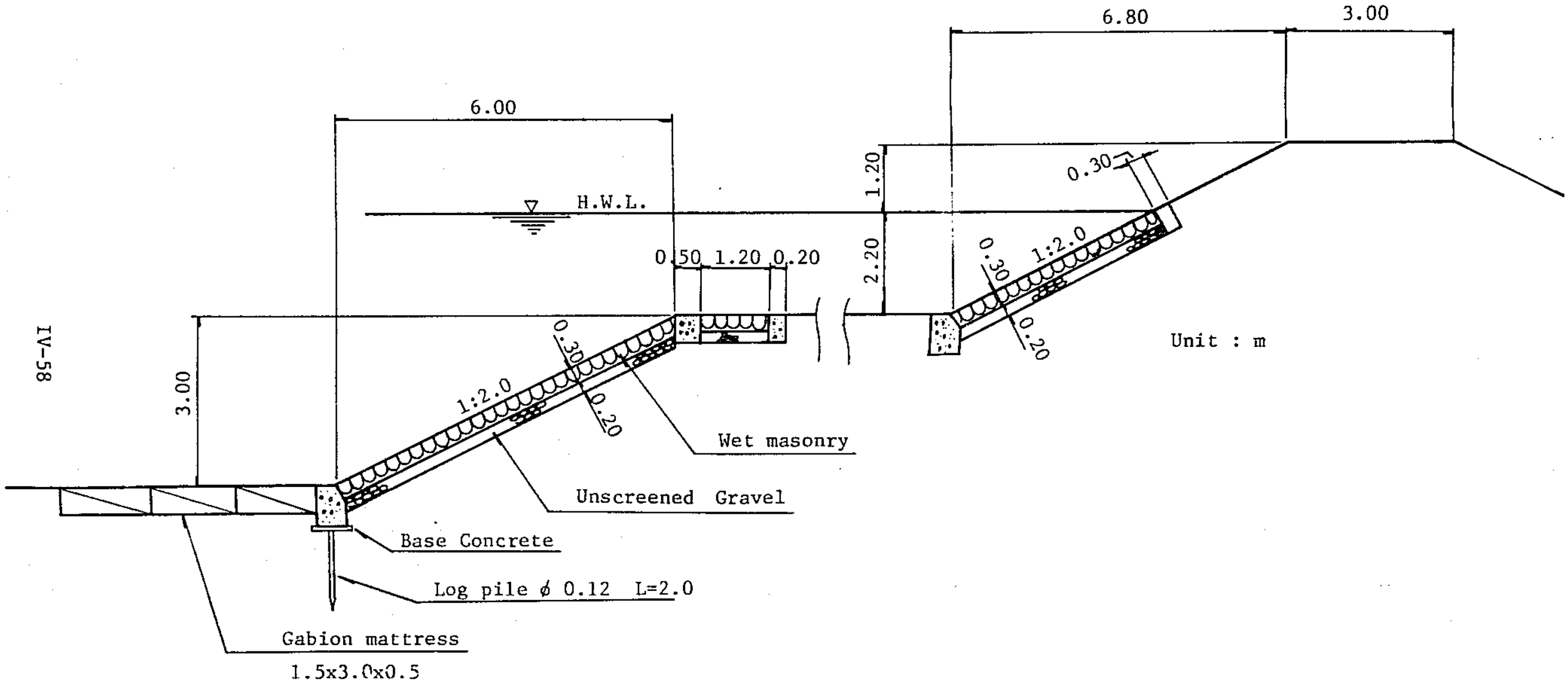


Fig. 3-8 PROFILE AND PLAN OF GROYNE (OVERALl PLAN)

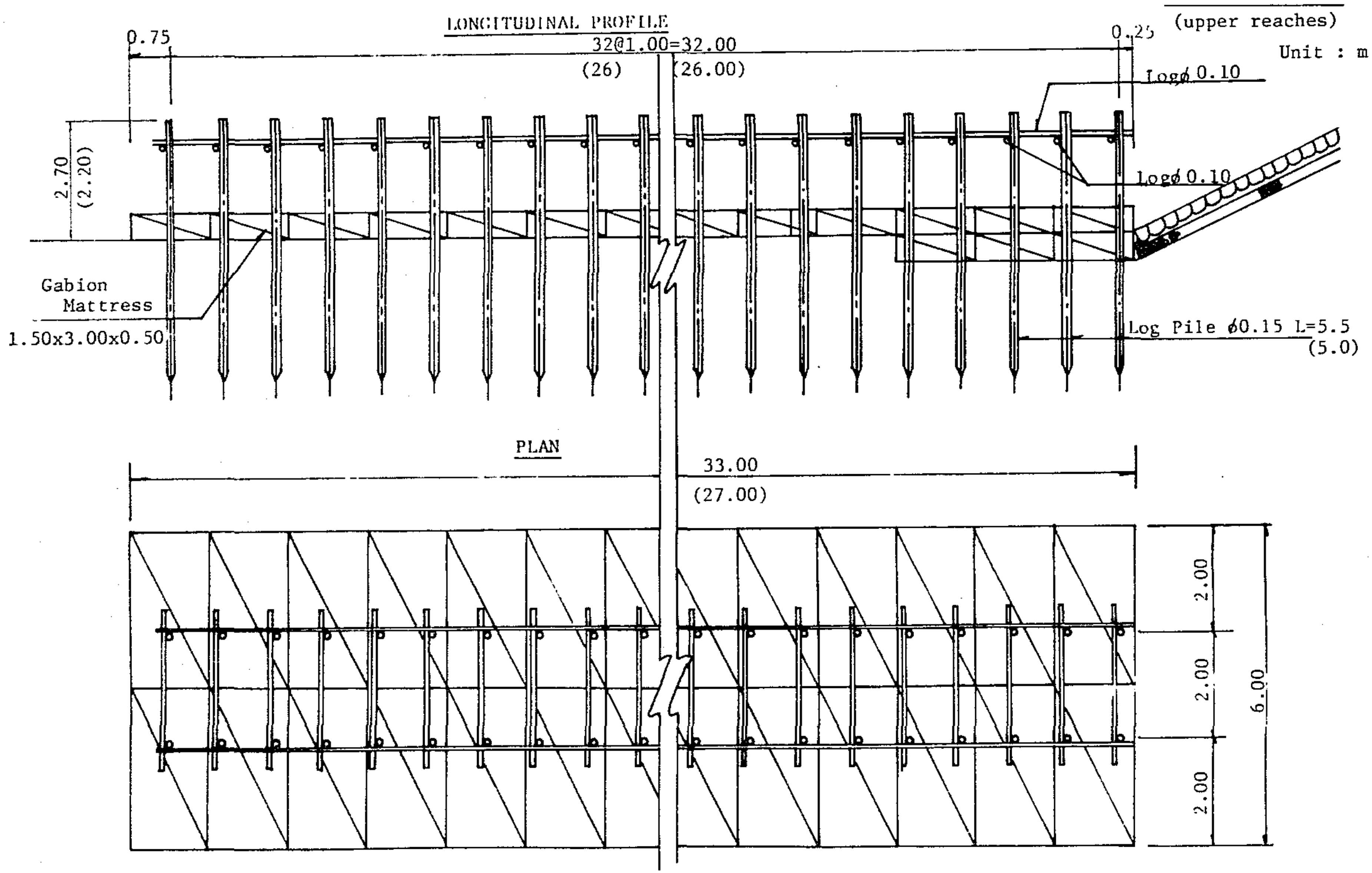
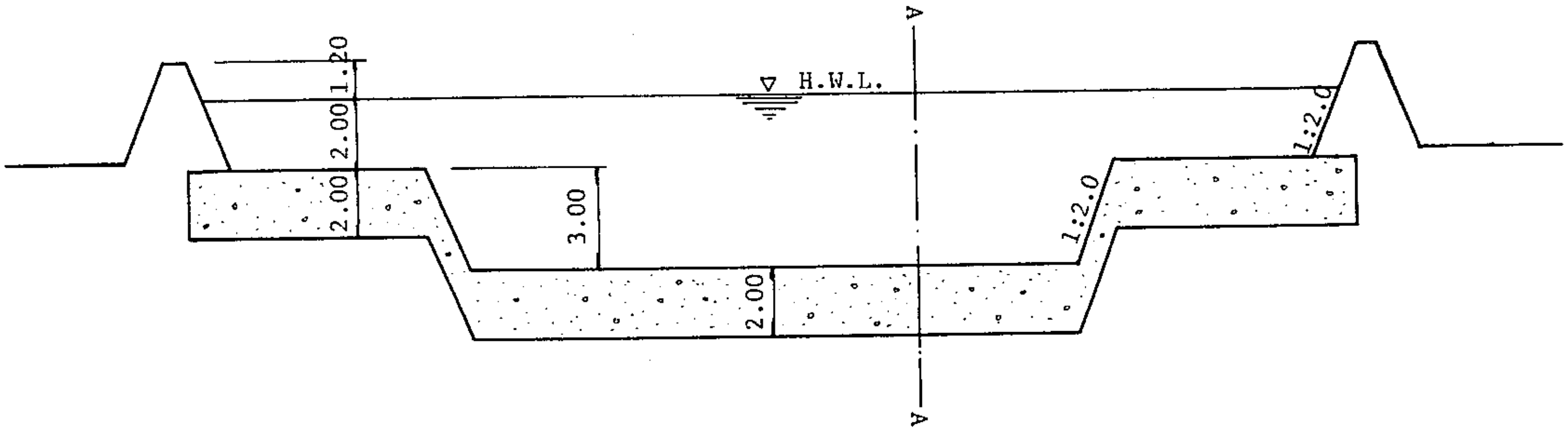


Fig. 3-9 CROSS-SECTION OF GROUND SILL

Unit : m



A - A SECTION

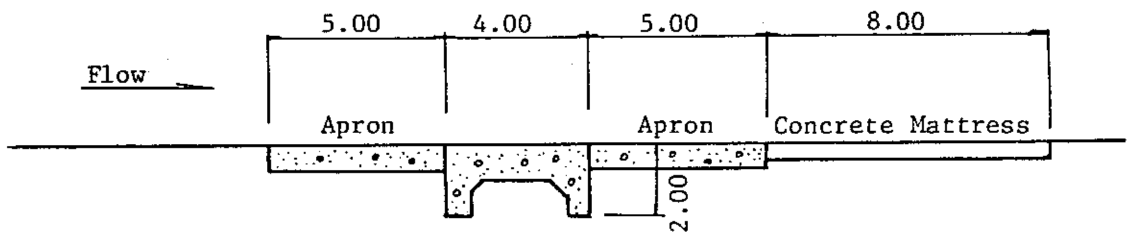


Fig. 3-10 SPECIFIC FLOW-AREA FOR A DRAINAGE CHANNEL

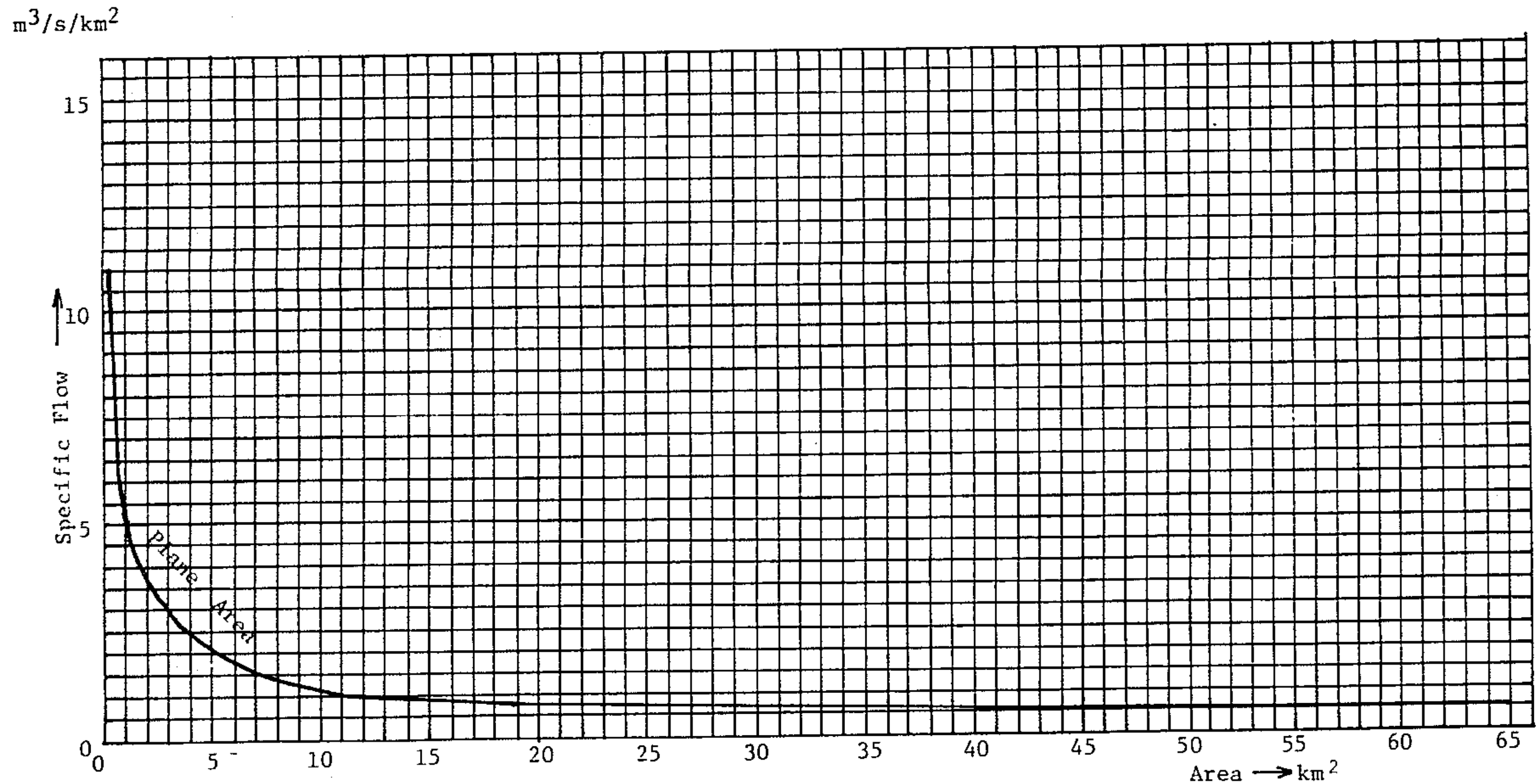


Fig. 3-11 STANDARD DESIGN OF SLUICE (OVERALL PLAN)

Unit : m

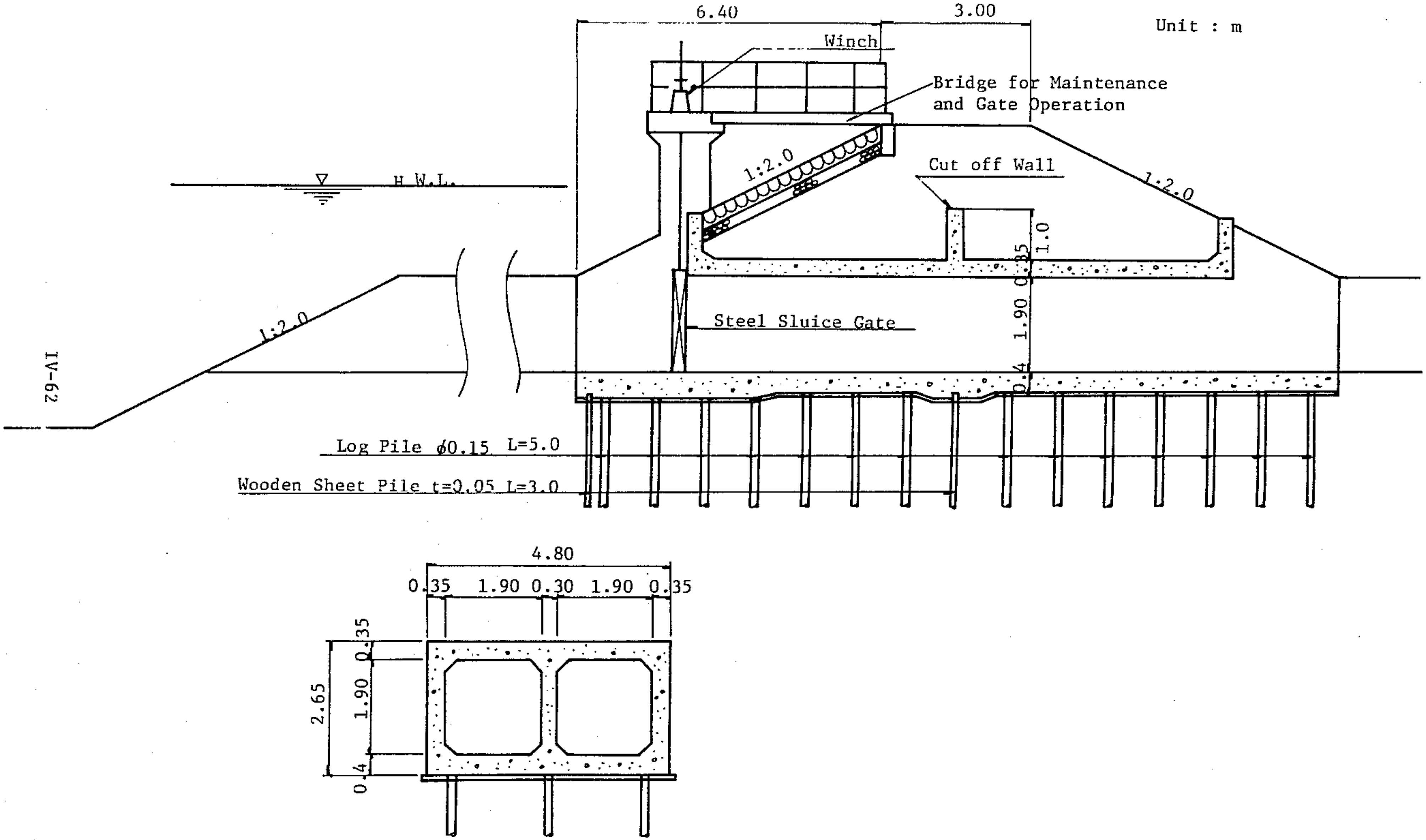


Fig.3-12 IMPROVEMENT PLAN OF SUNGGUMINASA BRIDGE

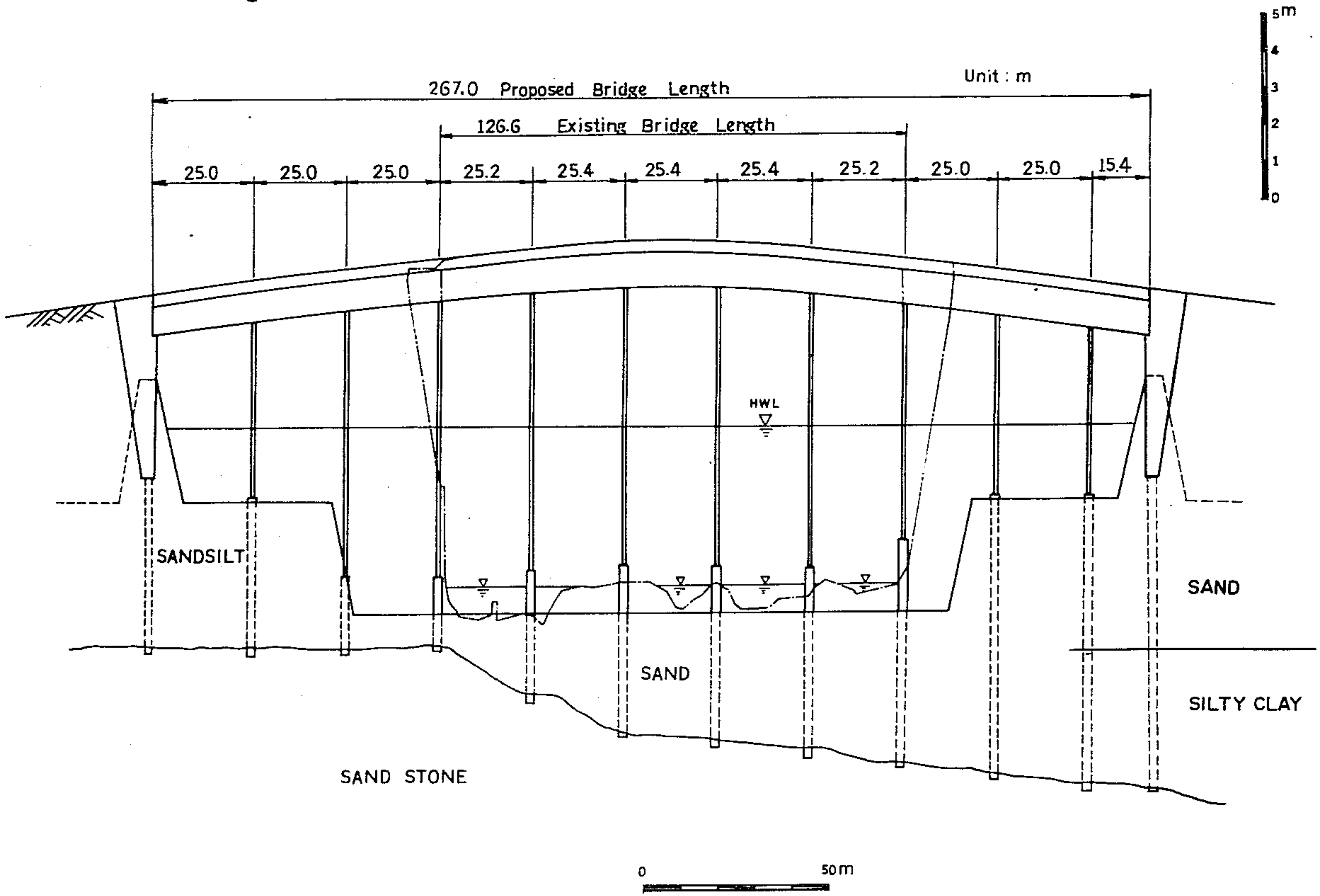


Fig. 4-1 COMPARATIVE STUDY OF RIVER IMPROVEMENT

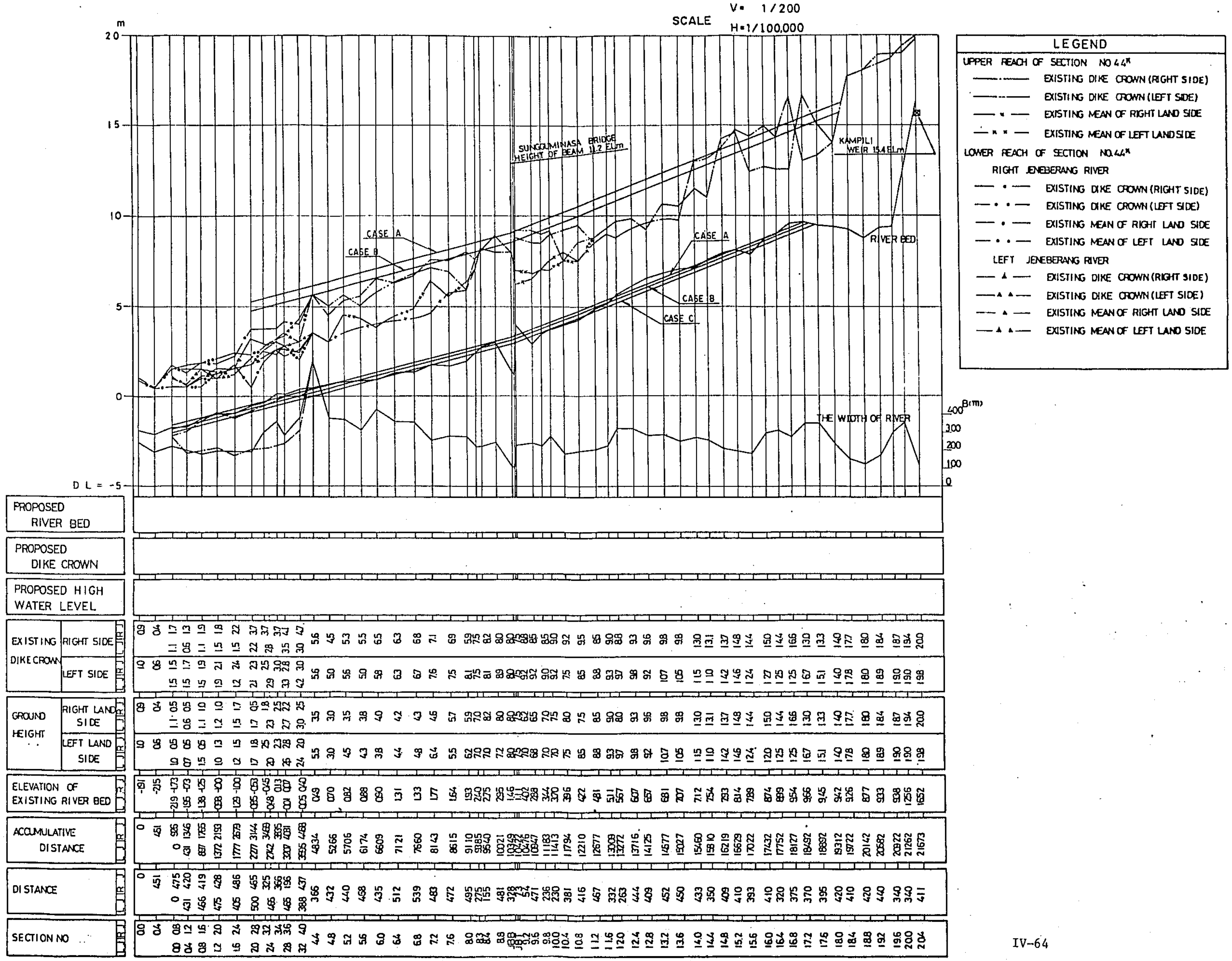


Fig. 4-2 PROPOSED LONGITUDINAL PROFILE OF JENEBERANG RIVER

(URGENT PLAN)

SCALE V= 1/200 H=1/100,000

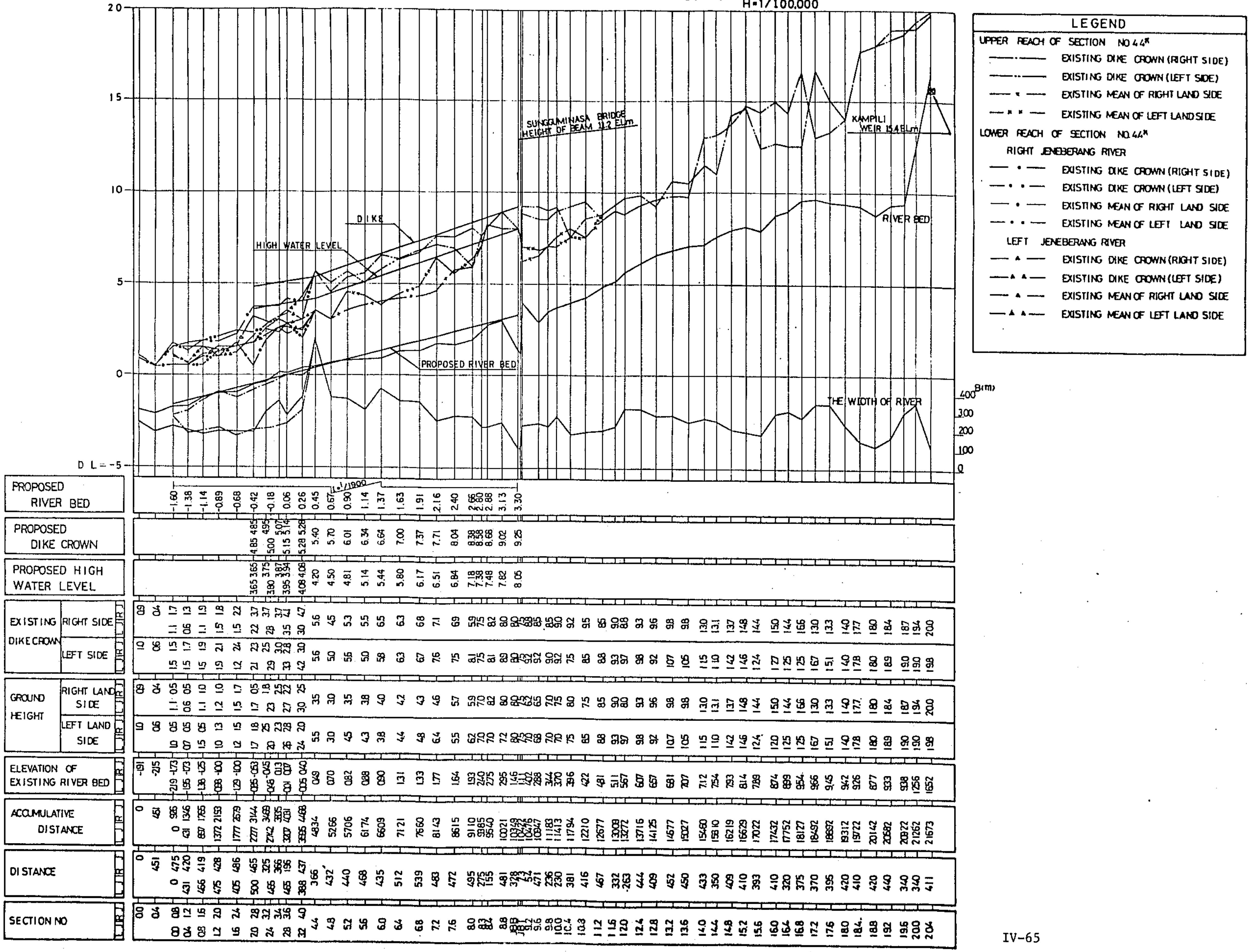
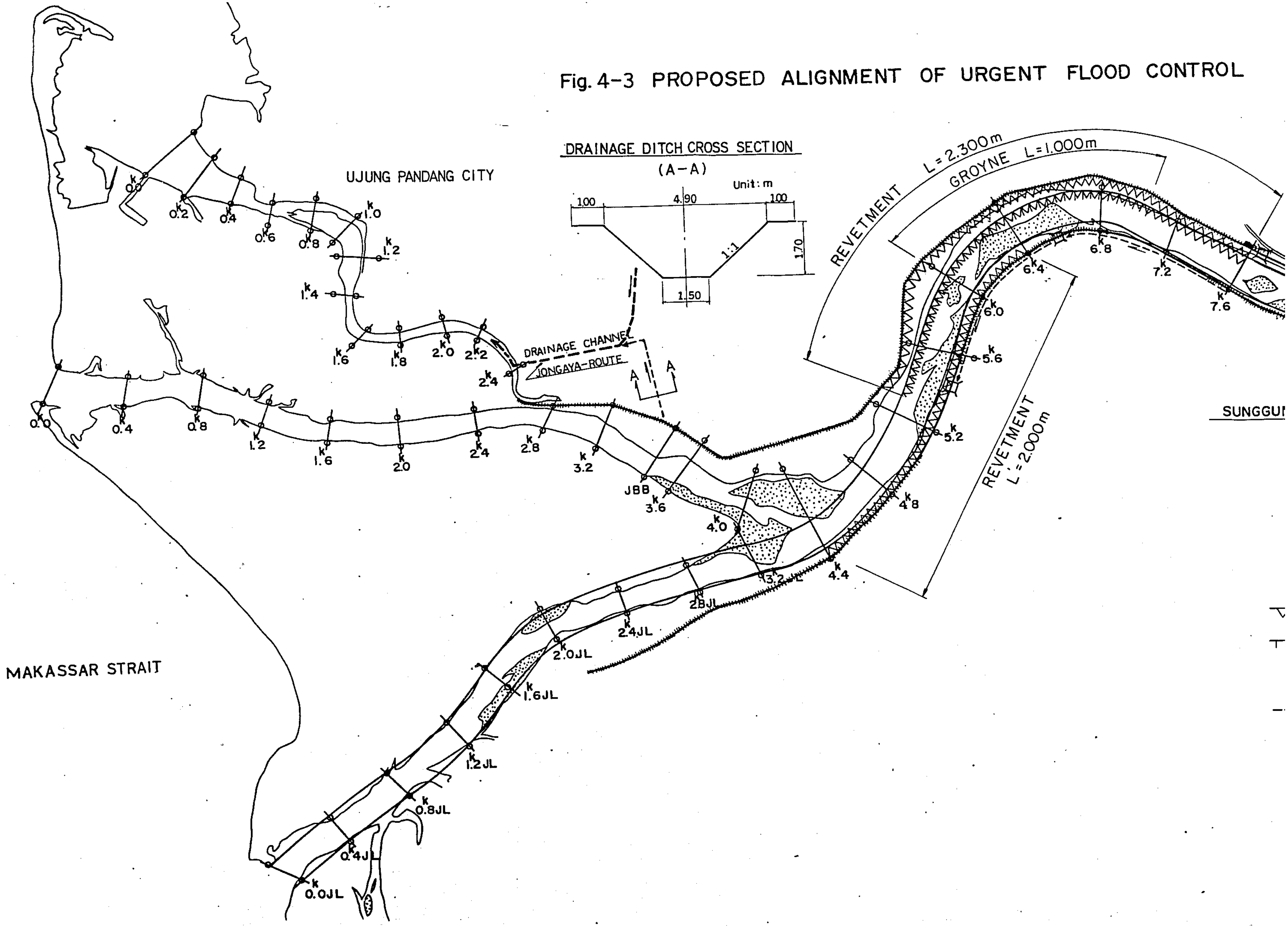


Fig. 4-3 PROPOSED ALIGNMENT OF URGENT FLOOD CONTROL



g. 4-3 PROPOSED ALIGNMENT OF URGENT FLOOD CONTROL

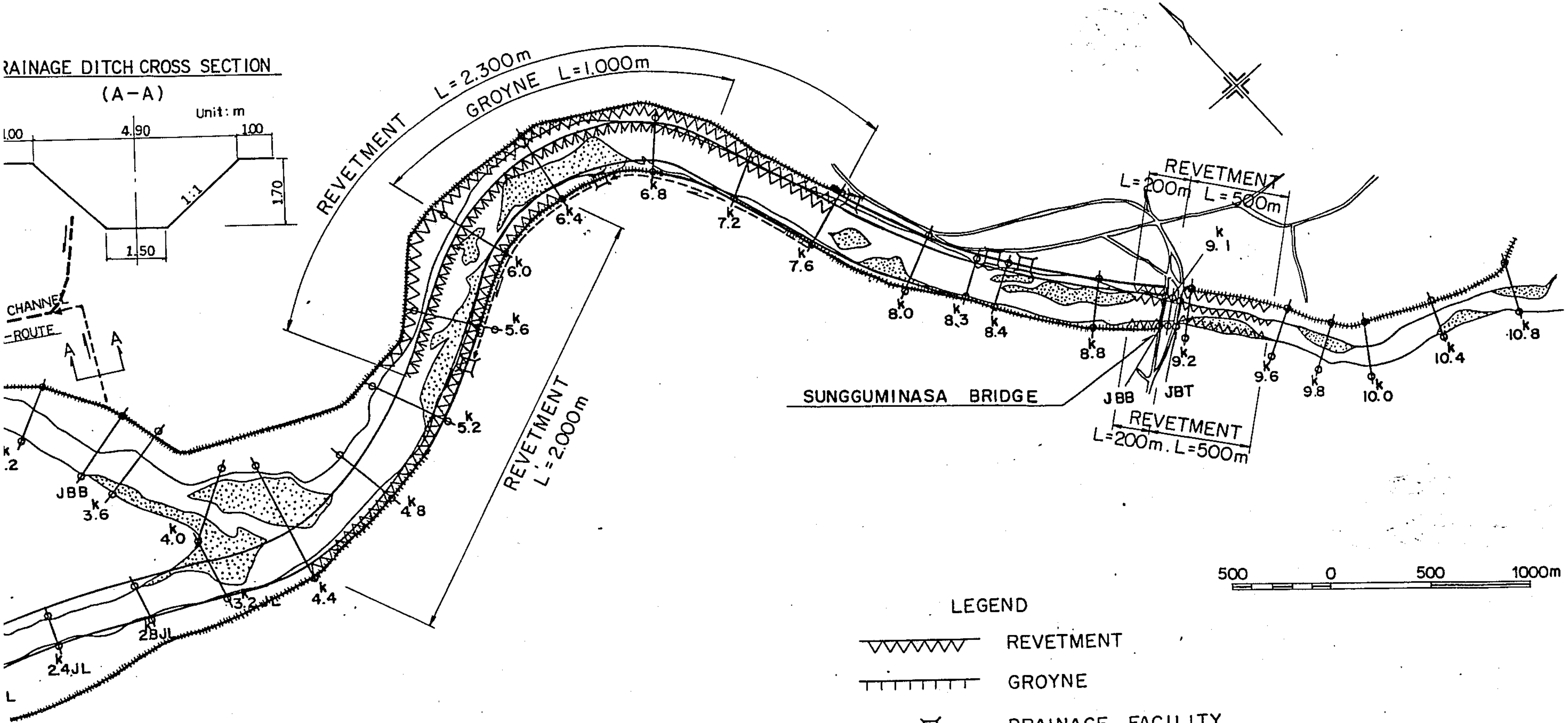
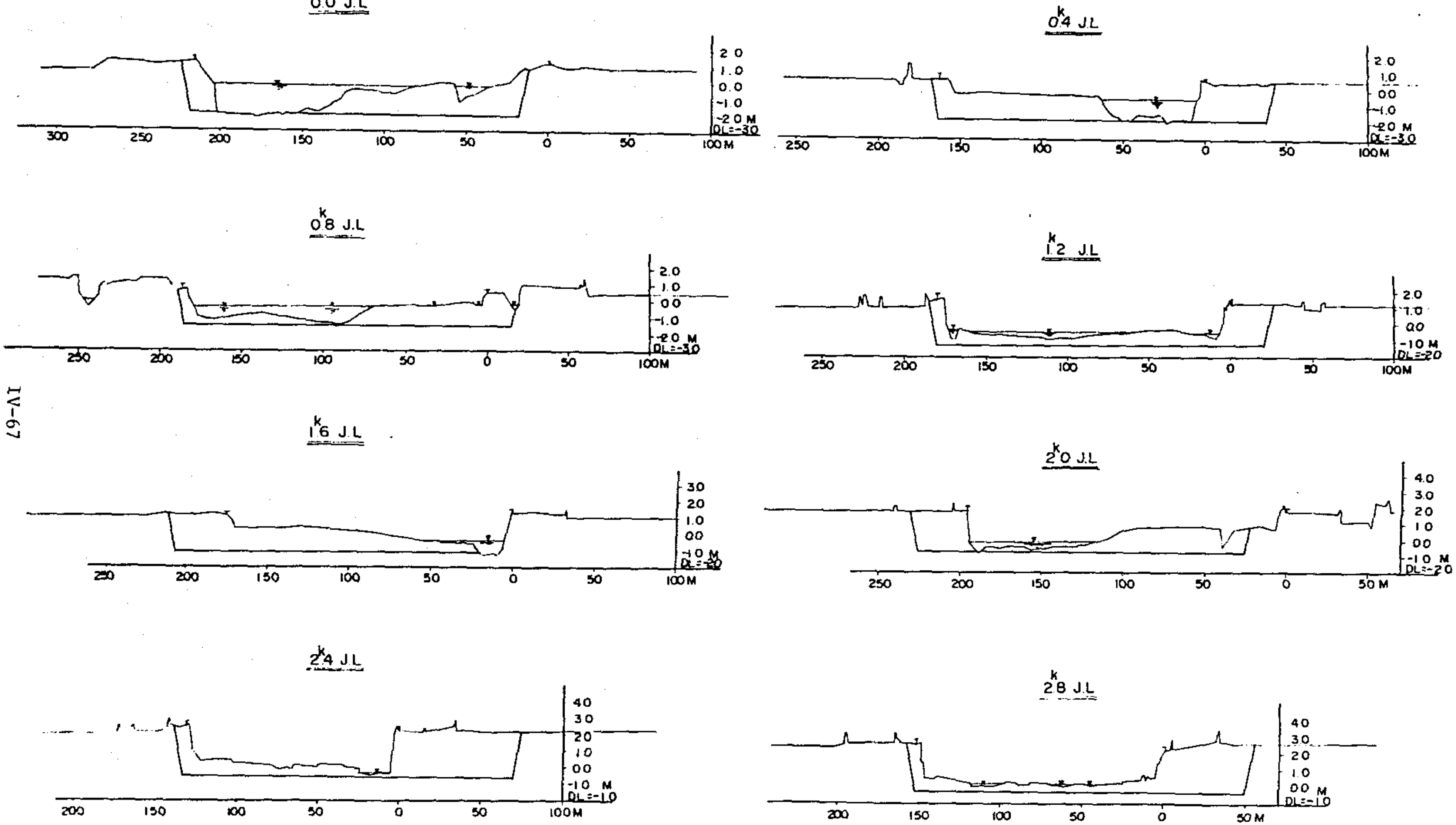


Fig.4-4 (1) PROPOSED CROSS-SECTIONS FOR URGENT PLAN



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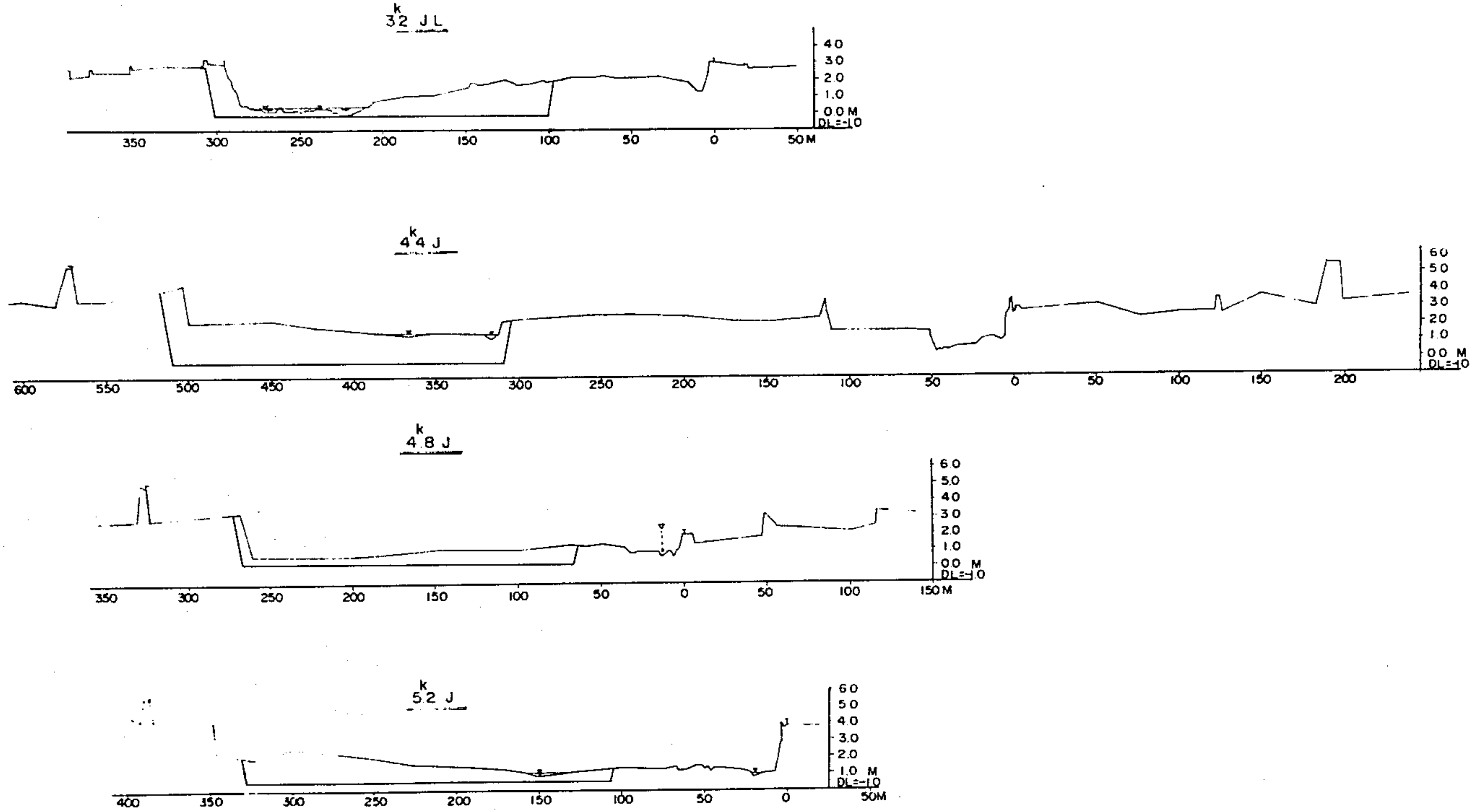

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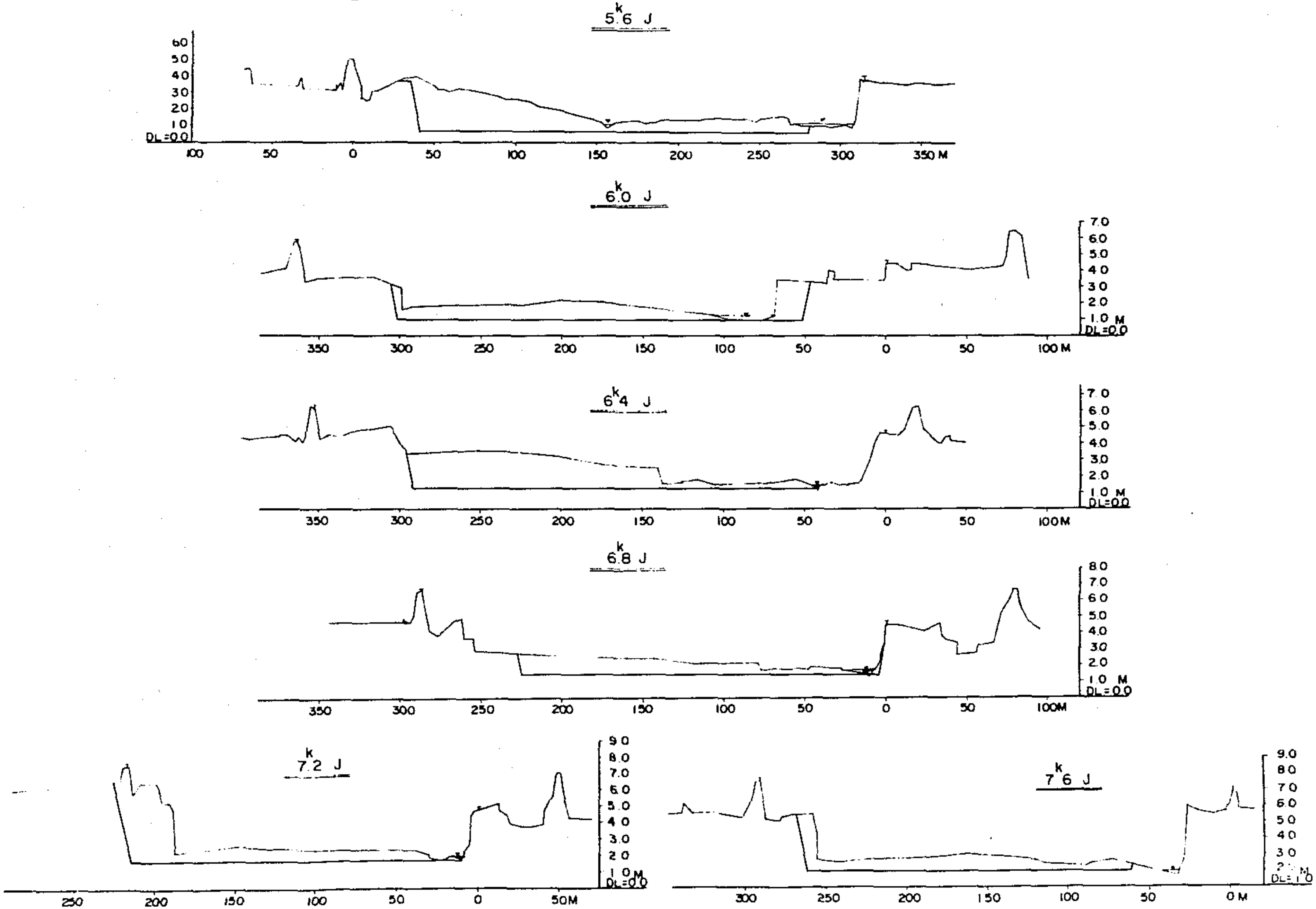


Fig.4-4 (4) PROPOSED CROSS-SECTIONS FOR URGENT PLAN

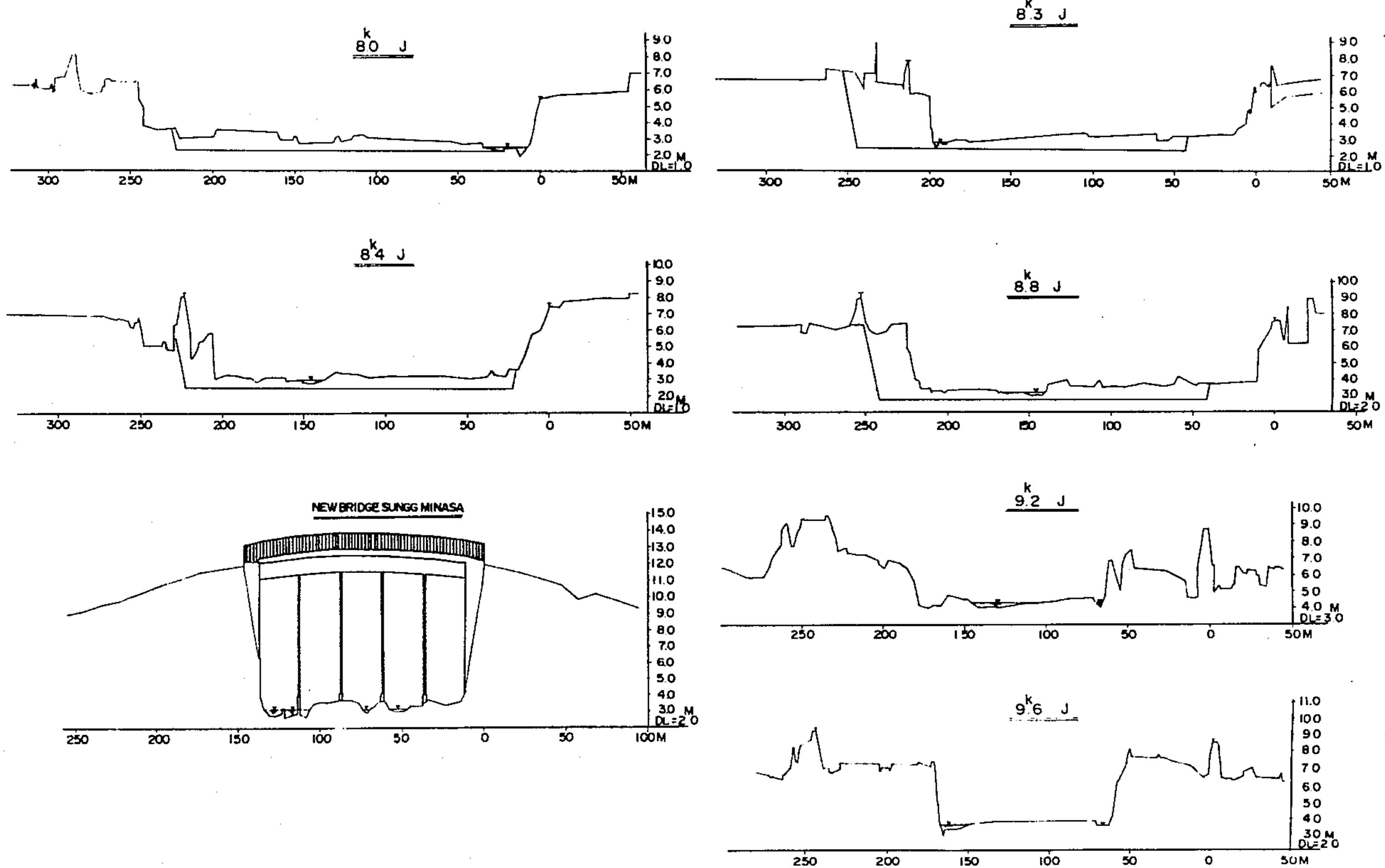


Fig. 4-5 NATURAL BUFFER

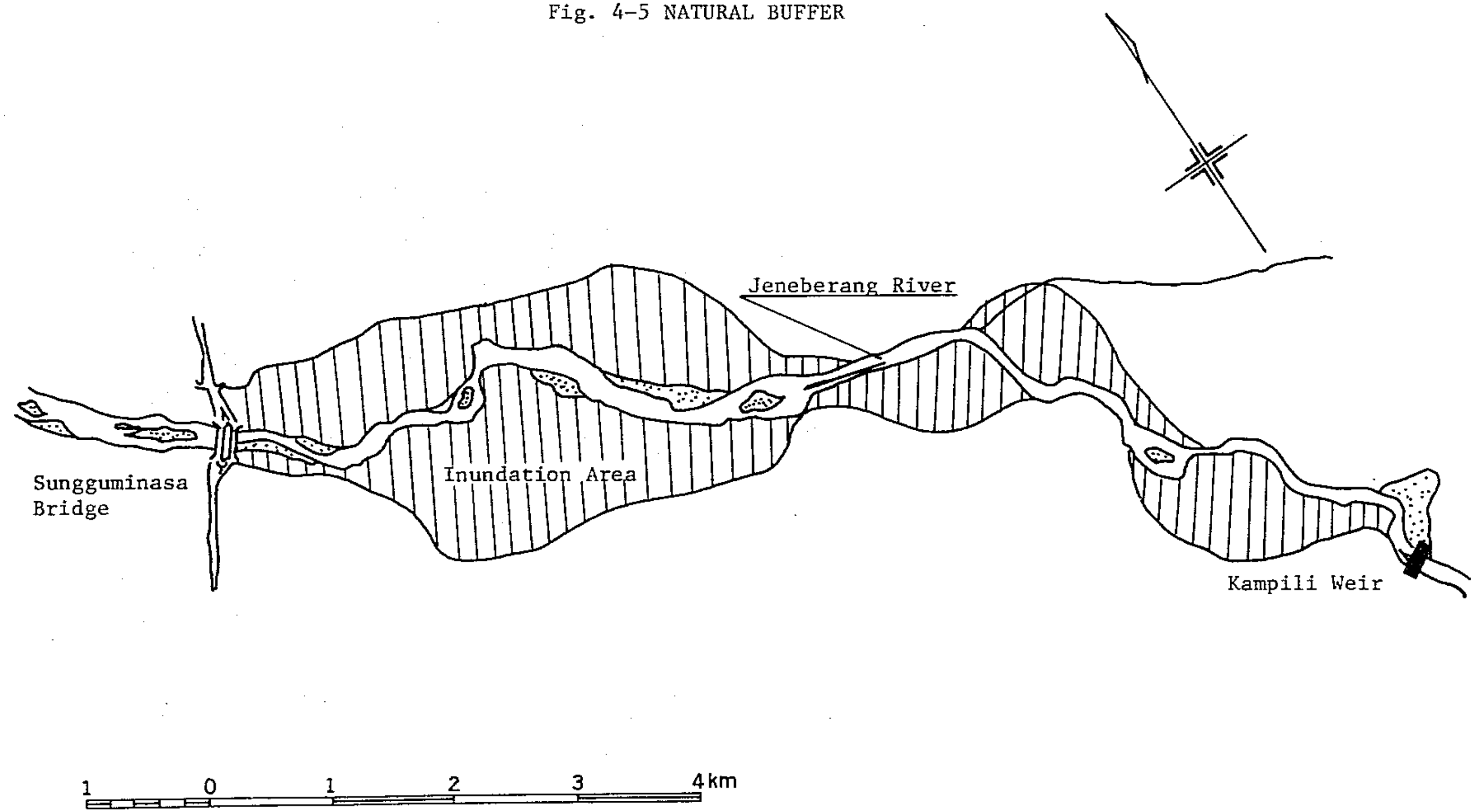


Fig. 4-6 ROAD RAISING SECTION

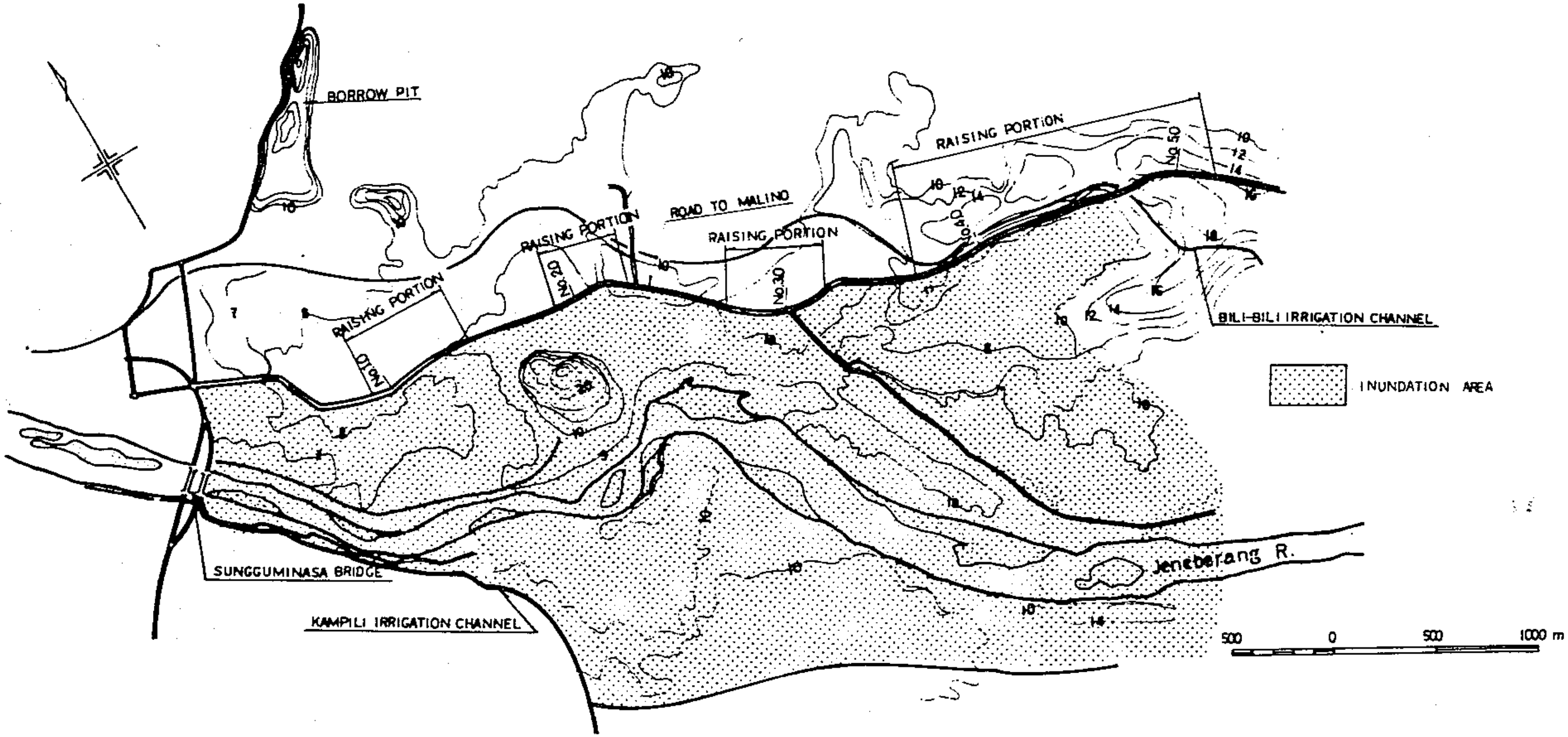


Fig. 4-7 LONGITUDINAL PROFILE OF ROAD RAISING

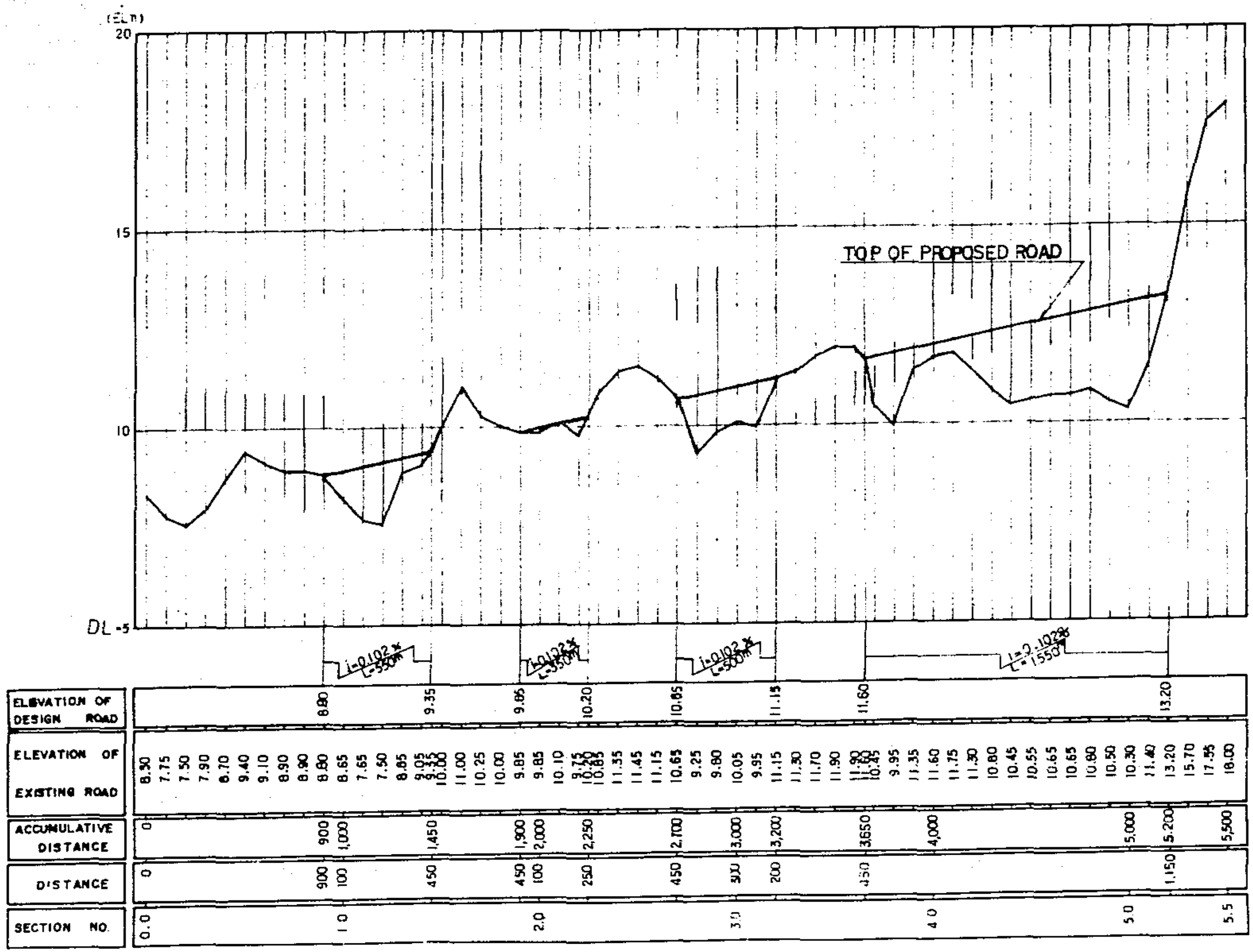
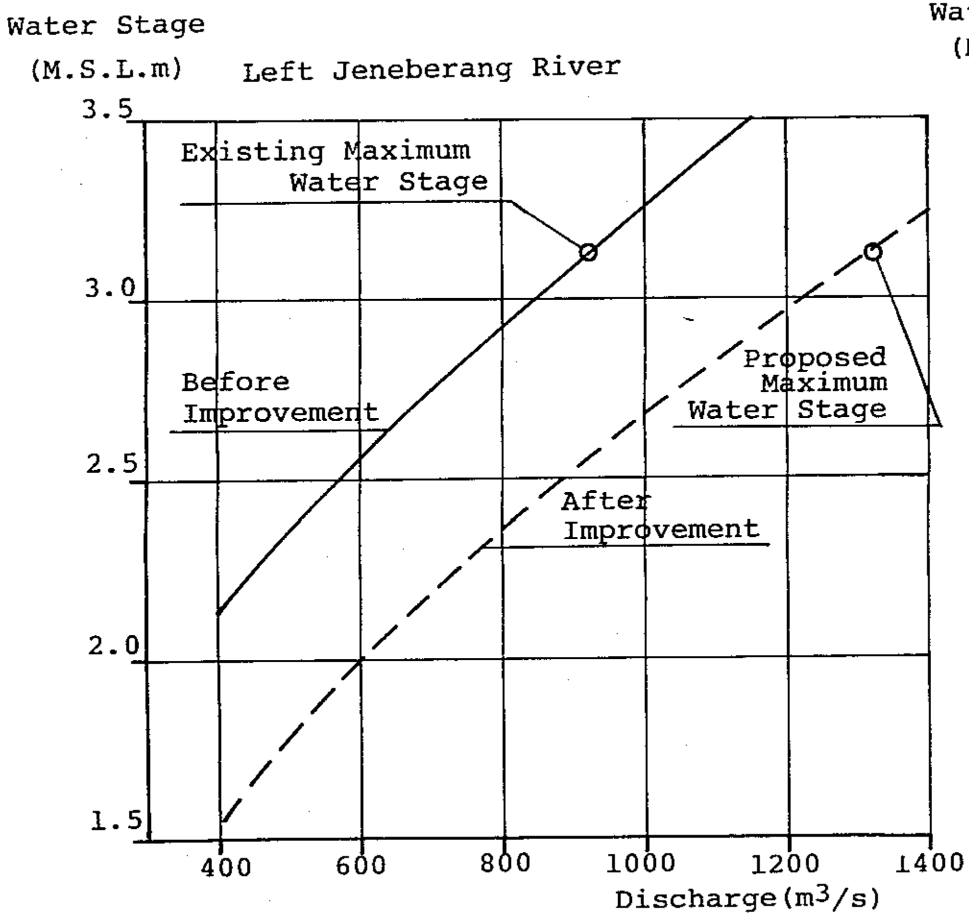
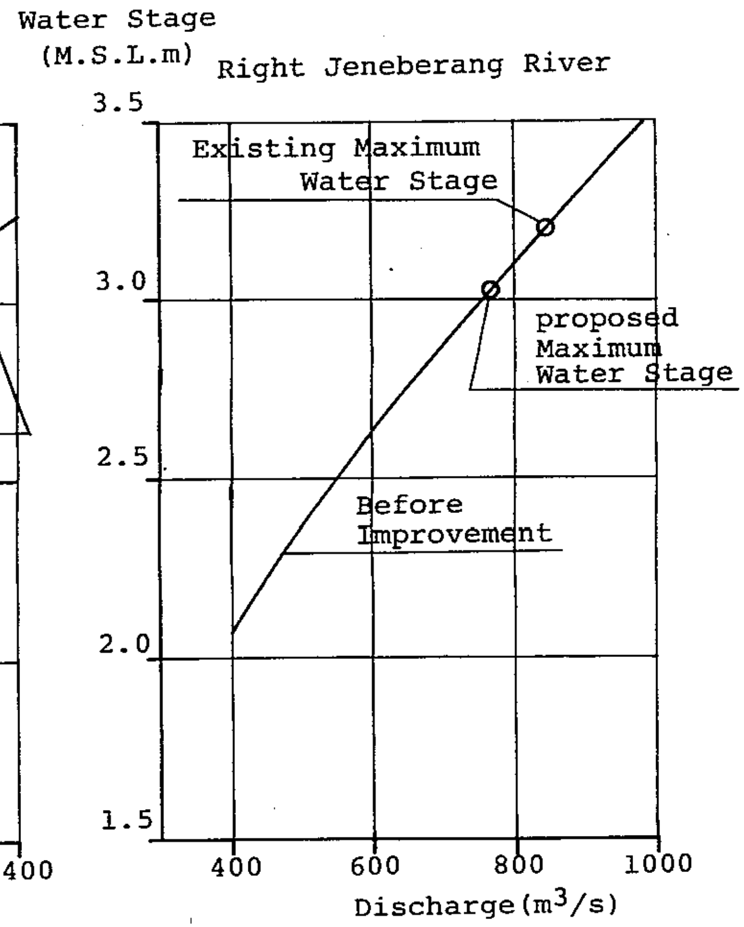


Fig. 4-8 RATING CURVE AFTER RIVER IMPROVEMENT
(SECTION NO. 2.0K)

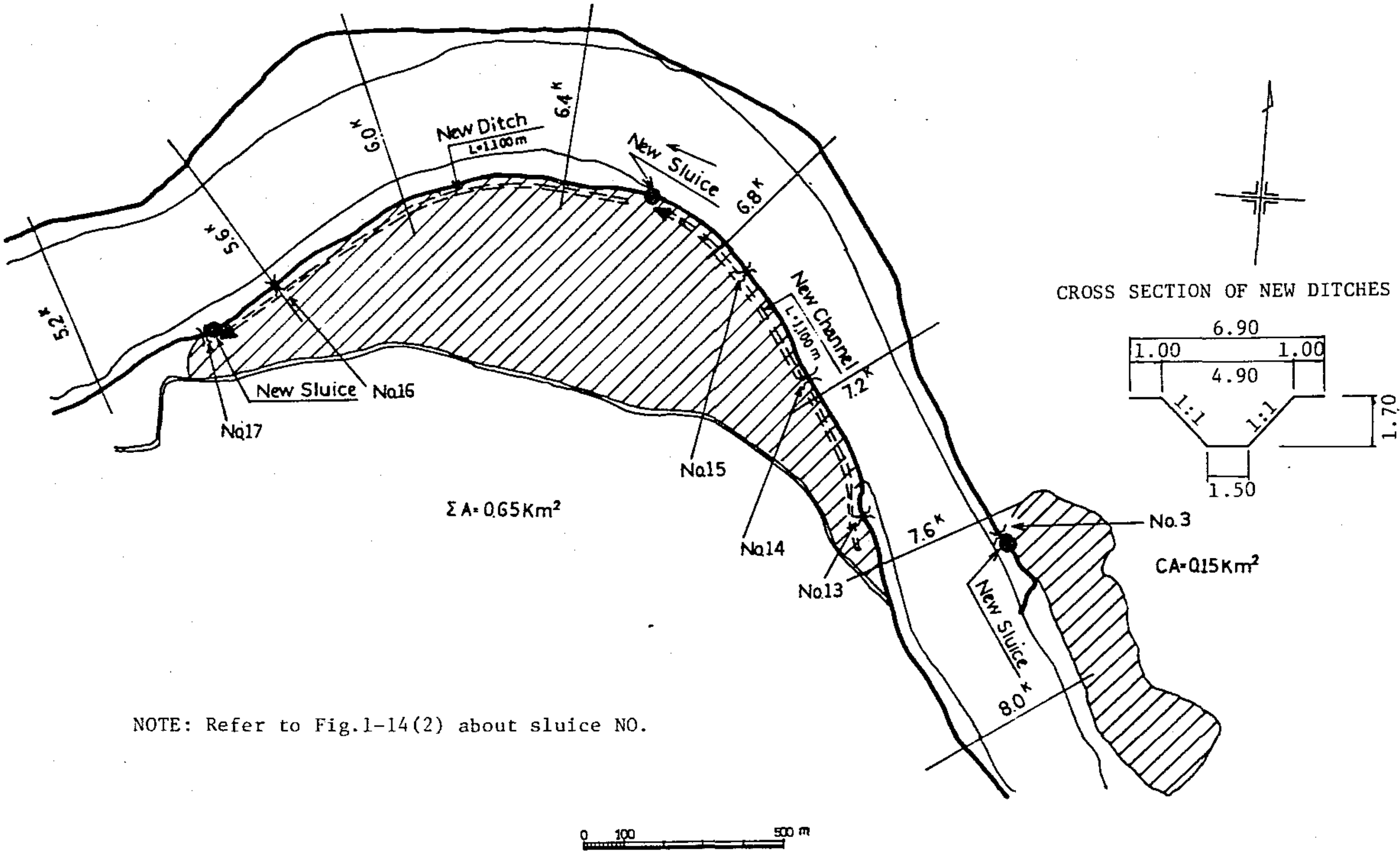




	Discharge (m^3/s)	Water Stage (M.S.L.m)
Before Improvement	950	3.13
After Improvement	1,330	3.11

	Discharge (m^3/s)	Water Stage (M.S.L.m)
Before Improvwment	850	3.20
After Improvement	770	3.05

Fig. 4-9 LOCATION OF THE PROPOSED SLUICE AND DITCHE



NOTE: Refer to Fig.1-14(2) about sluice NO.

Fig. 4-10 LOCATION OF THE NEW DITCH

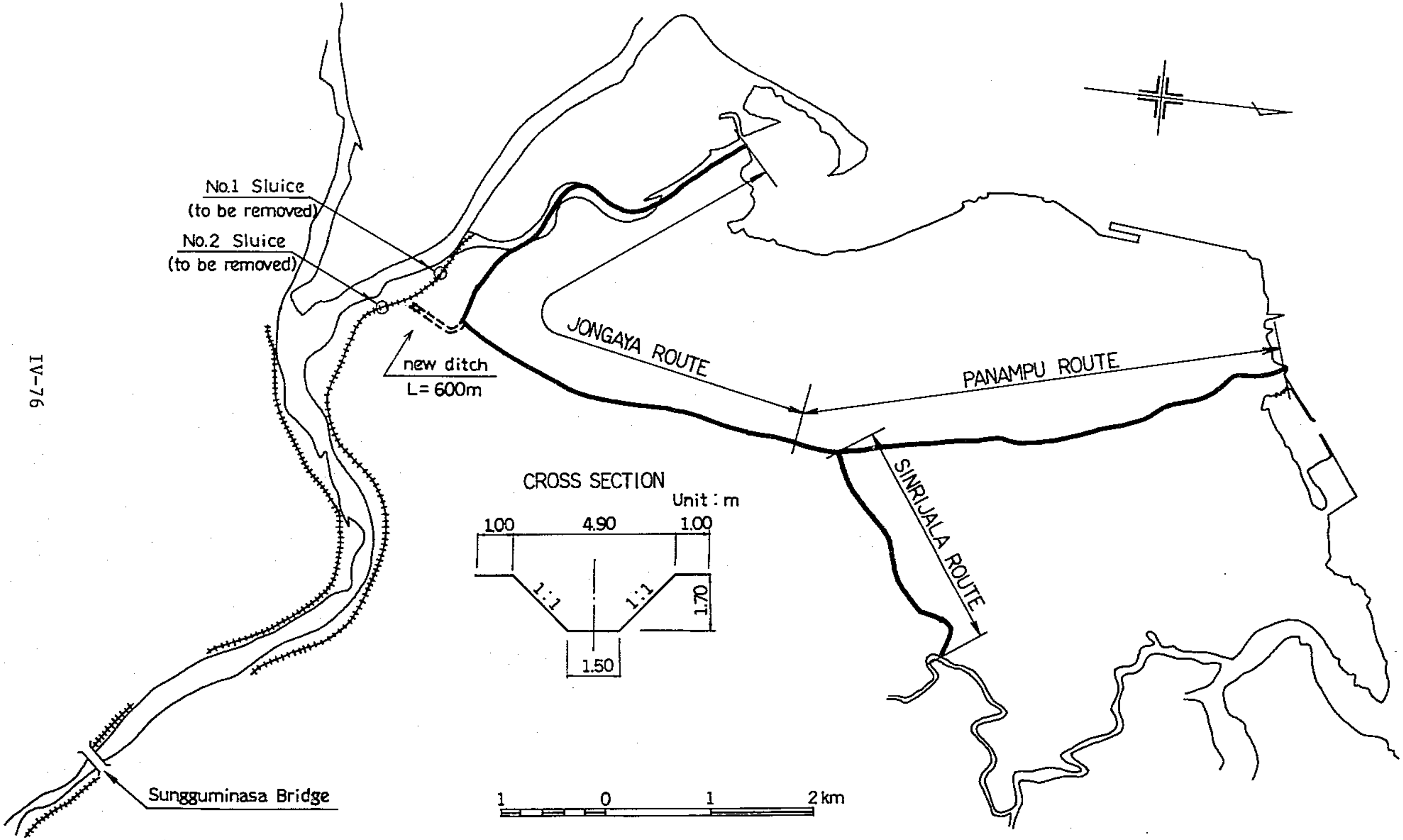


Fig. 4-11 PROFILE AND PLAN OF GROYNE (URGENT PLAN)

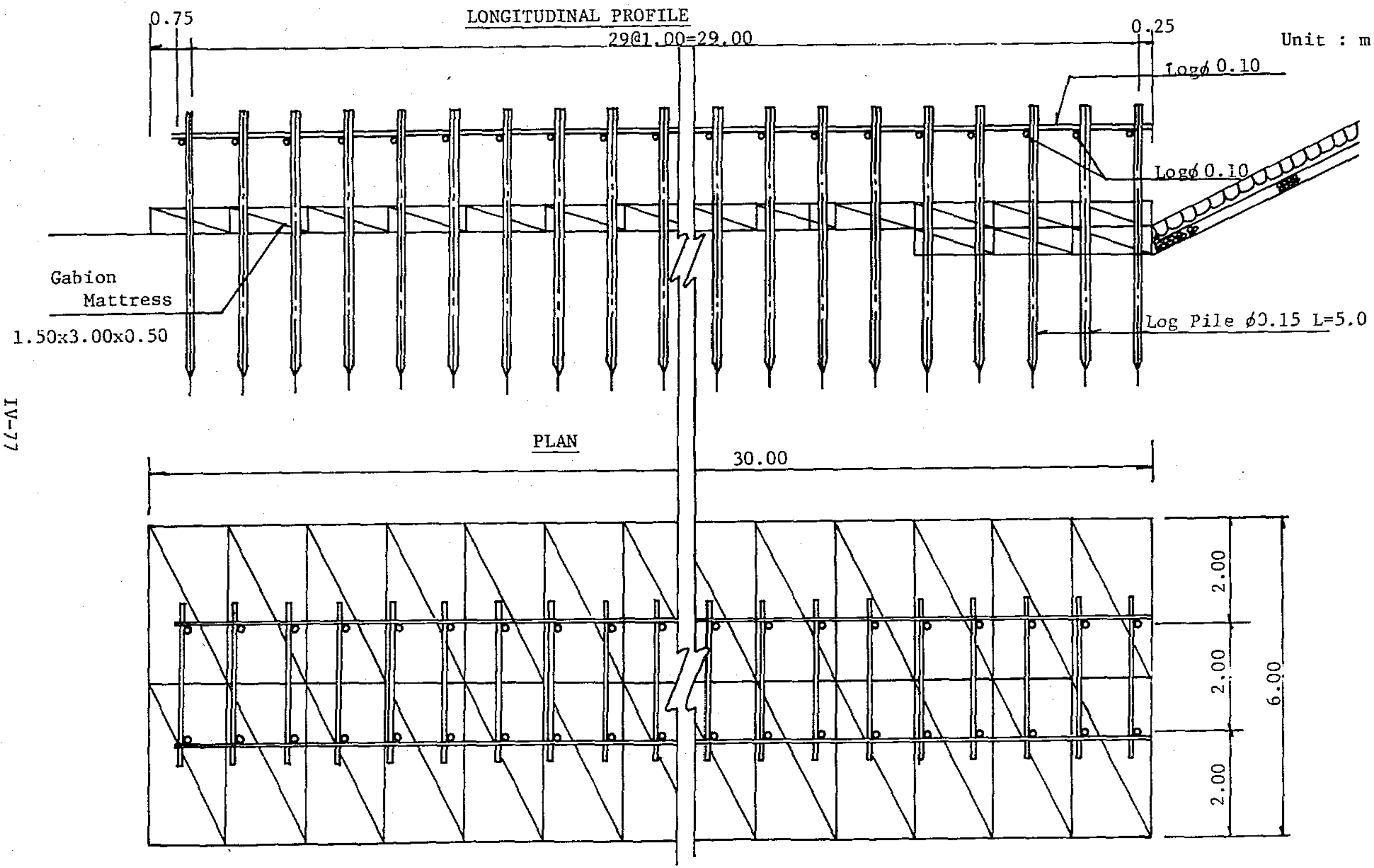
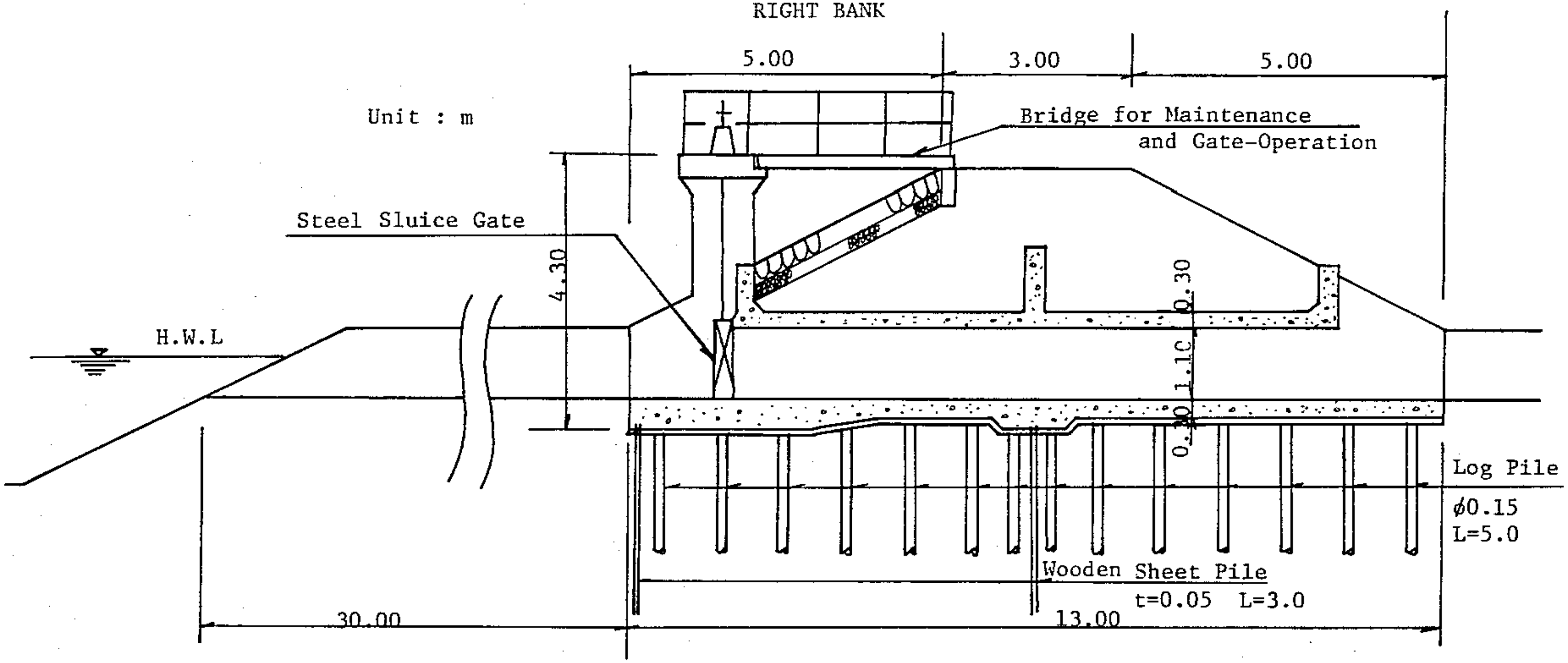


Fig. 4-12 STANDARD DESIGN OF SLUICE (URGENT PLAN)

RIGHT BANK



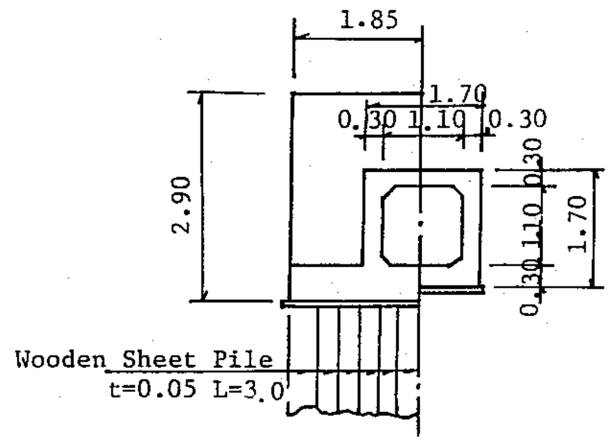


Fig.4-13 STANDARD DESIGN OF SLUICE (URGENT PLAN)
LEFT BANK

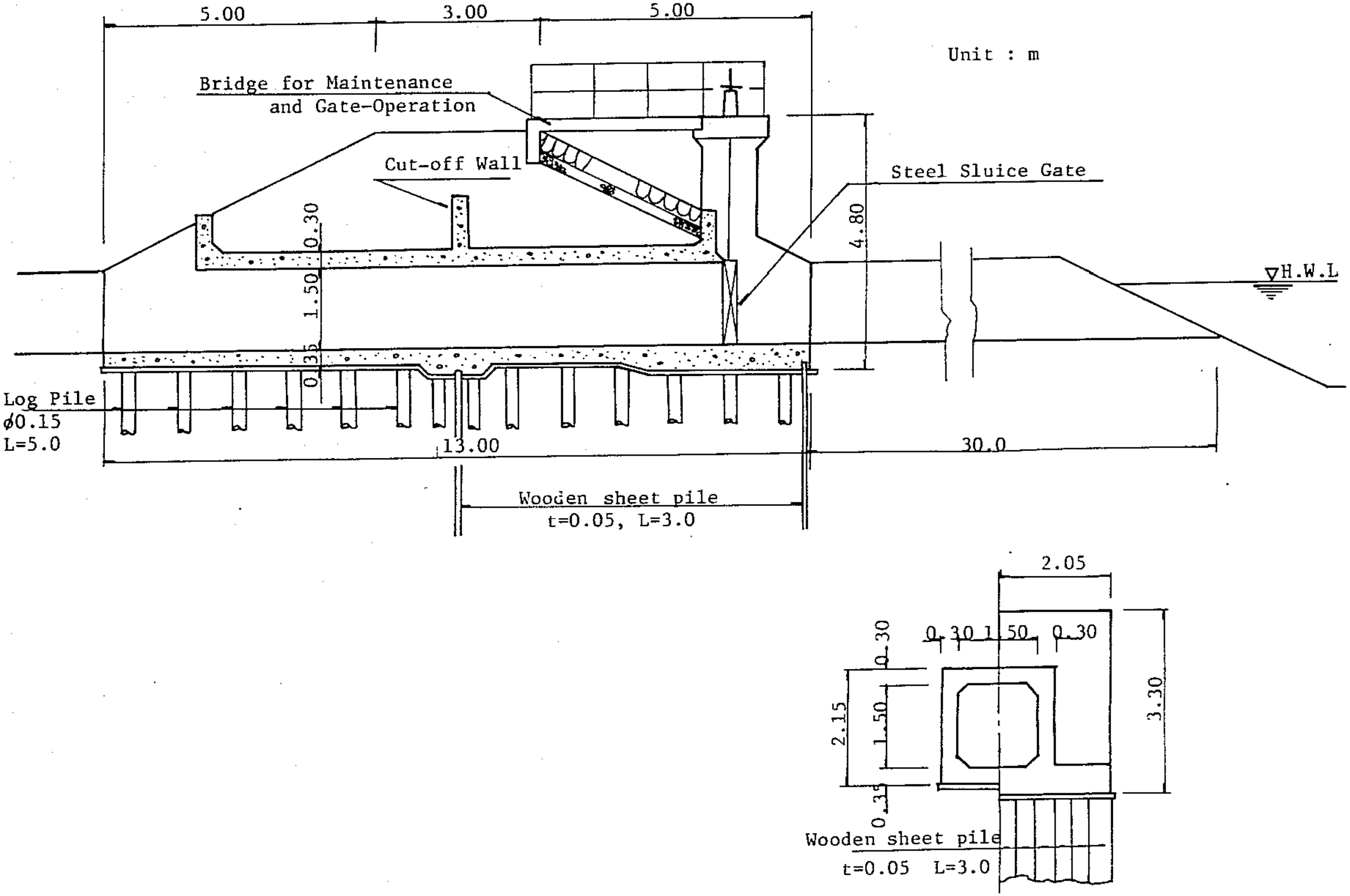
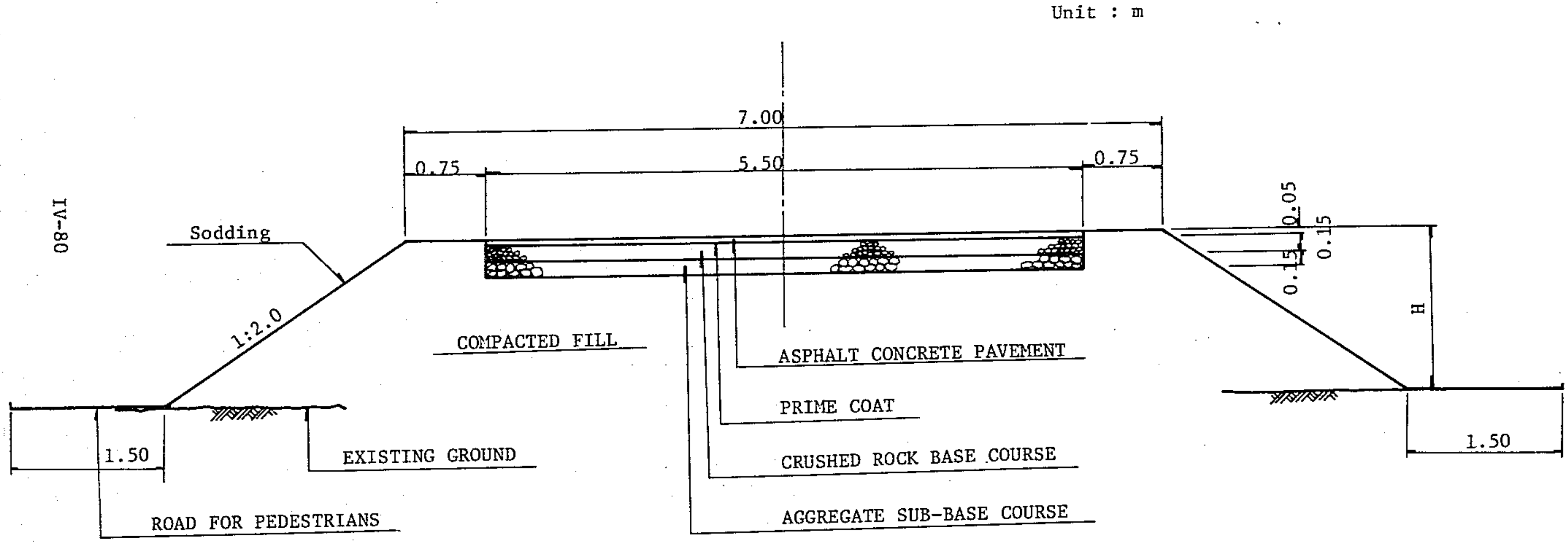


Fig. . 4-14 STANDARD CROSS-SECTION FOR ROAD RAISING



V DAM AND RESERVOIR

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1. SELECTION OF DAM SITE

1.1 Nomination of Possible Dam Site

Taking topographical and geological conditions into consideration, Bili-Bili, Pasaratowaya and Jonggoa on the Jeneberang river, shown in Fig. 1-1, are nominated as the sites for the proposed dam.

The selection of a dam site is based on the technical, economical and environmental conditions when each reservoir is developed in maximum.

A topographical map of the project area (scale: 1/50,000) is used for the computation of elevations and storage capacities of the respective reservoirs. (Refer to Figs. 1-2, 1-3 and 1-4.)

With regard to the geology in the above three dam sites, "Geology" in the Supporting Report can be referred to in detail. Table 1-1 shows the feature of the representative dam in Indonesia.

The location and dam heights of the nominated dams are determined by the facts as below.

<u>Bili-Bili Dam Site</u>

The dam site is located 31 km upper from the estuary of the Jeneberang river. The width of the river at the proposed dam site is about 200 m, which is the narrowest in the middle reaches, and the dam site is located at the down-most stream of the three nominated dam sites.

The Bili-Bili dam consists of a main dam linking two hills and also of two wing dams. The main dam will close the Jeneberang river, and the right wing dam will close the saddle through which Jl. Malino passes, while the left one will close the saddle along the water-shed of the Jenelata river (refer to Fig. 1-5.) The height of the crest is fixed at about 55 m above the riverbed (65 m above its fundation), the high water level is determined at EL. 100 m for the following reasons.

1) Since the height of the hill located at the dam site is EL. 100 m, construction of the spillway on the top of the hill is practical and economical.

2) It is economical to reduce the total volume content of the dam by utilizing the two hills for the downstream rock zone of the dam body.

3) The left side area 7 km upper from the dam site forms a water shed of the Jenelata river, a tributary of the Jeneberang river. This area has the elevation of about EL. 125 m. If the dam is designed higher than the proposed one, a coffer dam will be required in this area and its construction cost is very high.

4) Arakkuyu village with about 50 households located in the upper reaches of the reservoir will be submerged, if the resevoir stage exceeds EL. 100 m.

Pasaratowaya Dam Site

The dam site is located 44 km upper from the estuary of the Jeneberang river. The location is down-stream of the confluence of the Rakikang river and the Jeneberang river where there is a protruding low ridge. The river width at this site is about 700 m. The crest height of the dam is about 70 m above the river bed, and the high water level is EL. 185 m (refer to Fig. 1 - 6). These are considered appropriate for the reasons explained below.

1) The elevation of the saddle of the mountain ridge on the left bank is about EL. 185 m, and this is suitable for construction of a spillway.

2) The topographical conditions may allow a heightening of the crest by about EL. 205 m though it requires a deeper abutment on the left bank and also a longer crest length. However, the above would cause an increase of the total volume content of the dam. Accordingly, the high water level of 185 m is the changeover point from the economical viewpoint.

Jonggoa Dam Site

The site is located 53 km upper from the estuary of the Jeneberang river. The dam site is located up-most of the three nominated dam sites.

The site is located below the confluence of the Malino river, a tributary flowing into the Jeneberang river from the right side. And just like the Pasaratowaya site, a protruding low ridge is observed. The width of the river at the site is about 550 m (refer to Fig. 1 - 7).

Taking topographical conditions of the site into consideration, the high water level of EL. 350 m and the dam height of 100 m are employed in the study.

1.2 Study of Dam Type

For all the sites in the foregoing, a rockfill dam with a central core type is suitable for the following reasons.

1) According to the findings of the recent geological survey of the nominated dam sites and the surrounding areas, a sufficient volume of the required materials such as soil and rock can be obtained in the vicinity of the dam site.

2) Rock-bed of the nominated dam consists mostly of sedimentary rocks of the Neogene period, such as

sandstone, siltstone, tuff and tuff-breccia, and they all have a sufficient strength for the foundation rock of a fill type dam, while insufficient for that of gravity dam.

3) Taking "Shape of Factors" (ratio of crest length to height) of the nominated dam sites into consideration, a fill type shows an advantage over a concrete type.

1.3 Proposed Dam Site

A comparison of principal features relative to the proposed dam and reservoirs at the respective sites is shown in Table 1-2.

The Bili-Bili dam site is regarded as the most advantageous of the three from the technical, economical and geographical viewpoints, while compensation for houses to be evacuated and land to be submerged in the Bili-Bili dam site is estimated higher than any other dam sites and this will create social problems.

The result of comparative study for the three dams is represented in below.

1) The total effective storage capacity is larger.

2) The total effective storage capacity per dam cost is larger.

3) The location is topographically advantageous for flood control.

4) The storage efficiency of dam (annual run-off volume/effective storage capacity) is rather high.

5) The location is nearer to the area of power consumption.

2. BILI-BILI DAM

2.1 Topographic and Geological Features

The topographical and geological featrues of the Bili-Bili dam site are mentioned in the foregoing. Judging from the findings of the recent field survey and observations of the outcrops on the both banks, a sufficient strength can be expected at the bed-rock of the dam site. With regard to the depth of the river-bed gravel layer and permeability of the bed-rock, sufficient data are not readily available so far. So a further survey may have to be performed. However, it may safely be anticipated that the depth of the river-bed gravel layer is about 10m. For the water-tight facing, grouting is just sufficient.

2.2 Reservoir

Major design factors of the Bili-Bili dam include:

High water level	:	EL. 100 m
Reservoir area	:	16.5 km^2
Gross storage capacity	:	320 x $10^6 m^3$
Effective storage capacity	:	238 x $10^6 m^3$
Flood control capacity	:	24 x $10^6 m^3$
Sediment storage capacity	:	58 x $10^6 m^3$

A capacity of 24 x $10^6 m^3$ will be allocated for flood control which regulates 1,200 m^3/sec out of 2,750 m^3/sec of the standard project flood discharge at the dam site. The remaining capacity can be used for water utilization purposes (refer to Fig. 1-2).

2.3 Dam

Type and Cross-section of Dam

As mentiond in the foregoing, the proposed dam is a rockfill dam of central core type.

With regard to the cross-section of the dam body, the up-stream slope gradient is 1:2.6 and the down-stream slope is 1:2.2.

The total volume content of a dam is most seriously affected by slope gradient. This slope gradient determined by the quality of embankment materials and foundation rocks rather than the height of crest.

Since survey of matters mentioned above has not yet been fully performed, the slope gradient and zoning in the standard specification of central core rockfill dam will be applied to the proposed design. (Refer to Fig. 1-5.)

Spillway

The design flood discharge of the dam is fixed at 4,200 m^3/sec. This figure is fixed at by increasing 20% the discharge of 3,500 m^3/sec, which corresponds to a flood of 200-year return period flood at the dam site.

Construction of the spillway on the top of the hill located between the main dam and the right wing dam is economically most advantageous in view of effective utilization of the topographical features of the site.

The spillway is an overflow weir of chuteway type with control gates. Design flood discharge of 4,200 m^3/sec will be discharged from the spillway gate in full open. The overflow width of the spillway is computed by the formula given below.

$$Q = C \times B \times H^{3/2}$$

where,

- Q: overflow discharge (m^3/sec)
- C: coefficient of discharge (=2.0)
- B: overflow width (m)
- H: overflow depth (m)

The design flood stage is established at EL. 102 m; i.e., 2.00 m above the surcharge level.

Assuming that the overflow depth is 10.00 m,

$$B = \frac{Q.}{C \times H^{3/2}} = \frac{4,200}{2.0 \times 10.00^{3/2}} = 66.40 \text{ m}$$

therefore,

7 gate x 9.50 m/gate = 66.50 m

The total overflow width of the spillway is 84.50 m including the width of the piers.

Besides, the main spillway, an emergency spillway is constructed between the main dam and the left wing dam to protect the dam against any miss-operation of the gate and/or an extraordinary flood.

Temporary Diversion Channel

Construction of a temporary diversion channel requires special consideration to safely divert a flood discharge during construction of the dam.

The design discharge of the diversion channel at the Bili-Bili dam site is 2,200 m^3/sec, corresponding to a 20-year return period flood, which is usually applied for the fill dam.

Designing the diversion channel in a tunnel type with a design discharge of 2,200 m^3/s, the size of the diversion channel is roughly calculated below.

1) Deversion channel

Diameter x No.	:	10.00 m x 2
Length	:	400 m each

2) Cofferdam

Height	:	22.00 m
Crest Length	:	420.00 m
Total Volume Content	:	400,000 m^3

3. WATER RESOURCES DEVELOPMENT

Judging from the expected future water demand in the project area, the storage capacity thus developed can be utilized for irrigation, hydropower and municipal water supply.

Irrigation water in the lower reaches is seriously in demand, and furthermore demand for municipal water supply and for electric power will between the normal water level and the low water level will be 238 x $10^6 m^3$, which may be allotted for irrigation, hydro power generation and municipal water.

3.1 Irrigation and Municipal Water

It is possible to utilize a discharge of 21 m^3/s at Kampili throughout the year. This is estimated from the annual rainfall of the second droughtiest year in the last ten annual records. If this discharge will be used for irrigation, 60% of 31,000 ha agricultural land, about 19,000 ha/1 will be irrigable.

3.2 Power Generation

In the design of the Bili-Bili reservoir, the volume for power generation is not specifically secured. But the discharge for water supply can also be utilized for power generation. Therefore, a hydro-power station will be constructed at the direct down-stream of the dam (refer to Fig. 3-1.)

Power generation facilities of the Bili-Bili dam are planned in the premise that power is generated for 8 hours to cover daily peak period.

The maximum output of the power station mentioned above is 22,000 KW under the maximum discharge at 60 m^3/sec.

The annual energy output is calculated to be 75,000 MWH by using the prospected flow regime after the proposed dam is operated. The annual energy output is shown in Table 3-1.

The size of the power station is to be determined by the conditions as mentioned below.

<u>Power Station</u>

1) Bili-Bili reservoir

Intake level:	Max. normal high water level	: EL. 98.00 m
	Min. low water level	: EL. 74.00 m

/1 : Assuming that water requirement in depth is 8 mm/day and farm waste and conveyance losses are 10% each of the water requirement

2) Afterbay level

Water level of Jeneberang river at immediate down-stream of the dam . EL. 55.00 m

3) Head

Max. gross head	43.00 m
Max. effective head	40.50 m
Min. effective head	16.50 m

4) Discharge

Max. discharge $60\ m^3/sec.$

5) Generator

Vertical Shaft Kaplan Turbin, 11,000 KW, 2 units

The maximum output of the power station is computed by the equation below.

$$P = 9.8 \times \eta \times Q \times H$$

where,

P :	max. output (KW)
η :	efficiency (0.85)
Q :	discharge (30 m^3/sec.)
H :	head (40.50 m)

therefore,

$$P = 9.8 \times 0.85 \times 30 \times 40.50 = 10{,}100\ KW$$

Table 3-1 ANNUAL ENERGY OUTPUT

Year	Energy Output ($\times 10^3$ MWH)
1953	73.1
1956	78.3
1957	75.1
1958	78.1
1959	82.2
1960	71.0
1975	78.2
1976	66.4
1977	76.6
1978	75.0
Average	75.4 $\times 10^3$ MWH

Regulating Pondage

For a temporary storage of tailwater from the power station, a regulation pondage will be installed to meet demands for irrigation and municipal water in the lower reaches.

Regulating Pondage

For a temporary storage of tailwater from the power station, a regulation pondage will be installed to meet demands for irrigation and municipal water in the lower reaches.

The proposed site of the regulating pondage is planned about 4.0 km down-stream of the Bili-Bili main dam.

The required volume of the regulating pondage is computed by the formula below.

$$V = 3600 \times (qd - q) \times T$$

where,

V :	required storage volume of regulating pondage (m^3)
qd:	max. available discharge (60 m^3/sec)
q :	available irrigtion water (21 m^3/sec)
T :	peak load duration (8 hours)

therefore,

$$V = 3600 \times (60 - 21) \times 8 = 1,120,000\ m^3$$

The required storage volume of the regulating pondage is 1,200,000 m^3.

To secure the storage volume mentioned above, a fixed weir with 100 m in length and 5.0 m in height is appropriate for the purpose.

And also, two dewatering gates, 5.0 m (width) x 5.0 m (height), are to be constructed for water supply.

Flood discharge and spill water are allowed to overflow the crest of the weir.

4. LAND ACQUISITION AND HOUSE EVACUATION

The following shows number of houses to be evacuated and area of land to be acquired due to construction of the dam.

Land acquisition	1,400 ha.
House evacuation	550 houses

In addition, since a part of Jl. Malino will be submerged over the stretch of 15.0 km, a new road will have to be constructed.

Table 1-1 THE REPRESENTATIVE DAM IN INDONESIA

No.	Name of Dam	Catchment Area (km^2)	Type of Dam	Height of Dam (m)	Length of Dam (m)	Dam Volume ($\times 10^4$ m^3)	T. S. Capacity ($\times 10^6$ m^3)	A. S. Capacity ($\times 10^6$ m^3)	F. C. Capacity ($\times 10^6$ m^3)	S. Capacity ($\times 10^6$ m^3)	A.S.C./ D.V.	Design Flood (m^3/s)	Specific discharge ($m^3/sec/km^2$)
1	Wonogiri Dam	1,350	C. C.	38.0	1,440	180	730	440	220	120	244.4	6,250	4.63
2	Selorejo Dam	236	W	46.0	447	200	62.3	50.1			25.1	920	3.90
3	Karangkates Dam	2,050	C. C.	100.0	800	615	343	253			41.1	4,200	2.05
4	Wlingi Dam	2,890	C. C.	28.5	717	63	24	5.2			8.3	3,400	1.18
5	Lahor Dam	160	C. C.	72.0	620	131	34	29			22.1	690	4.31
6	Riamkanan Dam	1,043	H	66.0	195	67	1,200	-			-	1,950	[illegible]
7	Senpor Dam	43	C. C.	50.5	360	65	56	50.5			77.7	1,400	32.6
8	Jatiluhur Dam	4,500	I. C.	100.0	1,200	-	3,000	-			-	8,000	1.78
	Bili-Bili Dam	384.4	C. C.	65.0	1,670	800	320	262	24	58	32.8	4,200	10.9

Reference

* Type of Dam

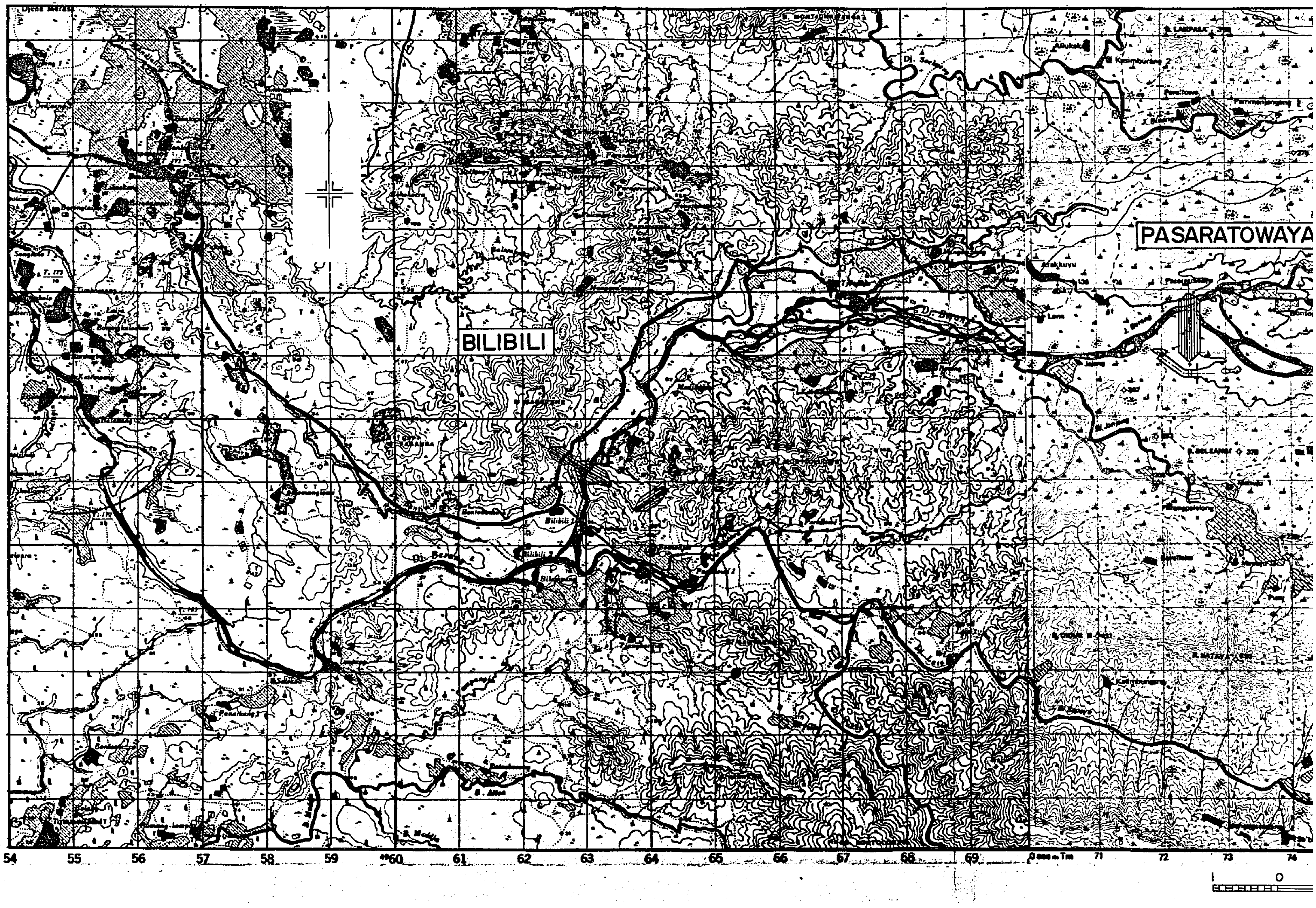
C. C. } Zone type dam - Center Core
I. C. } Zone type dam - Inclining Core
W } Zone type dam - Wide Core

H - Homogeneous type

* T. S. Capacity : Total storage capacity
A. S. Capacity : Available storage capacity
F. C. Capacity : Flood control capacity
S. Capacity : Sediment capacity

Table 1-2 COMPARISON OF PRINCIPAL FEATURES OF DAMS AND RESERVOIRS

	Items	Bili-Bili	Pasaratowaya	Jonggoa
a.	Catchment area (km^2)	384.4	319.2	242.0
b.	Surface Area (km^2)	16.5	7.8	8.2
c.	Dam height (m)	65.0	80.0	100.0
d.	Dam volume (m^3)	$8.0x10^6$	$12.5x10^6$	$19.1x10^6$
e.	Reservoir capacity (m^3)	$320x10^6$	$240x10^6$	$280x10^6$
f.	Total effective storage capacity (m^3)	$262x10^6$	$192x10^6$	$243x10^6$
g.	Construction cost ($)	$126x10^6$	$154x10^6$	$205x10^6$
h.	Total effective storage capacity per dam cost (f/g)	2.08	1.25	1.19
i.	Cost per dam volume (g/d)	15.8	12.3	10.7
j.	Submerged houses (nos.)	550	300	100
k.	Submerged agricultural land (ha)	1,400	500	350
l.	Annual runoff volume per effective storage capacity	3.1	3.4	2.1



BILIBILI
PASARATOWAYA
54
55
56
57
58
59
60
61
62
63
64
65
66
67
68
69
71
72
73
74
1
0

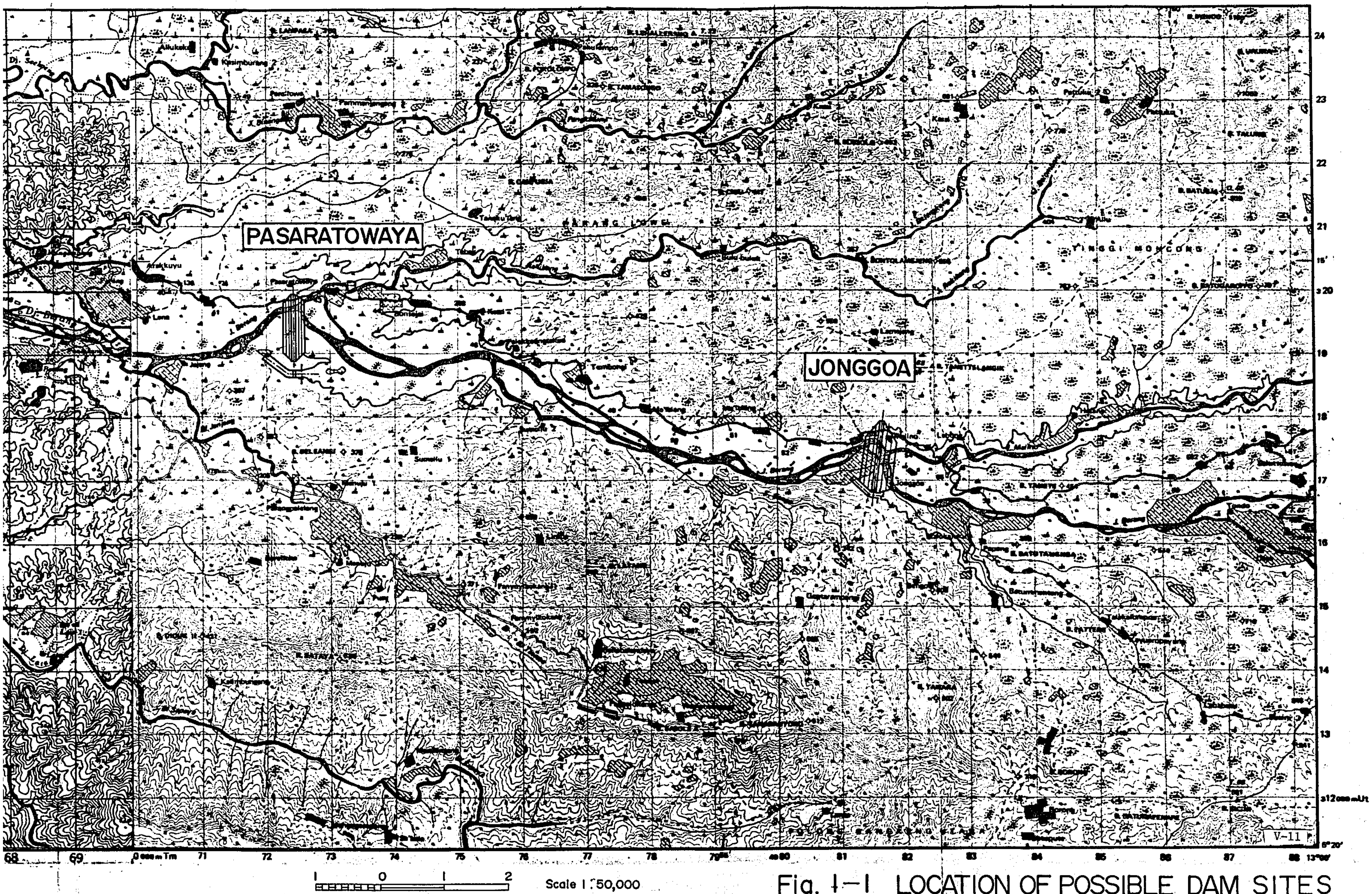


Fig. I-1 LOCATION OF POSSIBLE DAM SITES

Fig. 1-2 ELEVATION - STORAGE CAPACITY CURVE OF BILI-BILI RESERVOIR

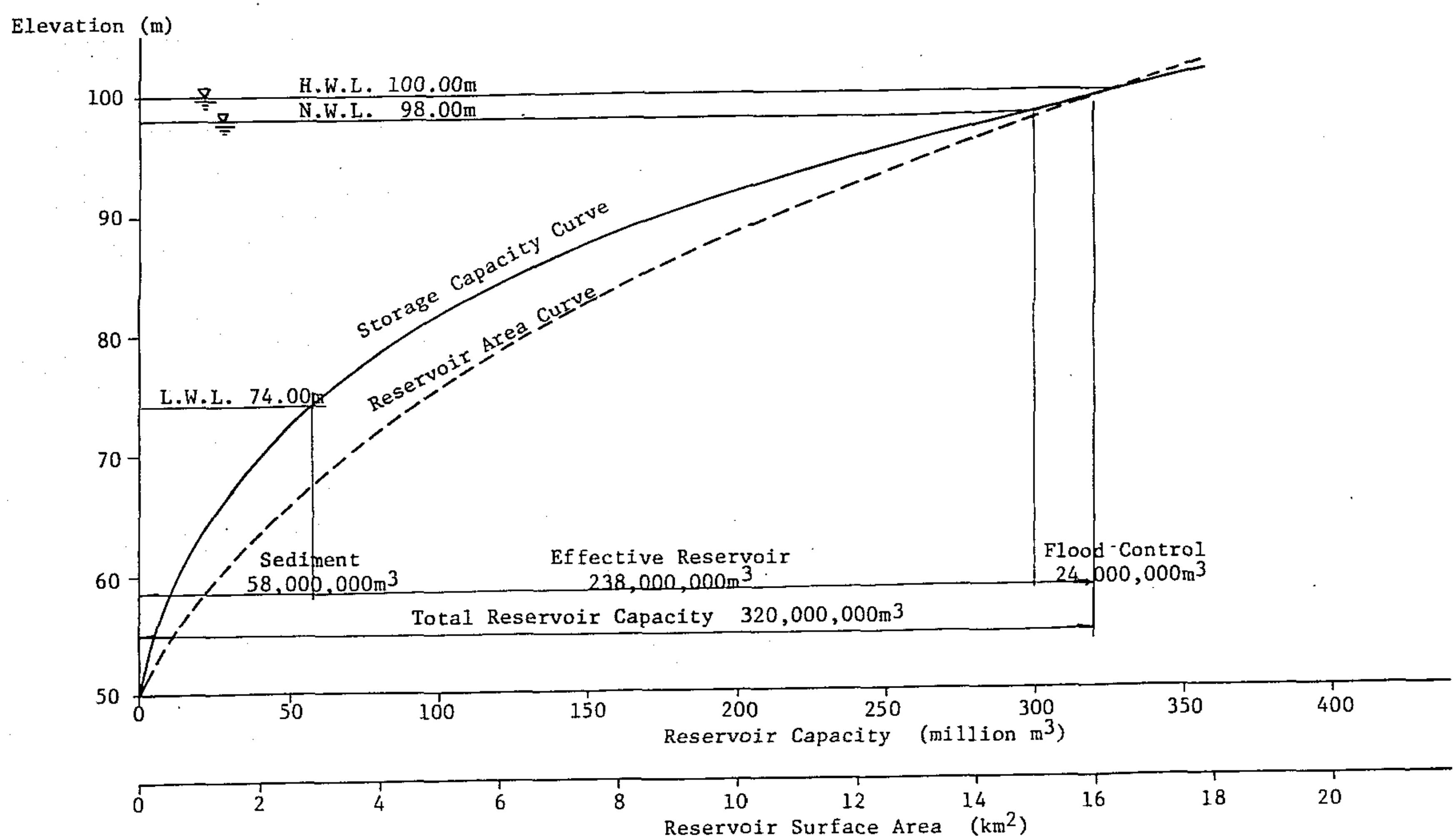


Fig. 1-3 ELEVATION -- STORAGE CAPACITY CURVE OF PASARATOWAYA RESERVOIR

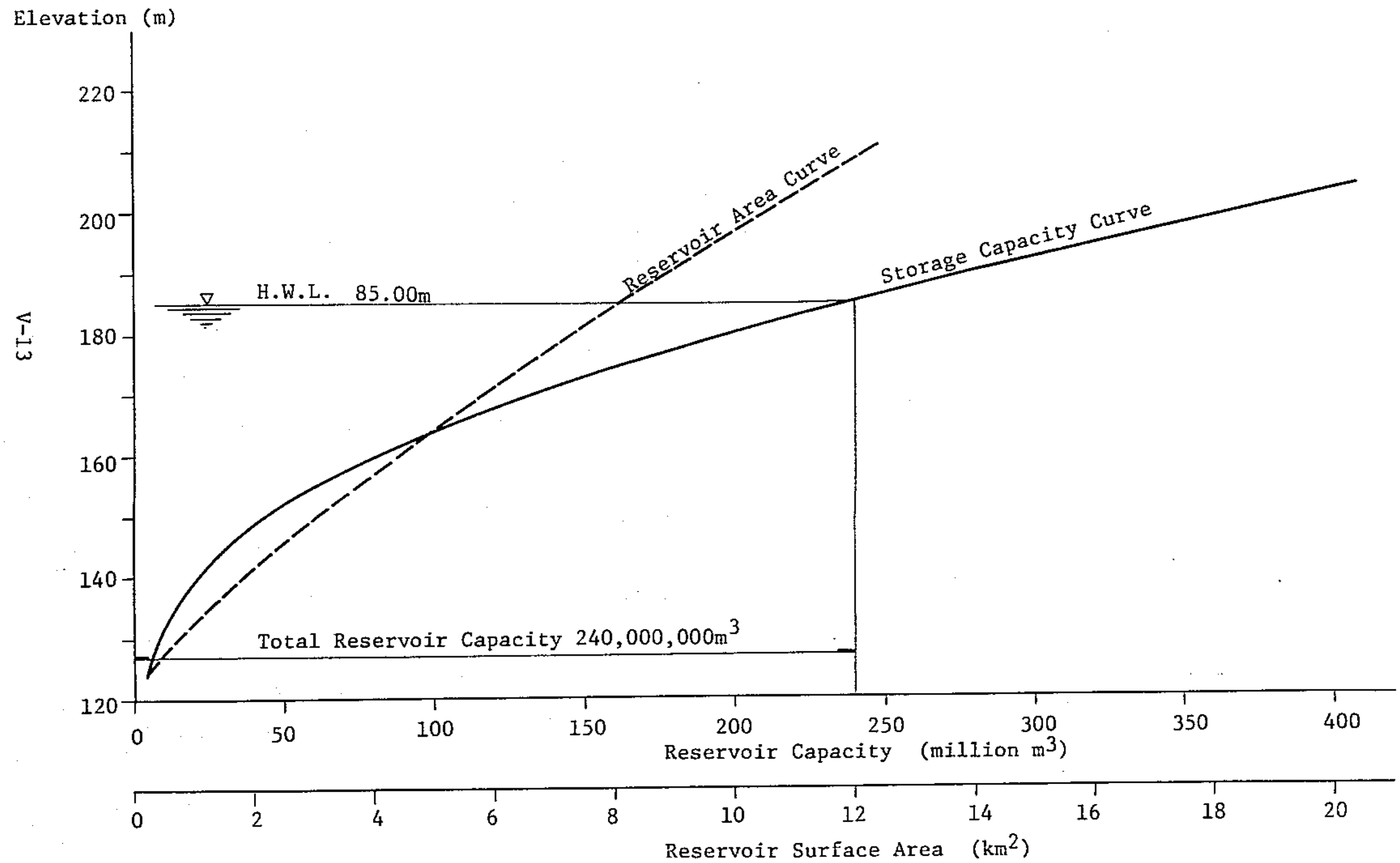
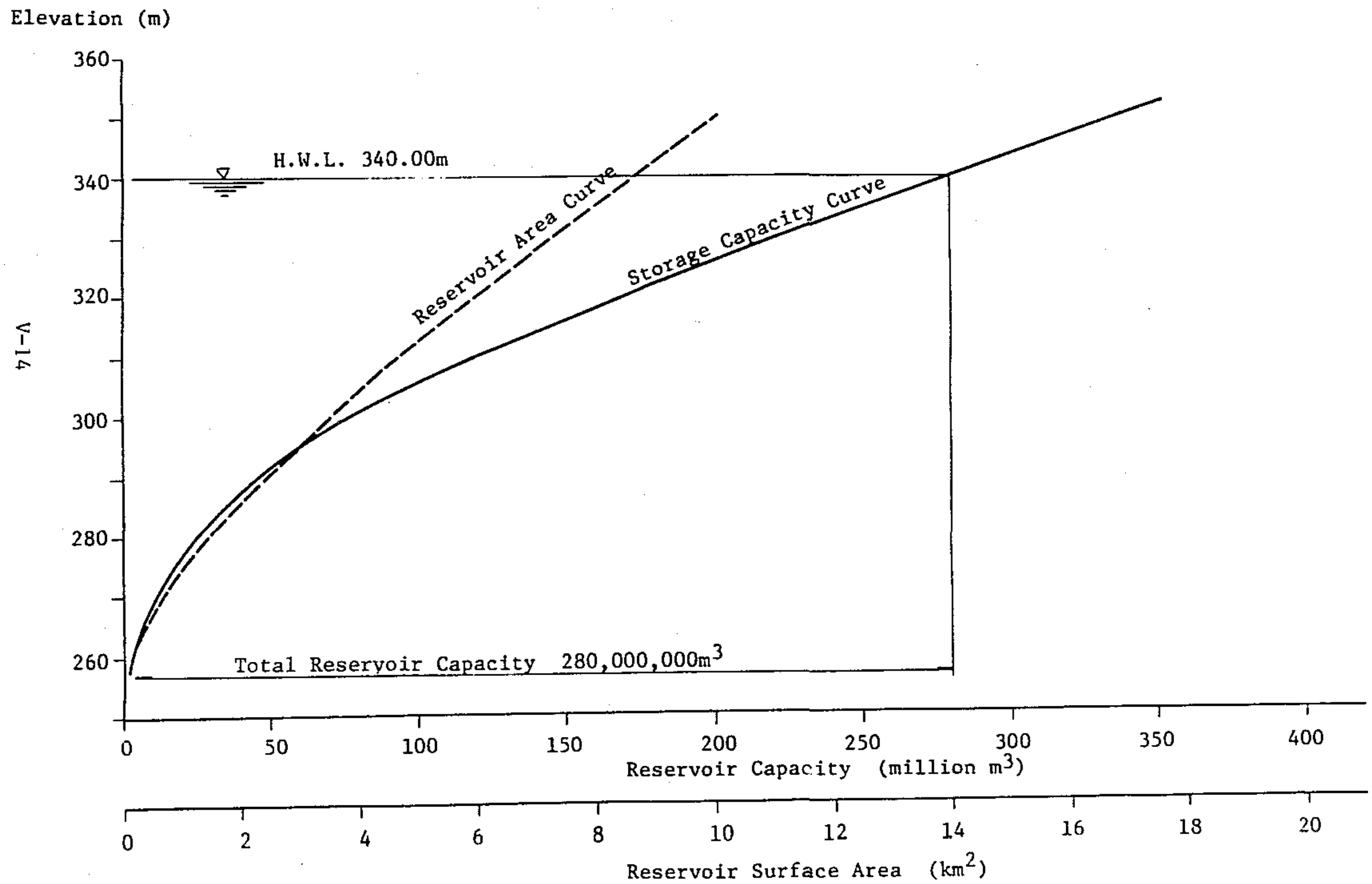
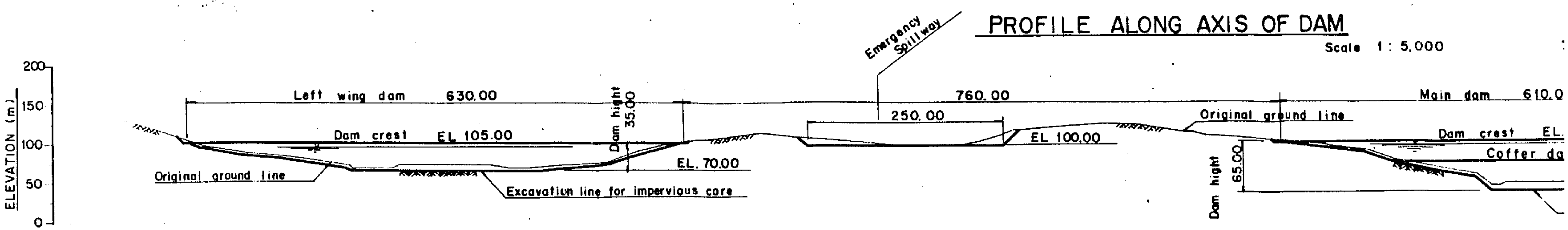


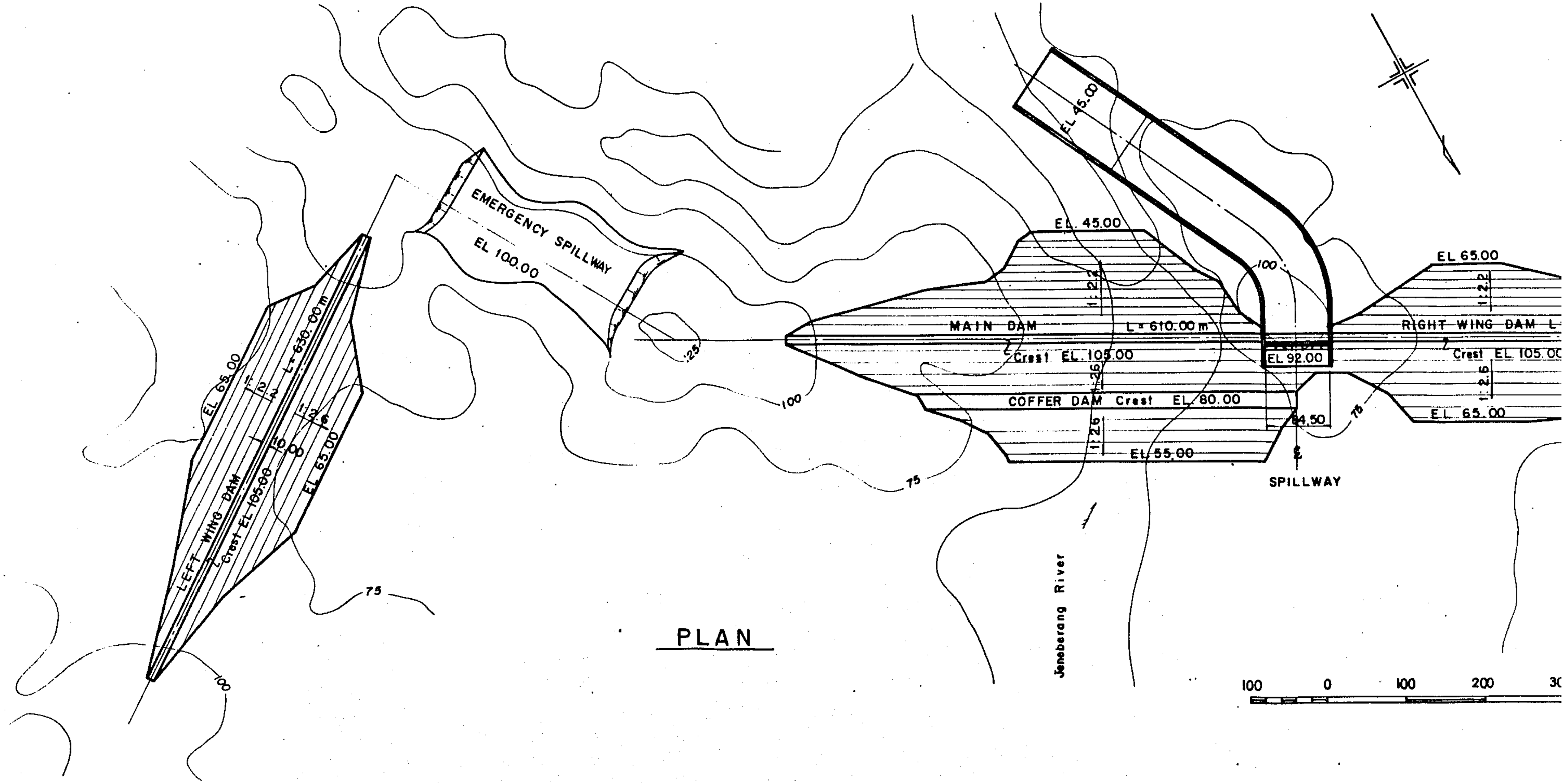
Fig. 1-4 ELEVATION - STORAGE CAPACITY CURVE OF JONGGOA RESERVOIR



PROFILE ALONG AXIS OF DAM

Scale 1 : 5,000





PLAN

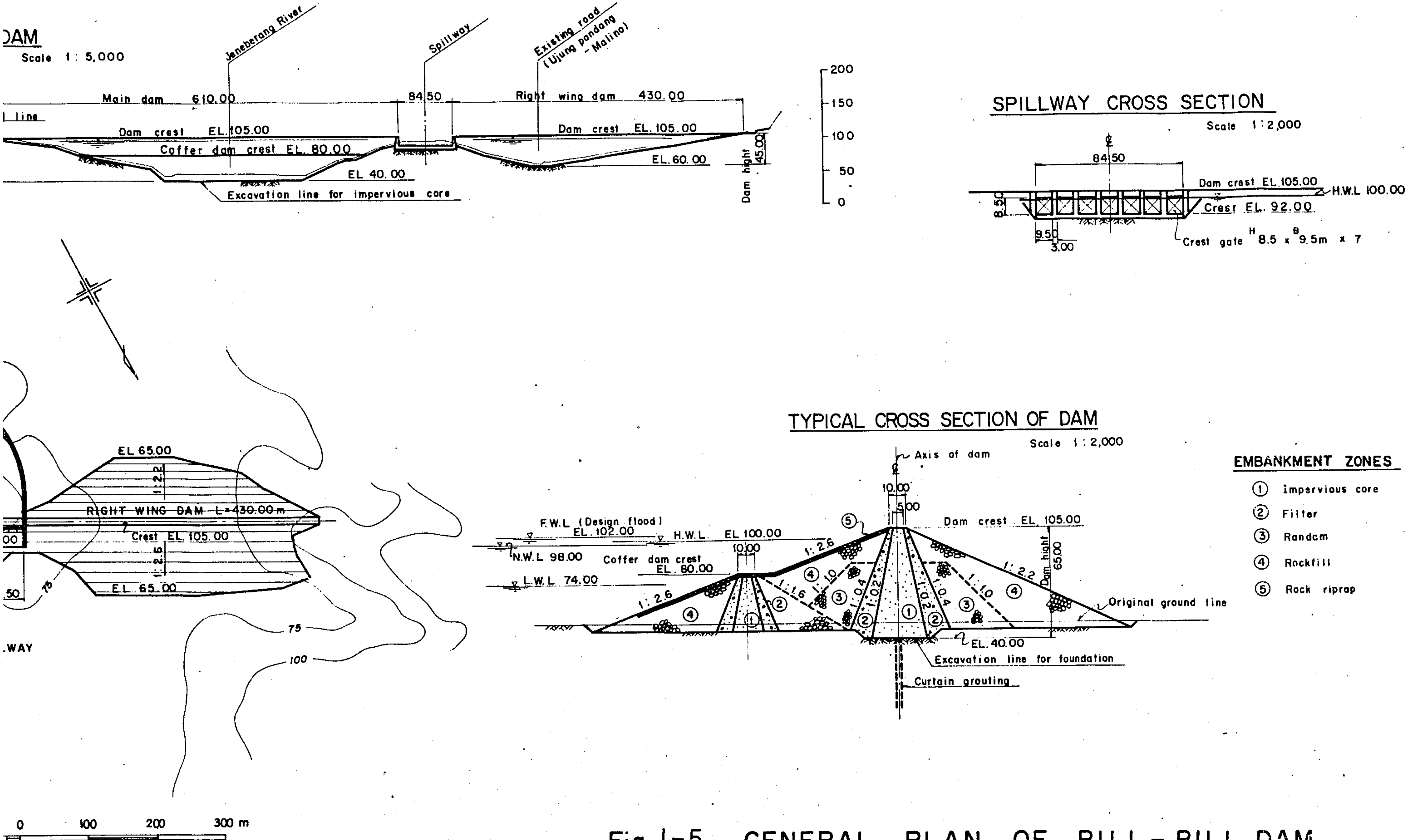


Fig. 1-5 GENERAL PLAN OF BILI - BILI DAM

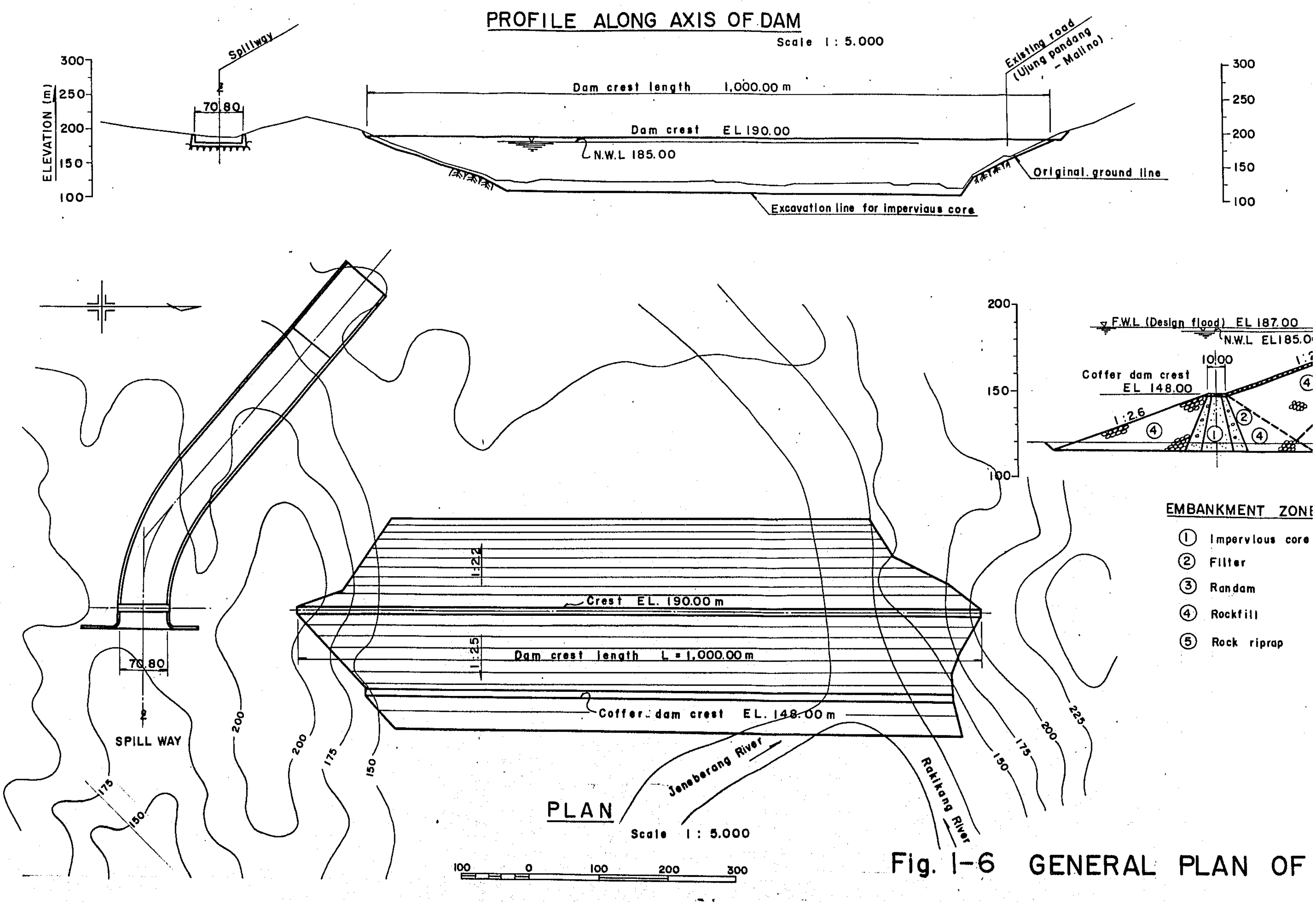


Fig. 1-6 GENERAL PLAN OF

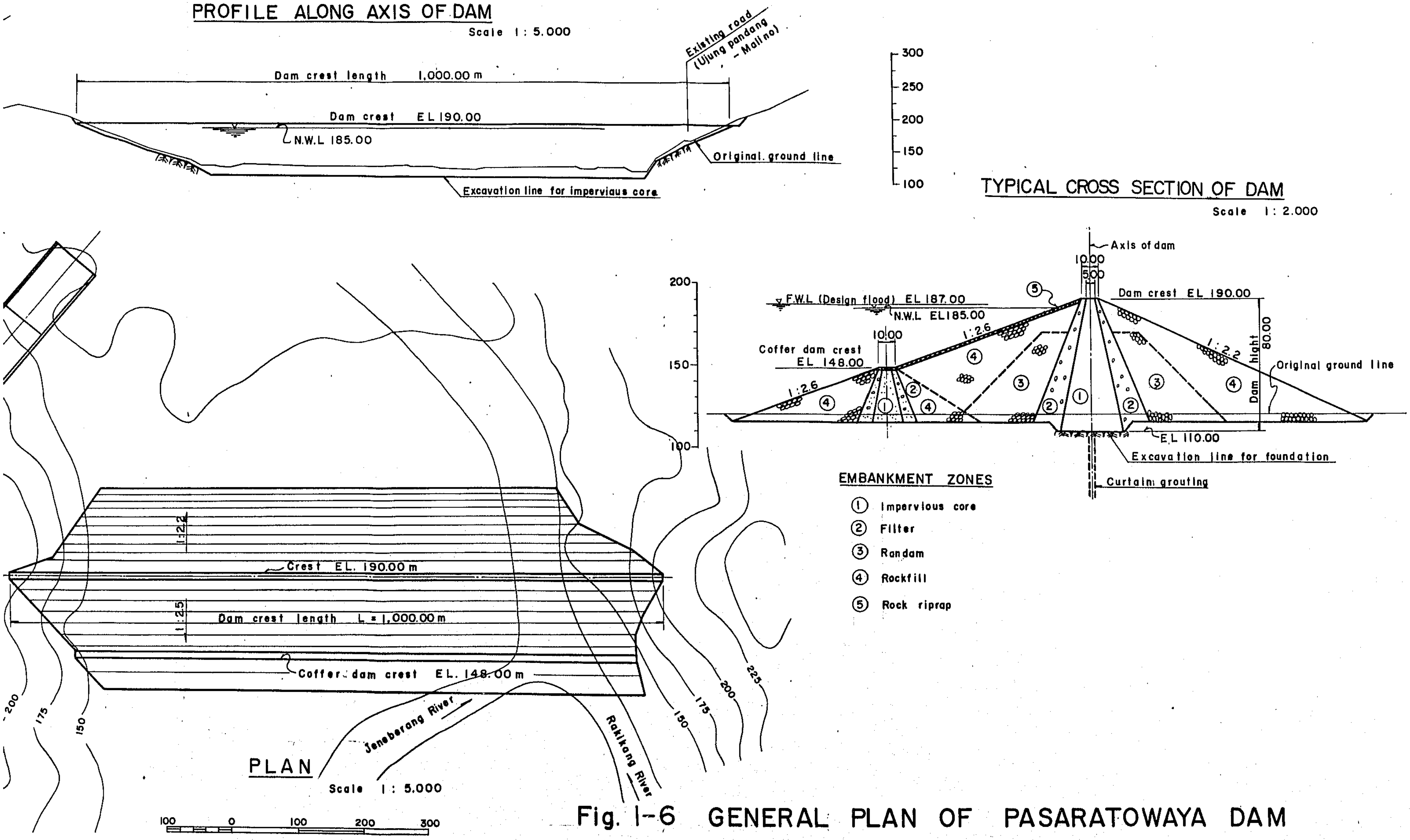
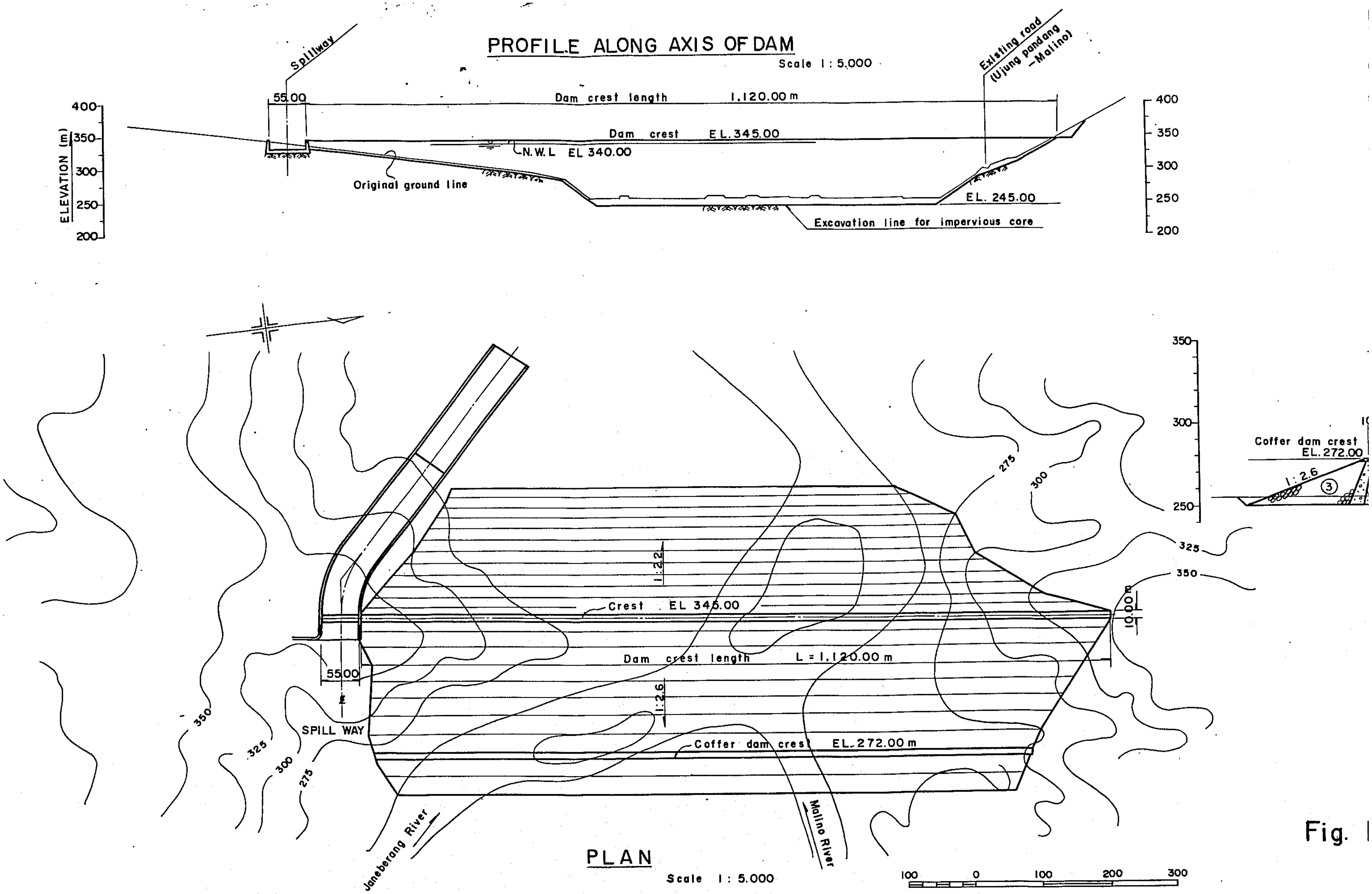


Fig. 1-6 GENERAL PLAN OF PASARATOWAYA DAM

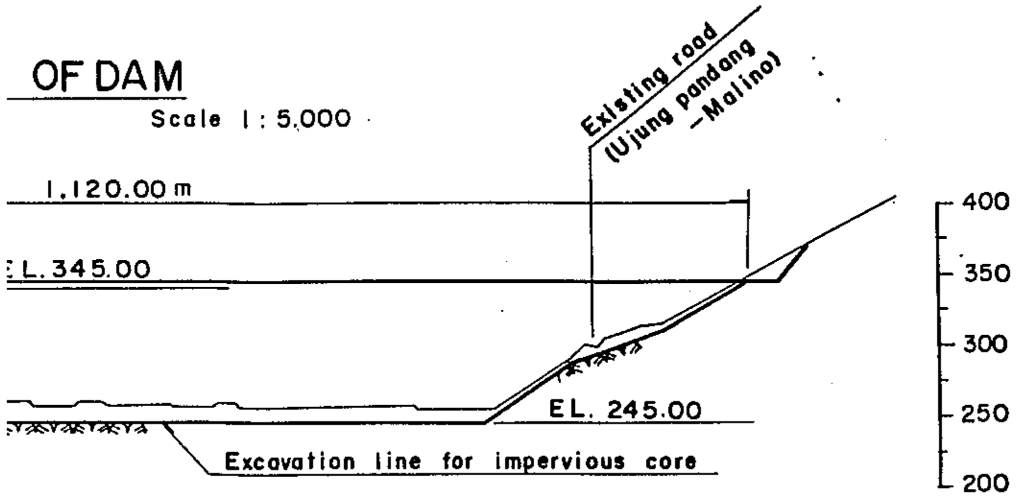


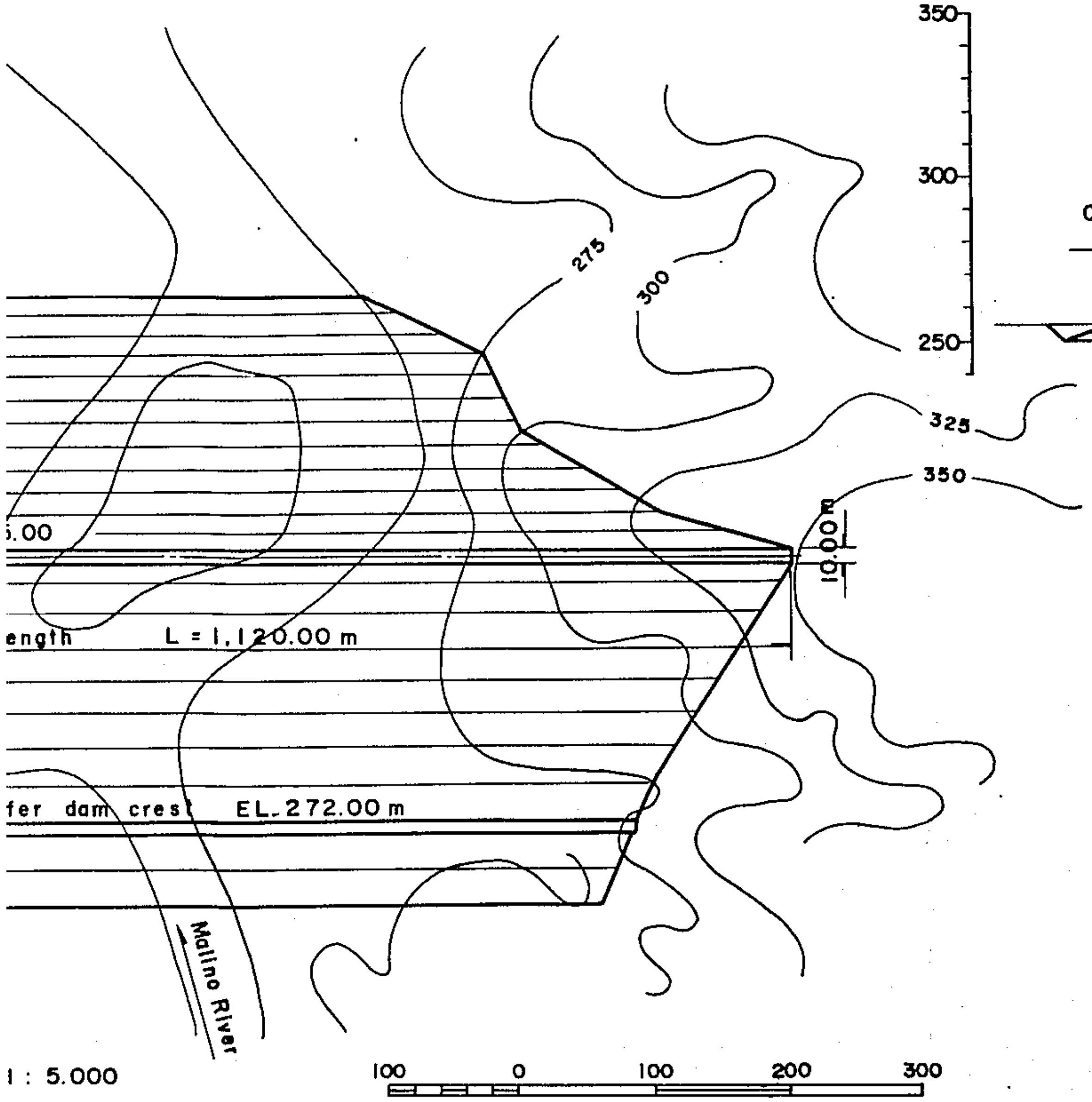
PROFILE ALONG AXIS OF DAM
Scale 1 : 5,000
Spillway
Existing road (Ujung pandang -Malino)
55.00
Dam crest length 1,120.00 m
Dam crest EL. 345.00
N.W.L EL 340.00
Original ground line
EL. 245.00
Excavation line for impervious core
ELEVATION (m)
400
350
300
250
200
Coffer dam crest EL. 272.00
1 : 2.6
③
1:2.2
Crest EL 345.00
Dam crest length L = 1,120.00 m
10.00 m
55.00
SPILL WAY
1:2.6
Coffer dam crest EL. 272.00 m
275
300
325
350
Janeberang River
Malino River
PLAN
Scale 1 : 5,000
100 0 100 200 300

Fig. 1

OF DAM

Scale 1 : 5,000





TYPICAL CROSS SECTION OF DAM

Scale 1 : 2,000

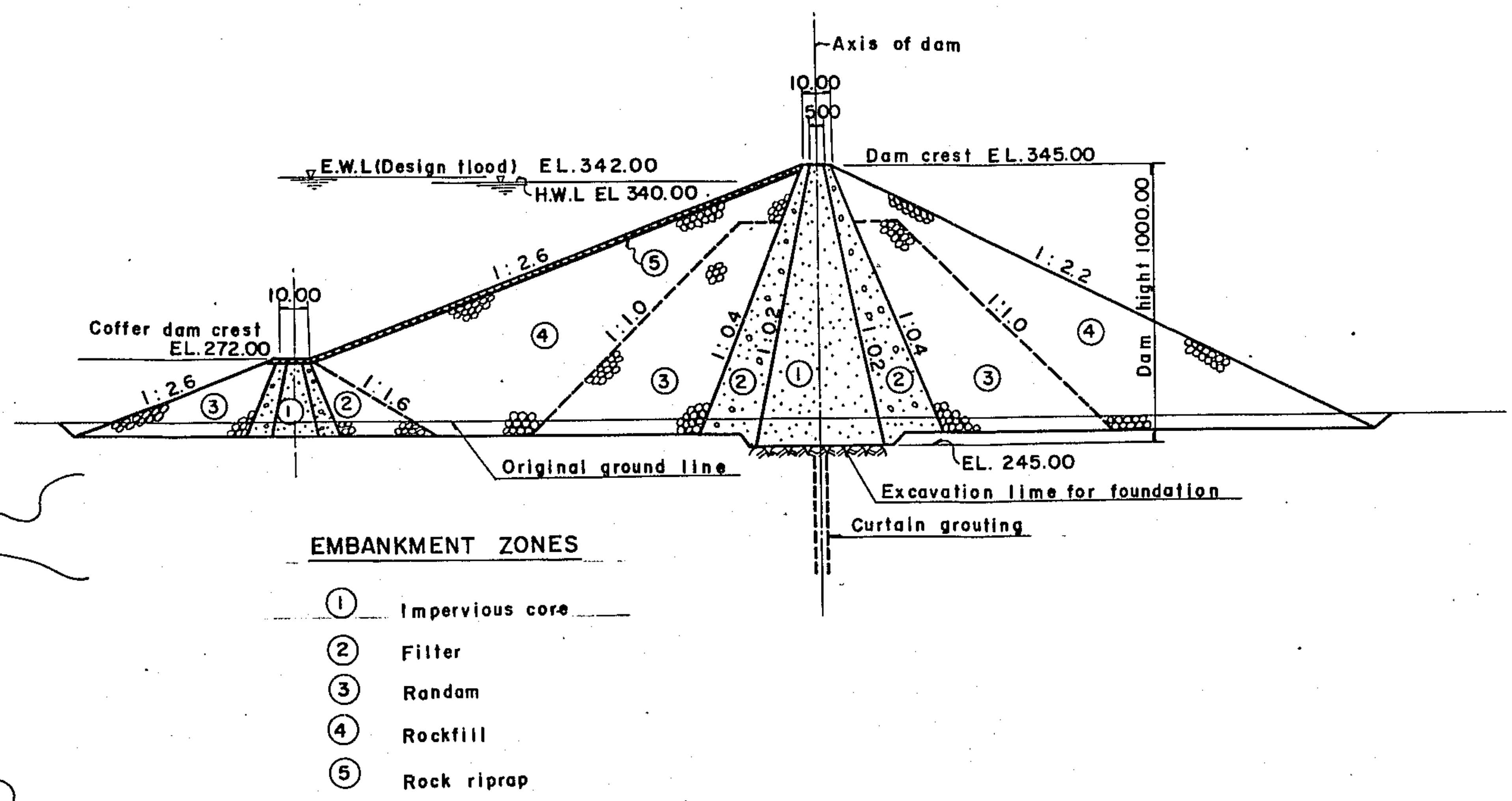
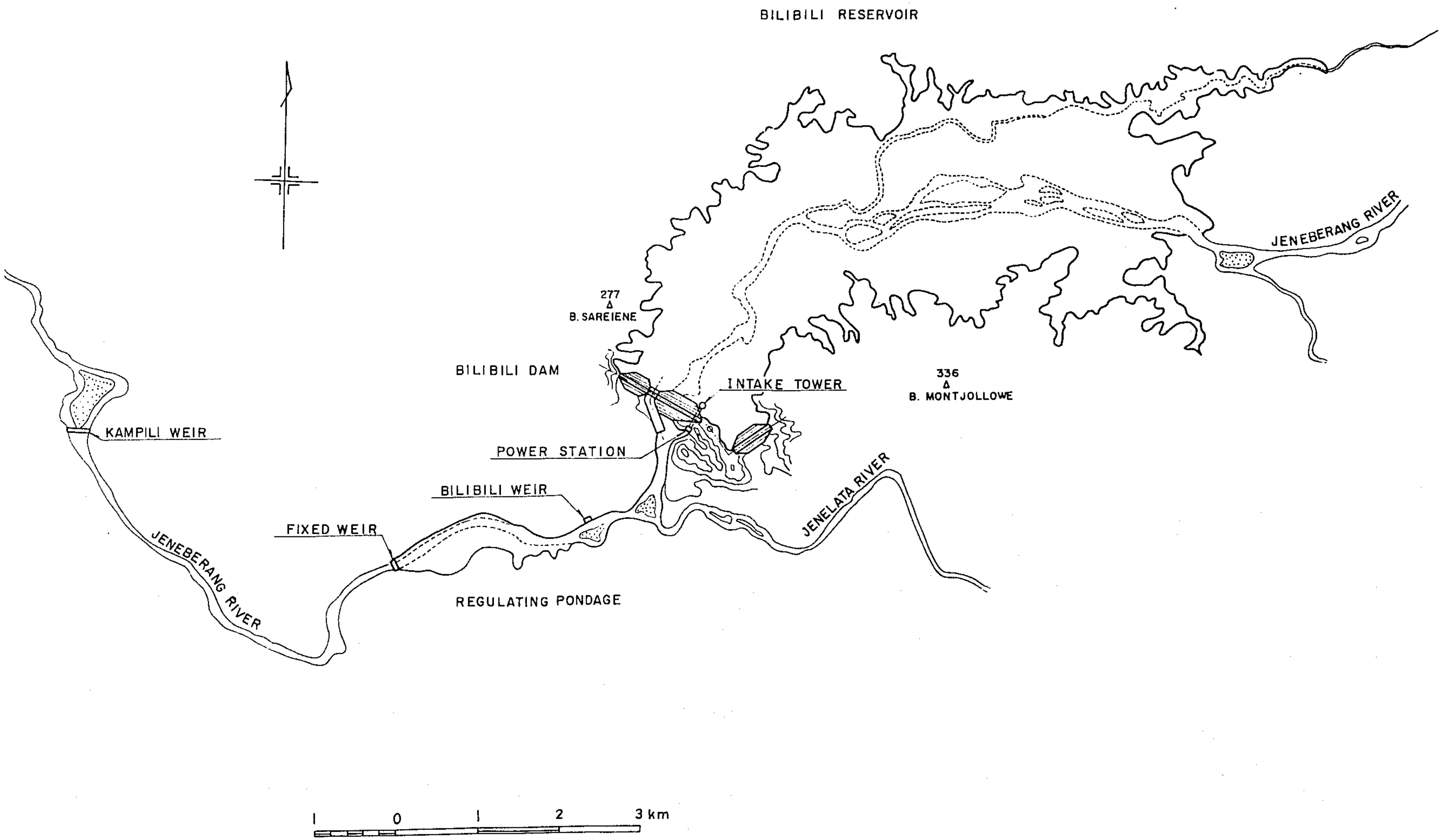


Fig. 1-7 GENERAL PLAN OF JONGGOA DAM

Fig. 3-1 LOCATION OF BILI-BILI DAM AND HYDRAULIC FACILITIES



VI DRAINAGE SYSTEM IMPROVEMENT

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1 PRESENT CONDITIONS OF DRAINAGE SYSTEM

1.1 Drainage Basin

The target area is situated in the lowland area surrounded by the Tallo river, the Jeneberang river and the old urban area of Ujung Pandang city as shown in Fig. 1-1. This area is divided into two areas from topographical and hydrological viewpoints; i.e., city-side area of 15.1 km^2 and mountain-side area of 45.4 km^2.

The lowest ground elevation in the mountain-side area is 0.3 m above M.S.L. and in the city-side area is 1.5 m above M.S.L. Therefore, the lowest ground elevations of the mountain-side area and the city-side area are lower and higher than the mean high water springs (0.56 m above M.S.L) respectively.

Most of the mountain-side area is at present paddy fields. A number of public buildings and private houses in the city-side area are so far completed or under construction since the regional development plan is now in a steady progress.

During a rainy season, the target area suffers from annual flood damage caused by poor drainage facilities, and especially the area experienced the biggest damage during the flood of January, 1976.

1.2 Drainage System

The existing drainage system consists of three drainage channels such as the Pampang river, the Panampu channel and the Sinrijala channel and two sluices along the Jeneberang river as shown in Fig. 1-1.

Inundation water in the city-side area is directly drained into the Makassar Strait through the Panampu channel; the water inundated in the mountain-side area is drained into the Tallo river, through the Pampang river. The Sinrijala channel is interconnecting two inundation areas.

In the lower reaches of the Pampang river, heavily meandering and flowing down to the Tallo river in the lowland swampy area, tidal influence is excessive, and it causes the poor drainage capacity to drain the inundation water.

1.3 Drainage Facilities

Drainage Channel

The Pampang river is a natural drainage channel. The channel width is about 1.5 m in the upper reaches, and about 25 m in the lower reaches.

The Panampu channel with a total length of 4.5 km was constructed decades ago, and is protected by cobble concrete revetment in the section between 2.5 K and 3.0 K and also in the section near the channel mouth. However, almost all of the remaining sections are excavated without protective works.

The Sinrijala channel is, in portions, embanked. However, most of the remaining section of the channel is a natural channel.

Flow capacities of the Pampang river, the Panampu channel and the Sinrijala channel are estimated at approximately 20 m^3/s, 4 m^3/s, and 4 m^3/s respectively. In all of these channels, water does not flow smoothly because of excessive mud sedimentation and dumped rubbish. The crosssections of the existing channels are shown in Fig. 1-2.

Sluice

Two sluices are located at 3.0 K on the right side dike from the river mouth of the Jeneberang river in order to prevent the reverse flow from the Jeneberang during a flood. However, drainage condition is not good because of the lack of drainage channel to guide inundation water and the lack of maintenance of sluices.

1.4 Flood Damage

The biggest inundation in the past was experienced in January of 1976 and inflicted a great deal of damage to the project area.

According to the rainfall data in January of 1976, rainfall lasted for 23 days from January 5 to 27, and the maximum daily rainfall was recorded at 300 mm on January 12. The total contineous rainfall amounted to 984 mm. The most severely damaged areas during the flood were a part of the old urban area along the Panampu channel and the area around the Rawa Landak, in which the maximum inundation water stage reaches 2.1 m above M.S.L. The average inundation water depth in the subject area was estimated at 0.6 m and duration of inundation was estimated for 3 days according to the reconnaissance survey. (Refer to Table 1-1) A number of houses in the area were submerged above the floor level.

The inundation area in 1976 flood is estimated at 35 km^2 as shown in Fig. 1-3, based on the result of interview held at 149 points in the project area. The inundation damage is estimated at 450 million Rupiahs approximately.

2. FORMULATION OF OVERALL DRAINAGE PLAN

The drainage plan of the city-side area including the first stage development area is taken up as the urgent plan, and the drainage plan of the mountain-side area including the second and the third stage development area is taken up as the overall plan, and the study to determine the optimum drainge system of overall plan is conducted in this chapter.

2.1 General Conception

Conception

The general conception to establish the drainage plan of the target area is as follows.

1) Pumping drainage has a possibility of raising mechanical troubles during a flood and its maintenance and operation cost is high. Therefore, it is generally recommended that natural drainage should be studied first in framing a drainage plan.

2) It is usually economically viable to set up a drainage system in any area that has a lot of properties and assests. But in so doing any harmful side effects upon other areas must be avoided.

3) The proposed drainage facilities should be easily operated and maintained.

4) The drainage system scale can be determined economically optimum, because the damage caused by serious water flow on the ground is not anticipated.

General Study

A drainage scheme usually includes natural drainage, mechanical drainge and use of a regulation pond.

However, in this case, it is topographically impossible to provide a space for regulation pond. And even if a regulation pond could be provided, its effectiveness would be doubtful on the grounds that the area is often subject to five days of continuous rainfall. Under such condition, any regulation pond would be partially filled within a first few days. In other words, the capacity of a regulation pond would be reduced well before the peak run-off occurs.

Natural drainage is also topographically unavailable in the overall plan, because the minimum elevation of the inundation area is lower than the outlet water stage of the channel. Therefore, only a mechanical drainage system can be employed.

The drainage system is designed for the area under urbanization. As the regional development progress, the present land elevation in the area is anticipated to vary remarkably. In this study, therefore, the optimum pumping capacity has been determined on the assumption that a pumping station is installed only at the Pampang bridge.

Fig. 2-1 shows the possible drainage system plan.

2.2 Study Procedure

Study Case

The optimum scale of drainage facilities should be determined by economical evaluation based on the estimate of cost and benefit which may by brought about after completion of the proposed facilities. In this study, the assets in the object area after completion of the second and the third development stages are assumed to be the assets existing in the project area. The existing facilities and the facilities to be constructed in the urgent plan are adopted as the base to evaluate the overall drainage plan.

Study cases in this plan are as follows.

Case I

Facilities : Existing
Assets : After completion of the second stage development

Case II

Facilities : Existing
Assets : After completion of the third stage development

Case III

Facilities : After completion of the urgent plan
Assets : After completion of second stage development

Case IV

Facilities : After completion of the urgent plan
Assets : After completion of the third stage development

The above cases are summarized as follows.

Drainage Facilities	Assets in the Area			
	Existing	1st Stage	2nd Stage	3rd Stage
Existing	-	-	Case I	Case II
After completion of the urgent plan	-	-	Case III	Case IV

Evaluation Method

To determine the optimum scale of drainage facilities from the economic viewpoint, the index α = B/C (B is the annual flood damage reduction and C is the construction cost) is applied to the calculation. Estimation procedure and results of B and C are described in detail in section 2.3 and the scale of drainage facilities, where α is at maximum, is adopted as the optimum scale. Evaluation was conducted in accordance with the procedure shown in Fig. 2-2.

2.3 Premises of Study

The study on the optimum drainage system is based on the following premises.

1) Drainage condition

The inundation area is divided into two parts (the city-side and the mountain-side areas) by Jl. Panakkukang. The calculation model has two inundation areas.

The Sinrijala channel linking these two areas has the function of transporting the inundation water. If the inundation water depth in the inundation area exceeds 2.2 m, the inundation water overflows onto the Panakkukang road.

The drainage system should be designed to drain the inundation water, not the overtopped water from the Jeneberang. The drainage calculation has been based on the assumption that there is no overtopped water from the Jeneberang.

The inundation water would be drained by the Pampang river, and by the Sinrijala, Panampu and Jongaya channels. Fig. 2-3 illustrates the calculation model.

2) Run-off

The suggested development work in the inner basin is expected to change the run-off coefficient in this area.

Most floods are caused by 5 days of continuous rainfall. The run-off after the 2nd and 3rd development stages has been estimated by using the rainfall intensity pattern as shown in Fig. 2-4, considering the change of run-off coefficients. Refer to Table 4-5 in "Hydrology" in the foregoing on this report.

3) Outlet water stage

The inundation water is drained into the Makkassar Strait directly through Panampu and Jongaya channel, and into the Tallo river through Pampang river. The outlet water stage of each drainage channel is shown in Fig. 2-5. The outlet water stages of the Tallo and Pampang rivers cannot be estimated exactly because of insufficient hydraulic data of the Tallo river. Consequently the model tidal hydrograph had to be used to estimate their outlet water stages, taking into account the topographical gradient.

4) Benefit estimation

The optimum pumping capacities for the 2nd and 3rd development stages were determined in two ways; 1) on a basis of the present facilities and 2) on the assumption that the facilities proposed in the urgent plan are considered to be the existing. The optimum scale of the drainage facilities is evaluated from flood damage inflicted on buildings and on household effects. The effect of flood damage caused to agricultural land and to crops has been disregarded in this evaluation since it is considered to be almost negligible.

The project benefit is calculated as the difference between the damages done with and without the new drainage system.

5) Cost estimation

The cost required for the installation of pumps and for the modification of the existing channels as well as the construction of the proposed channels is estimated under the following conditions.

i) Pumping station

The direct construction cost of the proposed pumping station includes pump cost, housing cost and civil engineering cost.

The indirect cost is fixed at 30% of the direct cost.

Land acquisition and house evacuation for pumping station are also included.

ii) Drainage channels

The drainage channels are designed to drain the discharge corresponding to the drainage capacity of pumps, and a trapezoid section with side slope gradient at 1:1 is adopted as the cross-section. Manning's roughness coefficient (n) is fixed at n=0.025 for the proposed channels.

The direct cost includes cost of excavation, backfill, spoil of soil, revetment, plain concrete, base concrete and logpiles.

The indirect cost is fixed at 30% of the direct cost.

Land acquisition and house evacuation are also included.

2.4 Determination of System Scale

Calculation Results

1) Inundation water stage

The inundation water stage was calculated for all combination of the below-mentioned factors (refer to Table 2-1).

i) Assets in the inner basin :

After completion of the second stage development

After completion of the third stage development

ii) Return periods of flood : 2-, 5-, 10-, 30- and 50-year.

iii) Drainage facilities :

Existing

After completion of the urgent plan

Drainage system with pumping capacity of 10, 20, 30, 50, 70, 100 and 160 m^3/s.

2) Benefit estimation

The estimated benefits are given in Table 2-2 and their calculation examples are presented in Table 2-3.

3) Cost estimation

The cost required for construction of the assumed facilities are summarized in Table 2-2. The breakdown of drainge channel construction cost and pump installation cost are set forth in Tables 2-4 and 2-5 respectively. And the relation curve between the cost and capacity are also presented in Fig. 2-6 for the drainage channel and Fig. 2-7 for the pump installation.

Evaluation of System Scale

The evaluation of each study case is presented Table 2-3 and Fig. 2-8. The evaluation results of the drainage system is summarized in Table 2-6.

Table 2-6 EVALUATION RESULT OF DRAINAGE SYSTEM

	Drainage Capacity (m^3/s)	Benefit ($x10^9Rp$)	Cost ($x10^9Rp$)	Benefit/Cost
Case I	20	3.98	12.86	0.309
Case II	30	17.45	18.16	0.961
Case III	30	2.84	13.11	0.217
Case IV	40	14.39	18.46	0.779

The second and third stages of regional development will commence after completion of the facilities proposed in the urgent plan. Therefore, the optimum pumping capacity can be determined on such an assumption that the facilities proposed in the urgent plan are the existing.

It has been concluded that the optimum pumping capacity should be 30 m^3/s in the second stage and 40 m^3/s in the third stages of regional development (refer to Fig. 2-8).

After the installation of a pumping station with the optimum pumping capacities stated above, it is very likely that a lot of area would still be submerged during a flood, although the area will be reduced.

It is economically reasonable to invest several times as much as the annual expected damage reduction. Thus a pumping capacity of about 150 m^3/s could be economically justified for the third development stage, but this would not coincide with the optimum. Since the exact conditions then applying to the area (after completion of the regional development plan) are not known now, a further study will be required to determine the most suitable capacity, based on the more realistic development plan.

2.5 Optimum Drainage System

The pumping station (30 m3/s of pump capacity in the second stage and 40 m3/s of pump capacity in the third stage) will be constructed at the right bank of the Pampang river at the Pampang bridge, and a check gate will be installed near the Pampang bridge. The drainage channels have been designed to meet each of the pumping capacities. It is important that the channel capacity and pumping capacity figures coincide with each other. Short-cuts will be provided in the heavily meandering lower reaches of Pampang bridge so as to increase its flow capacity and to lower its outlet water stage.

To prevent the reverse flow from the Tallo river, a sluice gate will be installed at Pannara where the municipal water channel is crossing. Also, the section of the road between Pannara and Sungguminasa, where the elevation of road surface is low, will be raised to prevent the flood water inflow from the Tallo river.

Figs. 2-9, 2-10 and 2-11 illustrate the location, longitudinal profile and standard cross-sections of the drainage channels respectively.

2.6 Required Earthwork

The required earthwork volume is estimated as follows. (Refer to Table 2-7 for details.)

Second : 240×10^3 m^3

Third (including Second): 660×10^3 m^3

A spoil bank can be prepared in the lowlands of the project area or along the Tallo river.

2.7 Land Acquisition and House Evacuation

The area of land acquisition consists of construction sites of the pumping station and the drainage channels.

The number of houses to be evacuated is obtained by making an actual counting of the houses spotted on aerophotographs of scale 1:20,000.

The land to be acquired and houses to be evacuated are as follows.

	Land Acquisition	Houses Evacuation
Second :	10 ha	12 houses
Third :	30 ha	20 houses

More detailed breakdown is shown in Table 2-8.

2.8 Drainage Facilities

Drainage Channel

The proposed drainage channel has a side-slope gradient of 1:1.0, and a channel width ranging from 6.3 m to 20.5 m (refer to Fig. 2-14). Major features of the drainage channel are shown in Table 2-9.

Since the Pampang river, Type A (No. T1-1.0-100 - No. T2-3.0) as shown in Fig. 2-9, is proposed in order to flow down the water drained by pumps, the freeboard of 0.5 m is included for the reason that the channel may be considered to be the same as rivers. No freeboard is considered on other drainage channels.

Since the drainage channel is to be constructed on the loose sand layers, the revetment will be constructed on the concrete foundation supported by wooden piles. The foundation is to be embedded in the ground for securing the stability, and the revetment is to be of wet masonry.

Roads are designed along the channel course on both sides for channel management. One of them with a width of 3.0 m enables vehicles to pass along and the other has a width of 1.5 m for pedestrians.

Pumping Station

The required pumping capacity is calculated at 30 m^3/s and 40 m^3/s in the second and third development stages respectively. The station will be set up at the Pampang bridge as shown in Fig. 2-12.

Geology of the proposed pumping station site is classified into alluvial deposits with a thickness of 20 m, consisting of sand with qc=40 kg/cm^2 and clay with qc=10 kg/cm^2, under which Neogene Teritiary sedimentary rocks lie.

From the longitudinal profile of the optimum drainage channel for overall plan as shown in Fig. 2-10, the actual head of the pumps is estimated at 1.2 m approximately.

Judging from the above, a horizontal axis mixed flow type pump, Q=5m^3/sec with 1,500 mm inside diameter is suitable for the system.

The proposed pumping station will require a floor space large enough for installation of eight pumps as six pumps are to be installed in the second development stage, and two more in the third stage.

Sluice

During the normal course of events, water in the project area flows naturally into the Tallo river. During a flood, however, the Tallo river water flows into the project area through the Pampang river and Pannara site. It is proposed, therefore, that the gates be built; one at the Pampang bridge site and the other at the Pannara site, as shown in Fig. 2-9. These will prevent reverse flow from the Tallo river. The structure of sluices at the Pampang bridge and at the Pannara site are shown in Figs. 2-12 and 2-13.

Road Raising

The boundary between the inner basin and the Tallo river basin is considered to be the road between Pannara and

Sungguminasa. A 100 m stretch of this road is low lying (3.5 m above M.S.L.), and during a big flood, flood water is likely to overtop this low stretch and flow into the inner basin.

Raising of this stretch is proposed as shown in Fig. 2-9.

3. FORMULATION OF URGENT DRAINAGE PLAN

The optimum drainage system over the city-side area including the first stage development area is studied and determined in this chapter from economical, technical and social viewpoints.

3.1 General Conception

Conception

The conception for planning is the same as the one in the overall plan.

General Study

Usually, a drainage scheme includes natural drainage, mechanical drainage and use of a regulation pond.

Within the scope of this urgent flood control plan, however, it is not necessary to use a mechanical drainage system because the lowest ground elevation in the first development stage area is 1.5 m above M.S.L. and is 0.94 m above the high water springs. Furthermore, it is economically unreasonable to provide space for a regulation pond. And even if a regulation pond could be provided, its effectiveness would be doubtful on the grounds that since the area is often subject to 5 days of continuous rainfall, any regulation pond would at least be partially filled within a first few days. In other words, the capacity of a regulation pond would be reduced well before the peak run-off occurs.

From the economic and technical viewpoints therefore, it is proposed that only a natural drainaae system should be employed in the context of the urgent flood control discussed herein.

The possible natural drainage is proposed as follows. (Refer to Fig. 3-1.)

1) Improvement of the existing drainage channels

   During a big flood, the Sinrijala channel has only the function of transporting the inundation water from the city-side area to the mountain-side area, because the Pampang river has an insufficient drainage capacity. Therefore, the Sinrijala channel will only be shaped to provide an usual drainage, and only the Panampu channel will be improved.

2) Excavation of new drainage channels

The possible new drainage channels are Jongaya and Monginsidi channels. The former is an open channel leading to the old Jeneberang river, and the latter is a culvert channel leading to the sea through the old urban area of Ujung Pandang city.

The right-of-way of the Jongaya channel lies mainly in the lowland with few houses, though evacuation of some houses in the densely populated area is unavoidable.

3.2 Study Procedure

Study Case

The optimum scale of drainage facilities should be determined by economical evaluation based on the estimate of cost and benefit. Besides the existing assets, the assets after completion of the first stage development are also assumed to be the assets existing in the project area. The existing facilities are regarded as the base to evaluate the urgent drainage plan.

A comparative study was conducted under the above mentioned conditions for the following cases.

Case I (Panampu channel)

Facilities : Existing

Assets : Existing

Case II (Panampu channel)

Facilities : Existing

Assets : After completion of the first stage development

Case III (Panampu + Jongaya)

Facilities : Existing

Assets : After completion of the first stage development

Case IV (Panampu + Monginsidi)

Facilities : Existing

Assets : After completion of the first stage development

Evaluation Method

The evaluation method for the urgent drainage improvement is the same as that for the overall drainage improvement (refer to page VI-4).

3.3 Premises of Study

The study on the optimum natural drainage system is based on the following premises.

1) Drainage condition

The inundation area is divided into two parts (city-side area and mountain-side area) by Jl. Panakkukang. The calculation model has two inundation areas.

The Sinrijala channel linking these two areas has the function of transporting the inundation water. If the inundation water depth in the inundation area exceeds 2.2 m, the inundation water overflows onto the Panakkukang road.

The drainage system should be designed to drain the inundation water, not the overtopped water from the Jeneberang. The drainage calculation has been based on the assumption that there is no overtopped water from the Jeneberang.

The inundation water would be drained by the Pampang river, through the Sinrijala and by the Panampu and Jongaya channels. Fig. 3-2 illustrates the calculation model.

2) Run-off

The suggested development work in the inner basin is expected to change the run-off coefficients in the area.

Most floods are caused by 5 days of continuous rainfall. The run-off after the first development stage has been estimated by using the rainfall intensity as shown in Fig. 2-4, considering the change of run-off coefficients.

3) Outlet water stage

The inundation water is drained into the Makassar Strait directly through Panampu and Jongaya channel, and into the Tallo river through Pampang river. The outlet water stage of each drainage channel is shown in Fig. 3-3. The outlet water stages of the Tallo and Pampang rivers cannot be estimated exactly because of insufficient hydraulic data of the Tallo river. Consequently the tidal curve had to be used to estimate their outlet water stages, taking into account the topographical gradient.

4) Cost estimation

Construction costs of the drainage system were estimated under the conditions mentioned below.

To determine the flow capacity of the channels, Manning's roughness coefficient (n) for the proposed channels is fixed at n = 0.025 for the open channel and n = 0.016 for the culvert, and the side slope gradient of the cross section is designed at 1:1.

The required direct cost consists of channel and bridge construction costs.

The indirect cost is fixed at 30 % of direct cost.

Land acquisition and house evacuation cost is also included.

5) Benefit estimation

The project benefit is calculated as the difference between the damages done with and without the new drainage system.

These estimated damages are assumed to be inflicted on the buildings and household effects existing after completion of the first development stage.

The effect of flood damage caused to agricultural land and crops has been disregarded in this benefit evaluation since it is considered to be almost negligible.

3.4 Determination of System Scale

Calculation Results

1) Inundation water stage

The outlet water stages of the Panampu channel and the proposed channels are uniform. Therefore, the total flow capacity of the channels can be calculated assuming that relation between the flow capacity and channel scale is same in any channel. The calculation results are summarized in Table 3-1.

2) Benefit estimation

The estimated benefits are presented in Table 3-2 and their calculation examples are shown in Table 3-3.

3) Cost estimation

The relation curves between the channel size and the required cost are shown in Fig. 3-4, and the breakdowns are given in Table 3-4.

Improvement of Panampu Channel (Case I and Case II)

In the case that only the Panampu channel is improved without providing new drainage channel, the optimum channel scale and drainage capacity of the Panampu channel are set forth in Table 3-5 (refer to Table 3-2, Figs. 3-5 and 3-6 for details).

Table 3-5 OPTIMUM SCALE OF PANAMPU CHANNEL

	Case I Existing Assets	Case II Assets after completion of the first stage
Channel Width (Bottom)	9 m	20 m
Maximum Drainage Capacity	28 m3/s	60 m3/s
B/C	0.10	0.21

As a matter of course, the B/C ratio calculated on a base of the assets after completion of the first stage is far bigger than that of the existing assets. The first stage will be completed in the near future. Therefore, the evaluation should be based upon the assets after completion of the first stage. However, the channelbed width has to be extended to 20 m which raises a social problem such as house evacuation (600 houses)

Panampu and New Channel (Case III and Case IV)

The above study certified that more than two systems would be required. A comparative study between Case III and IV was conducted to know the economically optimum system, based on the assets after completion of the first stage development, and its results are given in Table 3-6 (refer to Table 3-2 and Fig. 3-7 for details).

Table 3-6 OPTIMUM SCALE OF TWO DRAINAGE CHANNELS

	Case III		Case IV	
	Panampu	Jongaya	Panampu	Monginsidi
Channel Width (Bottom)	10 m	10 m	14 m	9 m x 3 m (Culbert)
Maximum Drainage Capacity	30 m3/s	30 m3	40 m3/s	40 m3/s
B/C Ratio	0.17		0.11	
House Evacuation	220	150	410	0

Note : The Sinrijala will be shaped only.

The combination of the Panampu and Jongaya channels was employed for the urgent plan for the reason of bigger B/C ratio, less number of house evacuation and easy maintenance.

The above is the result of a comparative study between Case III and IV, on the assumption that the drainage capacity of each channel is uniform. As stated before, Case III is employed for the project.

As the next stage, the allocation of the drainage capacity in Case III should be determined.

It is concluded that equivalent drainage capacity be allotted to the Panampu and Jongaya channels (refer to Table 3-7 and Fig. 3-8)

3.5 Optimum Drainage System

The optimum scale of both the Panumpu and Jongaya channels was studied to determine the most economical system. The optimum drainage system consists of the Panampu, Jongaya and Sinrijala channels and its essential features are tabulated below.

The location, longitudianl profile and standard cross sections are illustrated in Figs. 3-9, 3-10 and 3-11.

MAIN FEATURES OF THE PROPOSED DRAINAGE CHANNELS

	Panampu Channel	Jongaya Channel	Sinrijala Channel
Total Length	4.9 km	7.3 km*	2.3 km
Top Width	15.5 m	17.5 m	9.0 m
Bottom Width	10.5 m	12.5 m	3.2 m
Max. Depth	2.6 m	2.6 m	2.8 m
Channel gradient	1/4,300	1/6,400	1/5,700
Max. Drainage Capacity	30 m^3/s	30 m^3/s	7 m^3/s
Work required	Improvement	New excavation	Shaping

Note *:This includes the length of the old Jeneberang river.

3.6 Required Earthwork

The earthwork required for the drainage system in the urgent plan consists of excavation, dredging, filling for channel banking and spoil. The volumes of these works are given below.

Excavation:	399,000 m^3
Dredging :	139,000 m^3
Filling :	30,000 m^3
Spoil :	508,000 m^3

The breakdown of the excavation and dredging volume is given in Table 3-9.

A spoil bank can be found in the old Jeneberang course and in the lowland near the Hassanudding University. The total volume is estimated at 700 x 10^3 m^3.

3.7 Land Acquisition and Houses Evacuation

The area of land to be acquired and the number of houses to be evacuated are measured from the aerophotographs scaled 1/20,000, as given in the following. The breakdown is presented in Tabel 3-10.

	Land Acquisition	House Evacuation
Panampu :	6.2 ha	220 houses
Jongaya :	7.6 ha	150 houses
Sinrijala :	1.3 ha	0
Total	15.1 ha	370 houses

3.8 Drainage Facilities

Drainage Channel

The proposed drainage channel generally has a side-slope gradient of 1:1.0 and the section between points No. 0.0 and No. 2.4 of the Jongaya route (old Jeneberang river) will have to be dredged with the side slope gradient of 1:5.0. Since the proposed channel is not embanked, a freeboad is disregarded.

Roads are designed along the channel course on both sides for channel management. One of them is 3.0 m, and the other is 1.5 m in width.

The standard structure and major features of the proposed channels are shown in Fig. 3-14 and Table 3-8 respectively.

Sluice

Sluice equipped with a gate will be installed at 1.0 km of the Sinrijala channel as shown in Fig. 3-12 to control the reverse flow from the mountain-side area to the city-side area during a flood. Essential features of the proposed sluice are given in Table 3-11.

The cross-sectional area of the sluice is determined to be the same as that of its upper vicinity.

The structural design of the sluice is almost similar to the one described in "River Improvement" in the in the overall plan (refer to this Supporting Report, "River Improvement").

Bridge

The existing bridges crossing the Panampu and Sinrijala channels will be renewed because of the broadening of the channels. At the intersection of the newly proposed Jongaya route and the existing road, new bridges will be constructed. The location and the dimensions of these bridges are presented in Fig. 3-9 and Table 3-12.

The bridge existing at the intersection of the Sinrijala channel and Jl. Panakkukang will be replaced by the sluice mentioned above.

It is recommended that this structure be made of reinforced concrete and be of the simple beam type since not only all the materials are readily obtainable at the site but it will also be easy to construct.

As for the beam height, 0.6 m of clearance is added to the high-water level. The proposed length is 1.0 m over the newly proposed channel width. The width of a new bridge will accord with the existing width or the existing road width. Fig. 3-13 illustrates the standard design of the proposed bridge.

Table 1-1 INUNDATION SURVEY RESULT OF JAN. 1976 FLOOD

(Based on Information Based on Personal Inquiries)

Spot No.	Description	Remarks
1.	Tidal range, about 0.6 m. (based on tide marks). Revetment by cobble-concrete. No. overflow observed along the channel. Bottleneck is too small cross section of flow channel below the bridge.	Panampu No. 1 Bridge Site
16.	Flood inundation, 0.3 - 0.5 m. in depth, is observed over the refuse disposal ground. Continuous flood inundation area.	Refuse Disposal Ground
33.	Flood inundation, 0.1 m. in depth, above road surface is annually observed during rainly seasons. The 1976 Flood was not extra heavy.	
45.	No flood inundation reported as elevation of road surfaces is high. (Flood inundation, about 0.5 m. in deptch, over the neighboring lowland and houses submerged are also reported). Poor drainage is noticeable.	
83.	Flood inundation, about 0.2 m. in depth, above road surface is usually observed (due to poor drainage facilities).	Mission School
108.	No drainage problem in the new residential area (completed in 1977). Flood inundation, about 0.4 m. in depth, usually observed after 2-day rainfall.	
126.	Flood inundation, 0.3 - 0.5 m. in depth, for 1 - 2 days after rain is reported in January 1976. Normally, inundation is observed during rainly seasons caused by localized poor drainage. Longer the rain caused longer inundation.	

* Detail survey results are as shown in "Data Book"

Table 2-1 ESTIMATED INUNDATION WATER STAGE FOR OVERALL PLAN

(Unit:m)

	Return Period	Existing Facilities		Urgent Facilities		Proposed Facilities													
						$P=10m^3/s$		$P=20m^3/s$		$P=30m^3/s$		$P=50m^3/s$		$P=70m^3/s$		$P=100m^3/s$		$P=160m^3/s$	
	(yr)	H1	H2	H1	H2	H1	H2	H1	H2	H1	H2	H1	H2	H1	H2	H1	H2	H1	H2
Second Stage	2	1.46	2.03	1.31	1.54	1.24	1.52	1.09	1.51	1.01	1.51	0.89	1.51	0.78	1.51	0.71	1.51	0.43	1.51
	5	1.70	2.10	1.51	1.87	1.46	1.86	1.34	1.83	1.18	1.82	1.07	1.82	1.02	1.82	0.96	1.82	0.64	1.82
	10	1.84	2.14	1.65	2.01	1.60	2.01	1.50	2.01	1.31	2.00	1.14	2.00	1.09	2.00	1.05	2.00	0.79	2.00
	30	2.05	2.22	1.86	2.05	1.82	2.05	1.74	2.05	1.56	2.04	1.29	2.04	1.20	2.04	1.16	2.04	1.01	2.04
	50	2.15	2.28	1.97	2.07	1.93	2.07	1.85	2.07	1.69	2.06	1.37	2.06	1.25	2.06	1.21	2.06	1.05	2.06
Third Stage	2	1.48	2.03	1.33	1.55	1.26	1.53	1.13	1.51	1.04	1.50	0.96	1.51	0.85	1.51	0.71	1.51	0.51	1.51
	5	1.72	2.10	1.54	1.87	1.49	1.87	1.38	1.84	1.21	1.82	1.09	1.81	1.05	1.82	0.96	1.82	0.70	1.82
	10	1.87	2.14	1.68	2.01	1.63	2.01	1.54	2.01	1.35	2.01	1.17	2.00	1.12	2.00	1.05	2.00	1.01	2.00
	30	2.08	2.23	1.90	2.05	1.86	2.05	1.78	2.05	1.61	2.04	1.33	2.04	1.23	2.04	1.16	2.04	1.04	2.04
	50	2.19	2.29	2.01	2.07	1.97	2.07	1.90	2.07	1.75	2.07	1.42	2.06	1.28	2.06	1.21	2.06	1.08	2.06

H1 : Water stage in Mountain side Area
H2 : Water stage in City side Area

Table 2—2 ECONOMICAL JUSTIFICATION OF POSSIBLE DRAINAGE SYSTEM FOR OVERALL PLAN

After Completion of Second Stage Development

Case	Facility	P (m^3/s)	B ($x10^9$ Rp)	C ($x10^9$ Rp)	α
Case I	Existing	5	2.139	7.46	0.287
		10	2.737	9.14	0.299
		20	3.980	12.86	0.309
		30	4.880	16.51	0.296
		40	5.622	20.14	0.279
		50	6.164	23.75	0.278
Case III	After Urgent Plan	10	0.658	5.76	0.114
		20	1.704	9.46	0.180
		30	2.840	13.11	0.217
		40	3.128	16.74	0.187
		50	3.296	20.35	0.109

After Completion of Third Stage Development

Case	Facility	P (m^3/s)	B ($x10^9$ Rp)	C ($x10^9$ Rp)	α
Case II	Existing	10	8.372	10.21	0.820
		20	12.497	14.17	0.882
		30	17.450	18.16	0.961
		40	19.768	21.86	0.943
		70	24.007	33.38	0.719
		100	24.798	44.46	0.558
		160	29.135	66.36	0.439
Case IV	After Urgent Plan	10	3.475	6.81	0.510
		20	6.917	10.77	0.642
		30	10.563	14.76	0.716
		40	14.390	18.46	0.779
		70	17.795	29.98	0.594
		100	18.730	41.06	0.458
		160	23.069	62.96	0.248

B; Annual Average Flood Damage Reduction

C; Construction Cost

α; B/C

P; Drainage Capacity

Table 2—3 (1) ANNUAL EXPECTED FLOOD DAMAGE REDUCTION FOR OVERALL PLAN

- Case I -

Assets: Second Stage

Drainage Condition: Existing Facilities, Pump Capacity: 30 m^3/s

1	2		3		4		5	6	7	8
Return Period (1/T)	Inundation Water Stage (m)				Flood Damage (10^6Rp)		Flood Damage Reduction (10^6Rp)	Average (10^6Rp)	Expected Value	6 x 7 (10^6Rp)
	A		B		A	B				
	H_1	H_2	H_1	H_2						
1/1	1.28	2.04	0.88	1.26	3.9	0.8	3.1			
								3.3	0.500	1.65
1/2	1.46	2.06	1.01	1.51	5.5	2.0	3.5			
								4.4	0.300	1.32
1/5	1.70	2.10	1.18	1.82	8.0	2.7	5.3			
								5.8	0.100	0.58
1/10	1.84	2.10	1.31	2.00	10.0	3.7	6.3			
								6.6	0.067	0.442
1/30	2.05	2.22	1.56	2.04	13.1	6.2	6.9			
								7.35	0.013	0.069
1/50	2.15	2.28	1.69	2.06	15.6	7.8	7.8			
								7.80	0.020	0.156
								Direct Damage	:	4.244
								Indirect Damage:		0.636
								Grand Total	:	4.880

NOTE:
A: Without Pump
B: With Pump
H_1: Inundation Water Stage in the Mountain-Side Area
H_2: Inundation Water Stage in the City-Side Area

Table 2—3(2) ANNUAL EXPECTED FLOOD DAMAGE REDUCTION FOR OVERALL PLAN

- Case I -

Assets: Second Stage

Drainage Condition: Existing Facilities, Pump Capacity: 40 m^3/s

1	2		3		4		5	6	7	8
Return Period (1/T)	Inundation Water Stage (m)				Flood Damage (10^6Rp)		Flood Damage Reduction (10^6Rp)	Average (10^6Rp)	Expected Value	6 x 7 (10^6Rp)
	A		B		A	B				
	H_1	H_2	H_1	H_2						
1/1	1.28	2.04	0.82	1.26	3.9	0.4	3.5			
								3.95	0.500	1.975
1/2	1.46	2.06	0.95	1.51	5.5	1.0	4.4			
								4.95	0.300	1.485
1/5	1.70	2.10	1.09	1.82	8.0	2.5	5.5			
								6.05	0.100	0.605
1/10	1.84	2.14	1.20	2.00	10.0	3.4	6.6			
								7.55	0.067	0.506
1/30	2.05	2.22	1.38	2.04	13.1	4.6	8.5			
								9.20	0.013	0.120
1/50	2.15	2.28	1.47	2.06	15.6	5.7	9.9			
								9.90	0.020	0.198
								Direct Damage :		4.389
								Indirect Damage:		0.733
								Grand Total :		5.622

NOTE:
- A: Without Pump
- B: With Pump
- H_1: Inundation Water Stage in the Mountain-Side Area
- H_2: Inundation Water Stage in the City-Side Area

Table 2—3 (3) ANNUAL EXPECTED FLOOD DAMAGE REDUCTION FOR OVERALL PLAN

- Case III -

Assets: Second Stage

Drainage Condition: Urgent Facilities, Pump Capacity: 30 m^3/s

1	2		3		4		5	6	7	8
Return Period (1/T)	Inundation Water Stage (m)				Flood Damage(10^6Rp)		Flood Damage Reduction (10^6Rp)	Average (10^6Rp)	Expected Value	6 x 7 (10^6Rp)
	A		B		A	B				
	H_1	H_2	H_1	H_2						
1/1	1.16	1.30	0.88	1.26	2.4	0.8	1.6			
								1.9	0.500	0.950
1/2	1.31	1.54	1.01	1.51	3.4	1.2	2.2			
								2.5	0.300	0.750
1/5	1.51	1.87	1.18	1.82	5.4	2.7	2.7			
								3.4	0.100	0.340
1/10	1.65	2.01	1.31	2.00	7.6	3.6	4.0			
								4.4	0.067	0.290
1/30	1.86	2.05	1.56	2.04	9.8	5.0	4.8			
								4.4	0.013	0.060
1/50	1.97	2.07	1.69	2.06	11.8	2.8	4.0			
								4.0	0.020	0.080
								Direct Damage :		2.470
								Indirect Damage:		0.370
								Grand Total :		2.840

NOTE:
A: Without Pump
B: With Pump
H_1: Inundation Water Stage in the Mountain-Side Area
H_2: Inundation Water Stage in the City-Side Area

Table 2—3 (4) ANNUAL EXPECTED FLOOD DAMAGE REDUCTION FOR OVERALL PLAN

- Case III -

Assets: Second Stage

Drainage Condition: Urgent Facilities, Pump Capacity: 40 m^3/s

1	2		3		4		5	6	7	8
Return Period (1/T)	Inundation Water Stage (m)				Flood Damage (10^6Rp)		Flood Damage Reduction (10^6Rp)	Average (10^6Rp)	Expected Value	6 x 7 (10^6Rp)
	A		B		A	B				
	H_1	H_2	H_1	H_2						
1/1	1.16	1.30	0.82	1.26	2.4	0.4	2.0			
								2.15	0.500	1.075
1/2	1.31	1.54	0.95	1.51	3.4	1.1	2.3			
								2.60	0.300	0.780
1/5	1.51	1.87	1.12	1.82	5.4	2.5	2.9			
								3.55	0.100	0.355
1/10	1.65	2.01	1.22	2.00	7.6	3.4	4.2			
								4.70	0.067	0.315
1/30	1.86	2.05	1.42	2.09	9.8	4.6	5.2			
								5.65	0.013	0.073
1/50	1.97	2.07	1.53	2.06	11.8	5.7	6.1			
								6.10	0.020	0.122
Direct Damage : 2.720										
Indirect Damage: 0.408										
Grand Total : 3.128										

NOTE:
A: Without Pump
B: With Pump
H_1: Inundation Water Stage in the Mountain-Side Area
H_2: Inundation Water Stage in the City-Side Area

Table 2—3 (5) ANNUAL EXPECTED FLOOD DAMAGE REDUCTION FOR OVERALL PLAN

- Case II -

Assets: Third Stage

Drainage Condition: Existing Facilities, Pump Capacity: 30 m^3/s

1	2		3		4		5	6	7	8
Return Period (1/T)	Inundation Water Stage (m)				Flood Damage (10^6Rp)		Flood Damage Reduction (10^6Rp)	Average (10^6Rp)	Expected Value	6 x 7 (10^6Rp)
	A		B							
	H_1	H_2	H_1	H_2	A	B				
1/1	1.31	1.96	0.90	1.25	17.3	5.0	12.3			
								13.50	0.500	6.575
1/2	1.48	2.03	1.04	1.50	22.5	8.5	14.0			
								16.05	0.300	4.815
1/5	1.72	2.10	1.21	1.82	32.3	14.2	18.1			
								18.30	0.100	1.830
1/10	1.87	2.14	1.35	2.01	37.9	19.4	18.5			
								18.85	0.067	1.263
1/30	2.08	2.23	1.61	2.04	47.7	28.5	19.2			
								20.30	0.013	0.428
1/50	2.19	2.29	1.75	2.07	55.0	33.6	21.4			
								21.40	0.020	0.428
								Direct Damage :		15.174
								Indirect Damage:		2.276
								Grand Total :		17.450

NOTE:
- A: Without Pump
- B: With Pump
- H_1: Inundation Water Stage in the Mountain-Side Area
- H_2: Inundation Water Stage in the City-Side Area

Table 2—3 (6) ANNUAL EXPECTED FLOOD DAMAGE REDUCTION FOR OVERALL PLAN

- Case II -

Assets: Third Stage

Drainage Condition: Existing Facilities, Pump Capacity: 40 m^3/s

1	2		3		4		5	6	7	8
Return Period (1/T)	Inundation Water Stage (m)				Flood Damage (10^6Rp)		Flood Damage Reduction (10^6Rp)	Average (10^6Rp)	Expected Value	6 x 7 (10^6Rp)
	A		B		A	B				
	H_1	H_2	H_1	H_2						
1/1	1.31	1.96	0.86	1.25	17.3	3.5	13.8			
								14.05	0.500	7.025
1/2	1.48	2.03	1.01	1.51	22.5	8.2	14.3			
								17.50	0.300	5.250
1/5	1.72	2.10	1.12	1.81	32.3	11.6	20.7			
								21.80	0.100	2.180
1/10	1.87	2.14	1.24	2.00	37.9	15.0	22.9			
								25.60	0.067	1.716
1/30	2.08	2.23	1.42	2.04	47.7	19.4	28.3			
								29.90	0.013	0.389
1/50	2.19	2.29	1.53	2.06	55.0	23.5	31.5			
								31.50	0.020	0.630
								Direct Damage :		17.190
								Indirect Damage:		2.578
								Grand Total :		19.768

NOTE:
- A: Without Pump
- B: With Pump
- H_1: Inundation Water Stage in the Mountain-Side Area
- H_2: Inundation Water Stage in the City-Side Area

Table 2-3 (7) ANNUAL EXPECTED FLOOD DAMAGE REDUCTION FOR OVERALL PLAN

- Case IV -

Assets: Third Stage

Drainage Condition: Urgent Facilities, Pump Capacity: 30 m^3/s

1	2		3		4		5	6	7	8
Return Period (1/T)	Inundation Water Stage (m)				Flood Damage (10^6Rp)		Flood Damage Reduction (10^6Rp)	Average (10^6Rp)	Expected Value	6 x 7 (10^6Rp)
	A		B		A	B				
	H_1	H_2	H_1	H_2						
1/1	1.16	1.32	0.90	1.25	12.5	6.4	6.1	7.55	0.500	3.775
1/2	1.33	1.55	1.04	1.50	18.0	9.0	9.0	10.30	0.300	3.090
1/5	1.54	1.87	1.21	1.82	26.0	14.4	11.6	12.05	0.100	1.210
1/10	1.68	2.01	1.35	2.01	31.7	19.2	12.5	12.35	0.067	0.830
1/30	1.90	2.05	1.61	2.04	41.0	28.8	12.2	12.00	0.013	0.160
1/50	2.01	2.07	1.75	2.07	46.3	34.5	11.8	11.80	0.020	0.120
								Direct Damage :		9.185
								Indirect Damage:		1.378
								Grand Total :		10.563

NOTE:
A: Without Pump
B: With Pump
H_1: Inundation Water Stage in the Mountain-Side Area
H_2: Inundation Water Stage in the City-Side Area

Table 2—3(8) ANNUAL EXPECTED FLOOD DAMAGE REDUCTION FOR OVERALL PLAN

\- Case IV -

Assets: Third Stage

Drainage Condition: Urgent Facilities, Pump Capacity: 40 m^3/s

1	2		3		4		5	6	7	8
Return Period (1/T)	Inundation Water Stage (m)				Flood Damage(10^6Rp)		Flood Damage Reduction (10^6Rp)	Average (10^6Rp)	Expected Value	6 x 7 (10^6Rp)
	A		B		A	B				
	H_1	H_2	H_1	H_2						
1/1	1.16	1.32	0.86	1.25	12.5	3.5	9.0			
								10.55	0.500	5.075
1/2	1.33	1.55	1.01	1.51	18.0	6.7	11.3			
								12.85	0.300	3.855
1/5	1.54	1.87	1.12	1.81	26.0	11.6	14.4			
								15.55	0.100	1.555
1/10	1.68	2.01	1.24	2.00	31.7	15.0	16.7			
								19.15	0.067	1.283
1/30	1.90	2.05	1.42	2.04	41.0	19.4	21.6			
								22.20	0.013	0.289
1/50	2.01	2.07	1.53	2.06	46.3	23.5	22.8			
								22.80	0.020	0.456
								Direct Damage :		12.513
								Indirect Damage:		1.877
								Grand Total :		14.390

NOTE:
A: Without Pump
B: With Pump
H_1: Inundation Water Stage in the Mountain-Side Area
H_2: Inundation Water Stage in the City-Side Area

Table 2-4(1) CONSTRUCTION COST COMPARISON BY CHANNEL CAPACITY FOR OVERALL PLAN

Unit: Rp

Section A (T1-1.0-100m --- T2-3.0)

Capacity \ Item	Channel	Bridge	Land Acquisition	House Evacuation	Total
Q = 25m^3/s	306.0x10^6	-	110.6x10^6	-	416.6x10^6
Q = 45	370.0	-	137.6		507.6
Q = 90	472.0	-	191.6	-	663.6

Section B (T2-3.0 -- R10)

Capacity \ Item	Channel	Bridge	Land Acquisition	House Evacuation	Total
Q = 15m^3/s	782.2x10^6	-	15.9x10^6	-	798.1x10^6
Q = 55	1226.1	-	31.7	-	1257.8
Q = 135	1860.3	-	63.4	-	1923.7

Section B (T2-3.0 -- R7+100m)

Capacity \ Item	Channel	Bridge	Land Acquisition	House Evacuation	Total
Q = 15m^3/s	512.8x10^6	-	10.4x10^6	-	523.2x10^6
Q = 55	803.9	-	20.8	-	824.7
Q = 135	1219.7	-	41.6	-	1261.3

Section C (R10 --T16)

Capacity \ Item	Channel	Bridge	Land Acquisition	House Evacuation	Total
Q = 25m^3/s	746.6 x10^6	-	19.5x10^6	-	766.1x10^6
Q = 45	934.1	-	20.7	-	954.8
Q = 90	1198.5	-	31.0	-	1229.5

Table 2-4(2) CONSTRUCTION COST COMPARISON BY CHANNEL CAPACITY FOR OVERALL PLAN

Unit : Rp.

Section D (B -- R10)

Item / Capacity	Channel	Bridge	Land Acquisition	House Evacuation	Total
Q = 15m /s	423.9x10	-	8.8x10	-	441.7x10
Q = 60m /s	678.6	-	17.6	-	696.2
Q = 125m /s	931.3	-	29.3	-	960.6

Section E1 (R10 -- K)

Item / Capacity	Channel	Bridge	Land Acquisition	House Evacuation	Total
Q = 15m /s	492.1x10	-	10.0x10	-	502.1x10
Q = 60m /s	771.4	-	20.0	-	791.1
Q = 150m /s	1170.4	-	40.0	-	1210.4

Section E2 (K -- M)

Item / Capacity	Channel	Bridge	Land Acquisition	House Evacuation	Total
Q = 25m3/s	322.0x10	-	6.4x10	-	328.4x10
Q = 40	370.0	-	7.5	-	377.5
Q = 75	452.0	-	10.0	-	462.0

Table 2-5 CONSTRUCTION COST COMPARISON BY PUMP CAPACITY FOR OVERALL PLAN

Pump Capacity	Pumping Station (Rp)			Land Aquisition (Rp)	House Evacuation (Rp)	Total (Rp)
	Pump	Station House	Civil Work			
10 m^3/s	2500 $x10^6$	500 $x10^6$	2000 $x10^6$	1.2 $x10^6$	0	5001.2 $x10^6$
20	4250	850	3400	1.8	0	8501.8
50	9500	1900	7600	4.7	0	19004.7
100	18500	3500	14500	7.0	0	36507.0
160	28700	5800	23000	8.3	0	57508.3

Table 2—7 EARTH WORK VOLUME FOR OVERALL PLAN

Second Stage

Section	Excavation Volume ($x10^3$ m^3)
A (T1-1.0 – 100 m ...... T2-3.0)	72.0
B (T2-3.0 ...... R10)	163.5
Total	240×10^3 m^3

Third Stage

Section	Excavation Volume ($x10^3$ m^3)
A (T1-1.0 – 100 m ...... T2-3.0)	855.0
B (T2-3.0 ...... R10)	287.5
C (R10 ...... R16)	119.9
D (B ...... R10)	51.5
E (R10 ...... K)	69.2
F (K ...... M)	40.0
Total	660×10^3 m^3

Table 2-8 LAND AQUISITION AND HOUSE EVACUATION FOR OVERALL PLAN

Second Stage

		Land Aquisition				House Evacuation
		L (m)	B1(m)	B2(m)	A (m^2)	
Pumping Station		—	—	—	6,200	0
Channel	Section A	1,500	12.88	4.5	26,070	12
	Section B	2,772	17.51	4.5	61,011	0
Total					93,300	12

Third Stage

		Land Aquisition				House Evacuation
		L (m)	B1(m)	B2(m)	A (m^2)	
Pumping Station		—	—	—	7,800	0
Channel	Section A	1,500	14.88	4.5	29,070	20
	Section B	4,228	20.51	4.5	105,742	0
	Section C	4,122	8.51	4.5	53,770	0
	Section D	2,340	7.01	4.5	26,933	0
	Section E1	2,660	8.01	4.5	33,277	0
	Section E2	2,000	6.31	4.5	21,620	0
Total					278,200	20

L : Channel Length
B1: Channel Width
B2: Width of Maintenance Road
A : Land Aquisition Area

Table 2-9 FEATURES OF THE PROPOSED DRAINAGE CHANNELS

Cross Section No.	Length (m)	Type of Cross Section	Discharge (m^3/s)	Channel Cross Section				Longitudinal Gradient
				Top(m)	Bottom(m)	Depth(m)	Slope Gradient	
T1-1.0-100 - T2-3.0	1,500	A	40.0	14.88	6.00	3.94	1:1	1/4,170
T2-3.0 - R10	4,228	B	40.0	20.51	15.00	2.76	1:1	1/6,000
R10 - R14	2,124	C	16.7	8.51	3.00	2.76	1:1	1/2,200
B - R10	2,340	D	6.3	7.01	1.50	2.76	1:1	1/4,500
R10 - K	2,660	E1	8.2	8.01	2.50	2.76	1:1	1/5,000
K - M	2,000	E2	8.2	6.30	0.80	2.76	1:1	1/ 770
Total	14,862							

Table 3-1 ESTIMATED INUNDATION WATER STAGE FOR URGENT PLAN

First Stage

Unit : m

Return Period (Yr)	Discharge Capacity* (m^3/s)	Channel Bed width (m)	Water Stage (Sinrijala Channel : Shaping) H1	H2
2	Existing	–	1.45	2.03
	30	10	1.32	1.79
	60	20	1.30	1.54
	75	25	1.30	1.36
	150	50	1.29	1.12
5	Existing	–	1.69	2.10
	30	10	1.52	2.02
	60	20	1.50	1.87
	75	75	1.49	1.64
	150	50	1.48	1.28
10	Existing	-	1.83	2.14
	30	10	1.66	2.05
	60	20	1.64	2.01
	75	25	1.63	1.81
	150	50	1.62	1.41
30	Existing		2.04	2.22
	30	10	1.88	2.09
	60	20	1.85	2.05
	75	25	1.84	2.02
	150	50	1.82	1.62
50	Existing	--	2.14	2.28
	30	10	1.18	2.12
	60	20	1.95	2.07
	75	25	1.94	2.03
	150	50	1.92	1.72

* : Panampu, Mouginsidi Jongaya Route. H1 : Water Stage in Mountain Side Area. H2 : Water Stage in City Side Area

Table 3-2 ECONOMICAL JUSTIFICATION OF POSSIBLE DRAINAGE SYSTEM FOR URGENT PLAN

Existing Property
Panampu: Improvement, Sinrijala: Shaping

Case I

Panampu width (m)	Discharge (m^3/s)	B ($x10^9$Rp)	C ($x10^9$Rp)	α
5	15	0.137	1.29	0.106
10	30	0.194	1.70	0.114
20	60	0.239	2.40	0.099
30	90	0.275	3.25	0.085

First Stage Property
Panampu: Improvement, Sinrijala: Shaping

Case II

Panampu width (m)	Discharge (m^3/s)	B ($x10^9$Rp)	C ($x10^9$Rp)	α
10	30	0.378	1.70	0.222
20	60	0.588	2.40	0.245
30	90	0.699	3.25	0.215
50	150	0.751	4.95	0.152

First Stage Property
Panampu and Jongaya: Improvement, Sinrijala: Shaping

Cses III

Panampu and Jongaya width (m)	Discharge (m^3/s)	B ($x10^9$Rp)	C ($x10^9$Rp)	α
10	30	0.378	2.50	0.151
20	60	0.588	3.40	0.178
30	90	0.699	4.40	0.159
50	150	0.751	6.00	0.125

First Stage Property
Panampu and Monginsidi: Improvement, Sinrijala: Shaping

Case IV

Panampu and Monginisidi width (m)	Discharge (m^3/s)	B ($x10^9$Rp)	C ($x10^9$Rp)	α
10	30	0.378	4.70	0.080
20	60	0.588	5.85	0.100
30	90	0.699	6.70	0.104
50	150	0.751	8.15	0.092

B: Annual Expected Damage Reduction

C: Construction Cost

α: B/C

Table 3—3 (1) ANNUAL EXPECTED FLOOD DAMAGE REDUCTION FOR URGENT PLAN

- Case I -

Assets: Existing

Sinrijala: Existing, Panampu: B=10 m ($Q=30m^3/s$)

1	2		3		4		5	6	7	8
Return Period (1/T)	Inundation Water Stage (m)				Flood Damage(10^6Rp)		Flood Damage Reduction (10^6Rp)	Average (10^6Rp)	Expected Value	6 x 7 (10^6Rp)
	①		②		①	②				
	H_1	H_2	H_1	H_2						
1/1	1.36	1.97	1.21	1.65	250	80	170			
								180	0.500	90.0
1/2	1.52	2.04	1.35	1.80	340	150	190			
								140	0.300	42.0
1/5	1.74	2.10	1.53	2.02	400	310	90			
								130	0.100	13.0
1/10	1.87	2.14	1.64	2.05	500	330	170			
								195	0.067	13.1
1/30	2.04	2.22	1.82	2.09	660	440	220			
								295	0.013	3.8
1/50	2.14	2.28	1.91	2.12	840	470	370			
								370	0.020	7.4
								Direct Damage :		169.3
								Indirect Damage:		25.4
								Grand Total :		194.7

NOTE:
①: Existing Condition of Panampu and Sinrijala Channels
②: After Completion of Urgent Drainage Plan
H_1: Inundation Water Stage in the Mountain-Side Area
H_2: Inundation Water Stage in the City-Side Area
B : Bottom Width of Channel. Q: Discharge of Channel

Table 3-3 (2) ANNUAL EXPECTED FLOOD DAMAGE REDUCTION FOR URGENT PLAN

- Case I -

Assets: Existing

Sinrijala: Existing, Panampu: B=20 m ($Q=60m^3/s$)

1	2		3		4		5	6	7	8
Return Period (1/T)	Inundation Water Stage (m)				Flood Damage (10^6Rp)		Flood Damage Reduction (10^6Rp)	Average (10^6Rp)	Expected Value	6 x 7 (10^6Rp)
	①		②		①	②				
	H_1	H_2	H_1	H_2						
1/1	1.36	1.97	1.17	1.30	250	70	180	215	0.500	107.5
1/2	1.52	2.04	1.31	1.55	340	90	250	175	0.300	52.5
1/5	1.74	2.10	1.49	1.87	400	30	100	175	0.100	17.5
1/10	1.87	2.14	1.60	2.01	500	25	250	260	0.067	17.4
1/30	2.04	2.22	1.75	2.05	660	39	270	355	0.013	4.6
1/50	2.14	2.28	1.84	2.07	840	40	440	440	0.020	8.8
								Direct Damage :		208.3
								Indirect Damage:		31.2
								Grand Total :		239.5

NOTE:
- ①: Existing Condition of Panampu and Sinrijala Channels
- ②: After Completion of Urgent Drainage Plan
- H_1: Inundation Water Stage in the Mountain-Side Area
- H_2: Inundation Water Stage in the City-Side Area
- B : Bottom Width of Channel. Q : Discharge of Channel

Table 3–3 (3) ANNUAL EXPECTED FLOOD DAMAGE REDUCTION FOR URGENT PLAN

\- Case I -

Assets: Existing

Sinrijala: Existing, Panampu: B= 30 m ($Q=90m^3/s$)

1	2		3		4		5	6	7	8
Return Period (1/T)	Inundation Water Stage (m)				Flood Damage(10^6Rp)		Flood Damage Reduction (10^6Rp)	Average (10^6Rp)	Expected Value	6 x 7 (10^6Rp)
	①		②		①	②				
	H_1	H_2	H_1	H_2						
1/1	1.36	1.97	1.14	1.15	250	60	190	225	0.500	112.5
1/2	1.52	2.04	1.28	1.37	340	80	260	220	0.300	66.0
1/5	1.74	2.10	1.46	1.64	400	220	180	240	0.100	24.0
1/10	1.87	2.14	1.57	1.83	500	200	300	310	0.067	20.8
1/30	2.04	2.22	1.72	2.01	660	340	320	415	0.013	5.4
1/50	2.14	2.28	1.80	2.03	840	330	510	510	0.020	10.2
								Direct Damage :		238.9
								Indirect Damage:		35.8
								Grand Total :		274.7

NOTE:
①: Existing Condition of Panampu and Sinrijala Channels
②: After Completion of Urgent Drainage Plan
H_1: Inundation Water Stage in the Mountain-Side Area
H_2: Inundation Water Stage in the City-Side Area
B : Bottom Width of Channel. Q : Discharge of Channel

Table3—3 (4) ANNUAL EXPECTED FLOOD DAMAGE REDUCTION FOR URGENT PLAN

- Case II, III, IV -

Assets: First Stage

Sinrijala: Existing, Panampu: B= 10 m ($Q=30m^3/s$)

1	2		3		4		5	6	7	8
Return Period (1/T)	Inundation Water Stage (m) ①		Inundation Water Stage (m) ②		Flood Damage (10^6Rp) ①	Flood Damage (10^6Rp) ②	Flood Damage Reduction (10^6Rp)	Average (10^6Rp)	Expected Value	6 x 7 (10^6Rp)
	H_1	H_2	H_1	H_2						
1/1	1.36	1.97	1.21	1.65	570	90	480			
								440	0.500	220.0
1/2	1.52	2.04	1.35	1.80	710	310	400			
								350	0.300	105.0
1/5	1.74	2.10	1.53	2.02	850	660	190			
								180	0.100	180.0
1/10	1.87	2.14	1.64	2.05	940	770	170			
								300	0.067	20.0
1/30	2.04	2.22	1.77	2.09	1230	800	430			
								460	0.013	6.0
1/50	2.14	2.28	1.88	2.11	1440	960	480			
								480	0.020	10.0
								Direct Damage :		329.0
								Indirect Damage:		49.4
								Grand Total :		378.4

NOTE:
①: Existing Condition of Panampu and Sinrijala Channels
②: After Completion of Urgent Drainage Plan
H_1: Inundation Water Stage in the Mountain-Side Area
H_2: Inundation Water Stage in the City-Side Area
B : Bottom Width of Channel. Q : Discharge of Channel

Table 3—3 (5) ANNUAL EXPECTED FLOOD DAMAGE REDUCTION FOR URGENT PLAN

- Case II, III, IV -

Assets: First Stage

Sinrijala: Existing, Panampu: B=20 m ($Q=60m^3/s$)

1	2		3		4		5	6	7	8
Return Period (1/T)	Inundation Water Stage (m)				Flood Damage(10^6Rp)		Flood Damage Reduction (10^6Rp)	Average (10^6Rp)	Expected Value	6 x 7 (10^6Rp)
	①		②		①	②				
	H_1	H_2	H_1	H_2						
1/1	1.36	1.97	1.17	1.30	570	80	490			
								275	0.500	275.0
1/2	1.52	2.04	1.31	1.55	710	100	610			
								156	0.300	156.0
1/5	1.74	2.10	1.40	1.87	850	420	430			
								36	0.100	36.0
1/10	1.87	2.14	1.60	2.01	940	660	280			
								25	0.067	25.0
1/30	2.04	2.22	1.71	2.05	1230	750	480			
								7	0.013	7.0
1/50	2.14	2.28	1.79	2.07	1440	820	620			
								12	0.020	12.0
								Direct Damage :		511.0
								Indirect Damage:		76.7
								Grand Total :		587.7

NOTE:
- ①: Existing Condition of Panampu and Sinrijala Channels
- ②: After Completion of Urgent Drainage Plan
- H_1: Inundation Water Stage in the Mountain-Side Area
- H_2: Inundation Water Stage in the City-Side Area
- B : Bottom Width of Channel, Q : Discharge of Channel

Table 3—3 (6) ANNUAL EXPECTED FLOOD DAMAGE REDUCTION FOR URGENT PLAN

- Case II, III, IV -

Assets: First Stage

Sinrijala: Existing, Panampu: B=30 m ($Q=90m^3/s$)

1	2		3		4		5	6	7	8
Return Period (1/T)	Inundation Water Stage (m)				Flood Damage (10^6Rp)		Flood Damage Reduction (10^6Rp)	Average (10^6Rp)	Expected Value	6 x 7 (10^6Rp)
	①		②		①	②				
	H_1	H_2	H_1	H_2						
1/1	1.36	1.97	1.14	1.15	570	70	500			
								560	0.500	280.0
1/2	1.52	2.04	1.28	1.37	710	100	610			
								660	0.300	198.8
1/5	1.74	2.10	1.46	1.64	850	140	710			
								650	0.100	65.0
1/10	1.87	2.14	1.57	1.83	940	360	580			
								570	0.067	38.0
1/30	2.04	2.22	1.68	2.02	1230	680	550			
								620	0.013	8.0
1/50	2.14	2.28	1.75	2.03	1440	750	690			
								690	0.020	14.0
								Direct Damage :		608.0
								Indirect Damage:		91.2
								Grand Total :		699.2

NOTE:
①: Existing Condition of Panampu and Sinrijala Channels
②: After Completion of Urgent Drainage Plan
H_1: Inundation Water Stage in the Mountain-Side Area
H_2: Inundation Water Stage in the City-Side Area
B : Bottom Width of Channel. Q : Discharge of Channel

Table 3-4 CONSTRUCTION COST COMPARISON BY CHANNEL CAPACITY FOR URGENT PLAN

Unit : 10^6Rp

Panampu (No. 0.0 – P1)

Capacity \ Item	Channel	Bridge	Land Aquisition	House evacuation	Total
5 m3/s	504.0	235.0	110.6	3.0	870.6
10	674.8	337.0	154.4	12.0	1178.2
20	832.6	405.0	189.3	34.0	1460.9
95	1462.2	1235.0	612.2	172.0	3481.4

Jongaya (No. 2.4 – P1)

Capacity \ Item	Channel	Bridge	Land Aquisition	House evacuation	Total
5 m3/s	547.7	220.0	114.3	2.0	884.0
10	733.5	295.0	138.1	6.0	1172.6
20	905.0	353.0	157.2	16.0	1431.2
90	1589.4	1070.0	386.8	86.0	3131.9

Sinrijala (No. 0.0 – P1)

Capacity \ Item	Channel	Bridge	Land Aquisition	House evacuation	Total
5 m3/s	319.4	55.0	33.2	1.0	408.6
10	427.6	72.0	51.0	3.0	553.6
20	535.8	87.0	67.0	10.0	699.8
90	966.5	250.0	252.1	50.0	1518.6

Monginsidi (No. P11-2 – P1)

Capacity \ Item	Channel	Bridge	Land Aquisition	House evacuation	Total
5 m3/s	2723.0	–	–	–	2723.0
10	3283.0	–	–	–	3283.0
20	3844.0	–	–	–	3844.0
120	5874.0	–	–	–	5874.0

Table 3-7 ALLOCATION OF DRAINAGE CAPACITY

Plan	Channel	Section	Length of Channel (m)	Discharge of Channel (m^3/s)	Bottom width of Channel (m)	Longitudinal Gradient	Construction Cost ($\times 10^6$ Rp)
Plan (A)	Panampu	No. 0.0 ~ H_{11}	6704.75	39.6	16.00	1/5890	3,287
	Jongaya	No. 0.0 ~ H_{11}	5459.00	20.4	8.00	1/4790	
Plan (B)	Panampu	No. 0.0 ~ H_3	4901.75	30.6	10.50	1/4300	3,014
	Jongaya	No. 0.0 ~ H_3	7262.00	29.4	12.50	1/6400	
Plan (C)	Panampu	No. 0.0 ~ H_7	5694.75	33.5	13.00	1/5000	3,139
	Jongaya	No. 0.0 ~ H_7	6469.00	26.5	10.50	1/5680	
Plan (D)	Panampu	No. 0.0 ~ No. 3.0	2898.75	16.7	4.00	1/2550	3,221
	Jongaya	No. 0.0 ~ No. 3.0	9265.00	43.3	21.00	1/8150	

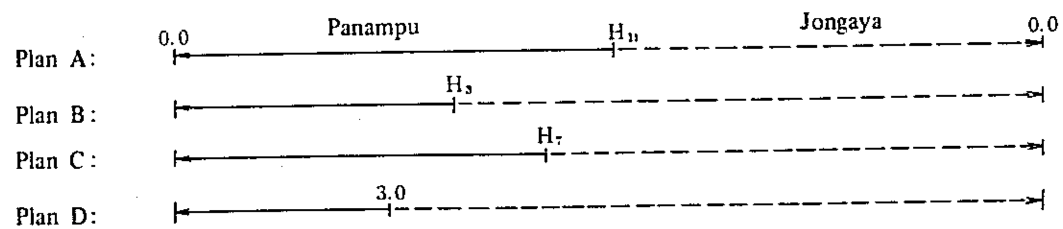


Table 3-8 FEATURES OF THE PROPOSED DRAINAGE CHANNELS (URGENT DRAINAGE SYSTEM IMPROVEMENT)

Route	Cross Section No.	Length (m)	Type of Cross Section	Discharge (m^3/s)	Channel Cross Section				Longitudinal Gradient
					Top(m)	Bottom(m)	Depth(m)	Slope Gradient	
Panampu	0 - 3.0	2,898	A	30.6	15.62	10.50	2.56	1:1	1/4,300
	3.0 - H3	2,003	B	14.0	10.12	5.00	2.56	1:1	
	sub-total	4,901.75							
Jongaya	0.0 - 2.4	3,018	Dredging	29.4	38.10	12.50	2.56	1:5	1/6,400
	2.4 - H18	1,139	C	29.4	17.62	12.50	2.56	1:1	
	H18 - H11	1,302	A	24.0	15.62	10.50	2.56	1:1	
	H11 - H3	1,803	D	9.0	9.12	4.00	2.56	1:1	
	sub-total	7,262							
Sinrijala	0.0 - 1.0	1,010	E	8.1	8.80	3.22	2.79	1:1	1/5,740
	1.0 - P1	1,566	F	6.9	8.34	3.22	2.56	1:1	
	sub-total	2,576							
	Total	14,739.75							

Table 3-9 DREDGING AND EXCAVATION VOLUME FOR URGENT PLAN

Channel	Section	Exca. Volume (m^3)
Panampu	No. 0.0 – No. 3.0	119,200
,,	No. 3.0 – No. H3	50,400
	Sub-total	169,600
Jongaya	No. 0.0 – No. 2.4 (Dredging only)	138,400
,,	No. 2.4 – No. H22	148,400
,,	No. H11– No. H3	51,200
	Sub-total	338,000
Sinrijala	No. 0.0 – No. 1.0	11,500
,,	No. 1.0 – No. P 1	18,500
	Sub-total	30,000
	Total	537,600
	Net Excavation Volume	399,200

Table 3-10 LAND AQUISITION AND HOUSE EVACUATION FOR URGENT PLAN

Panampu

Section	Land Aquisition				House Evacuation
	L (m)	B1(m)	B2(m)	A (m^2)	
No. 0.0 -- No. 3.0	2898.75	15.62	4.5	35780	140
No. 3.0 -- No. H3	2003.00	10.12	4.5	26790	80
Total				62570	220

Jongaya

Section	Land Aquisition				House Evacuation
	L (m)	B1(m)	B2(m)	A (m^2)	
No. 0.0 -- No. 2.4	3018.0	17.62	4.5	—	—
No. 2.4 -- No. H18	1139.0	17.62	4.5	25200	70
No. H18 -- No. H11	1302.0	15.62	4.5	26200	50
No. H11 -- No. H3	1803.0	9.12	4.5	24560	30
Total				75960	150

Sinrijala

Section	Land Aquisition				House Evacuation
	L (m)	B1(m)	B2(m)	A (m^2)	
No. 0.0 -- No. 1.0	1010.0	9.10	4.5	5270	0
No. 1.0 -- No. P1	1316.0	8.34	4.5	7920	0
Total				13790	0

L : Channel Length B1: Channel Width

B2: Width of Maintenance Road A : Land Aquisition Area

Table 3-11 MAJOR FEATURES OF THE PROPOSED SLUICE

ITEM	FEATURES
Height, inside	2.56 m
Width, inside	2.90 m
Type of sluice	double - channel
Length	19.60 m
Cross sectional area	7.42 m^2
Coefficient of roughness	0.016
Bed gradient	1/5740
Flow capacity	7.0 m^3/s
Gate type	steel manual operation gate
Gate height	2.66 m
Gate width	3.10 m
Number of gate	2 gates
Width of Jl. Panakkukang	18.00 m

Table 3-12 MAJOR DIMENSIONS OF THE PROPOSED BRIDGES

Pampang

No.	Existing Bridge		New Bridge	
	Length (m)	Width (m)	Length (m)	Width (m)
1	10.4	4.0	16.6	4.0
2	4.1	5.0	16.6	5.0
3	7.8	12.4	16.6	12.4
4	6.7	7.2	16.6	7.2
5	6.4	14.3	16.6	14.3
6	8.4	21.7	16.6	21.7
7	4.8	2.6	11.1	2.6
8	1.0	17.1	11.1	17.1
9	-	6.0	11.1	6.0
10	3.7	9.0	11.1	9.0

Jongaya

No.	Existing Road		New Bridge	
	Length (m)	Width (m)	Length (m)	Width (m)
11	–	3.0	10.1	3.0
12	–	3.4	10.1	3.4
13	–	14.3	10.1	14.3
14	–	9.1	16.6	9.1
15	–	6.9	16.6	6.9
16	–	8.2	18.6	8.2
17	–	8.5	18.6	8.5
18	–	11.3	18.6	11.3
19	–	5.9	18.6	5.9
20	–	16.1	18.6	16.1

Sinrijala

No.	Existing Bridge		New Bridge	
	Length (m)	Width (m)	Length (m)	Width (m)
21	6.8	18.0	Sluice	
22	8.0	1.0	9.3	1.0
23	19.3	1.0	9.3	1.0

Fig. 1-1 PRESENT DRAINAGE BASIN AND DRAINAGE CHANNEL

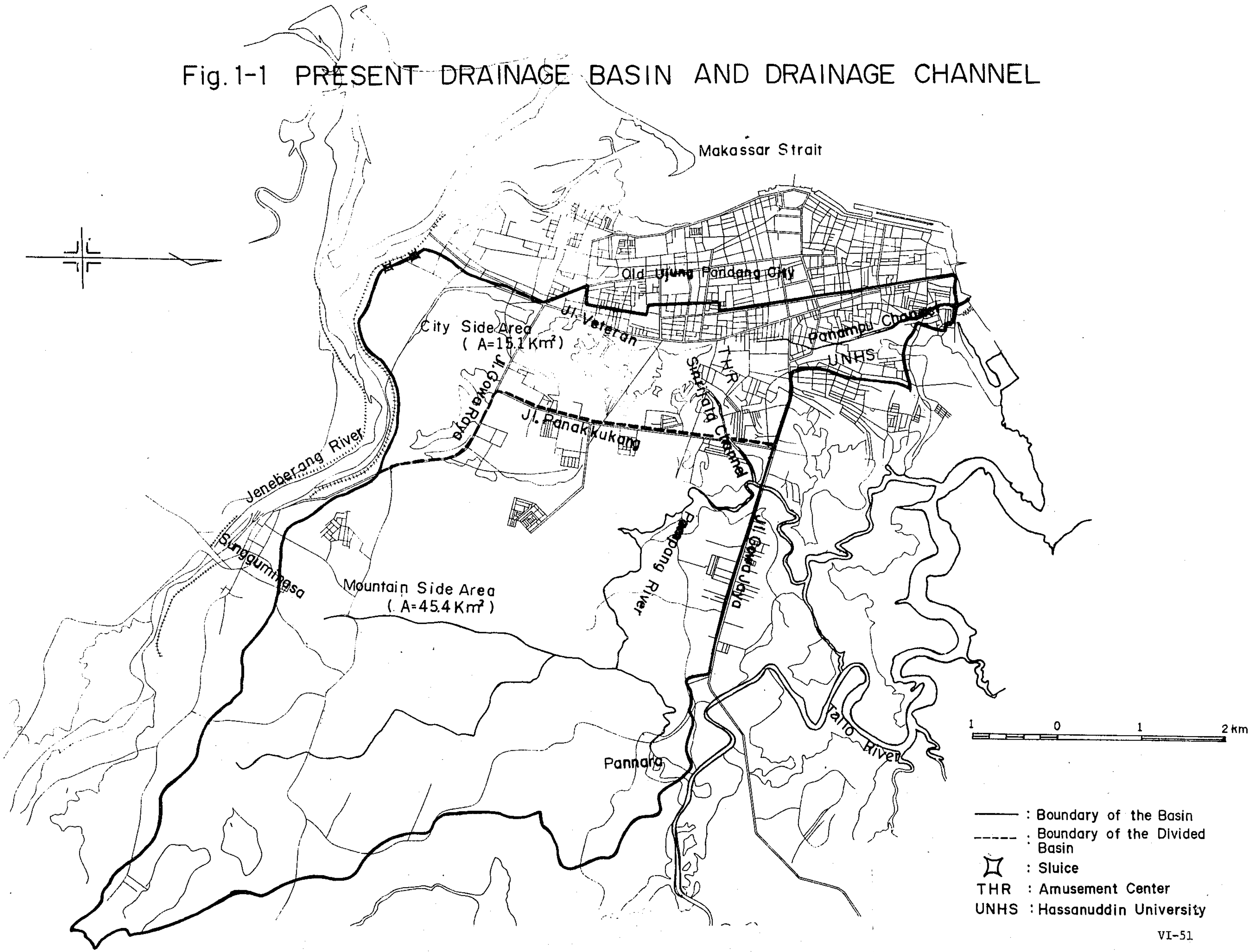
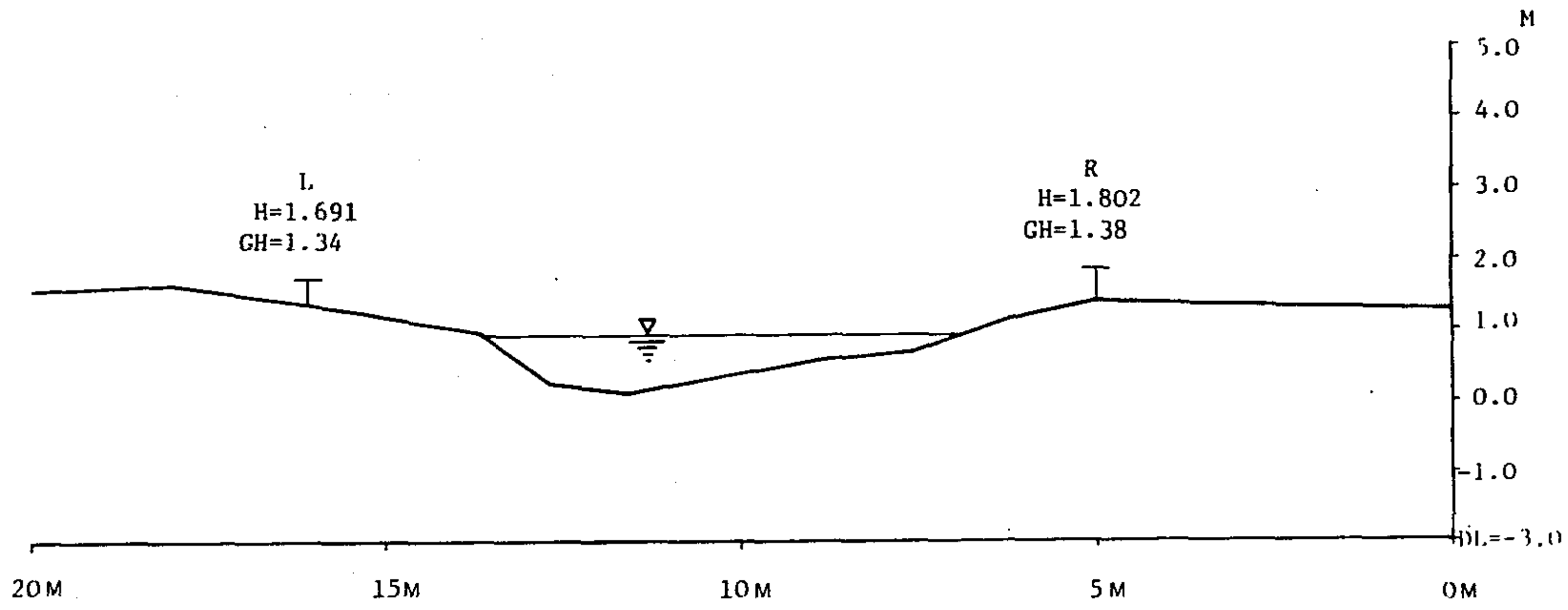


Fig. 1-2 (1) CROSS-SECTION OF EXISTING CHANNEL

---- Panampu (No. 2.5 k) -----



$H=1.30m$ $A=9.1m^2$ $S=13.2m$ $R=0.69m$ $i=1/4000$ $n=0.033$

$$V=\frac{1}{0.033} \times (0.69)^{2/3} \times (1/4000)^{1/2} = 0.374m^3/s$$

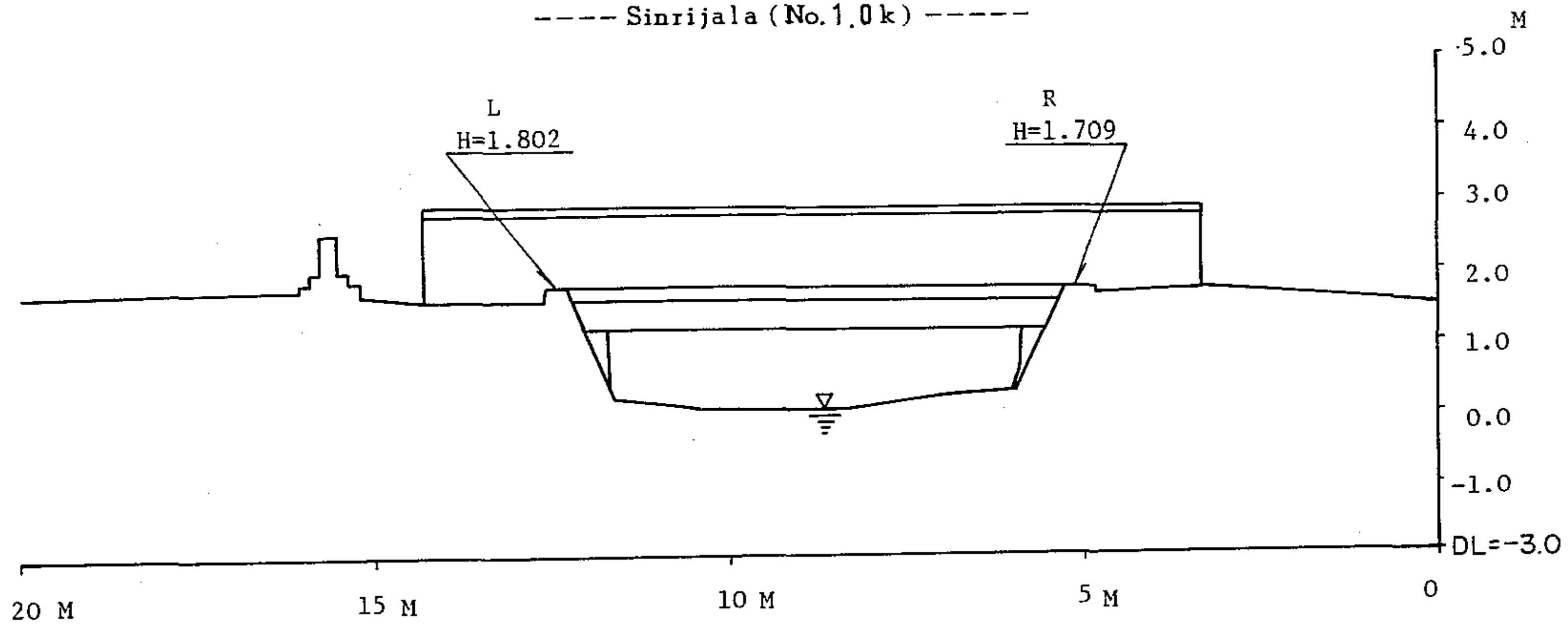
$$Q= 9.1 \times 0.374 = 3.4m^3/s$$

* H : Water Stage, A : Flow Area, R : Hydraulic Depth (A/S), i = Longitudinal Graident

n : *Manning's* Roughness Coefficient, V : Velocity, Q : Dischange.

Fig. 1-2 (2) CROSS-SECTION OF EXISTING CHANNEL

---- Sinrijala (No.1.0k) -----



H=1.2m $A=8.2m^2$ S=7.7m R=1.1m i=1/5750 n=0.033

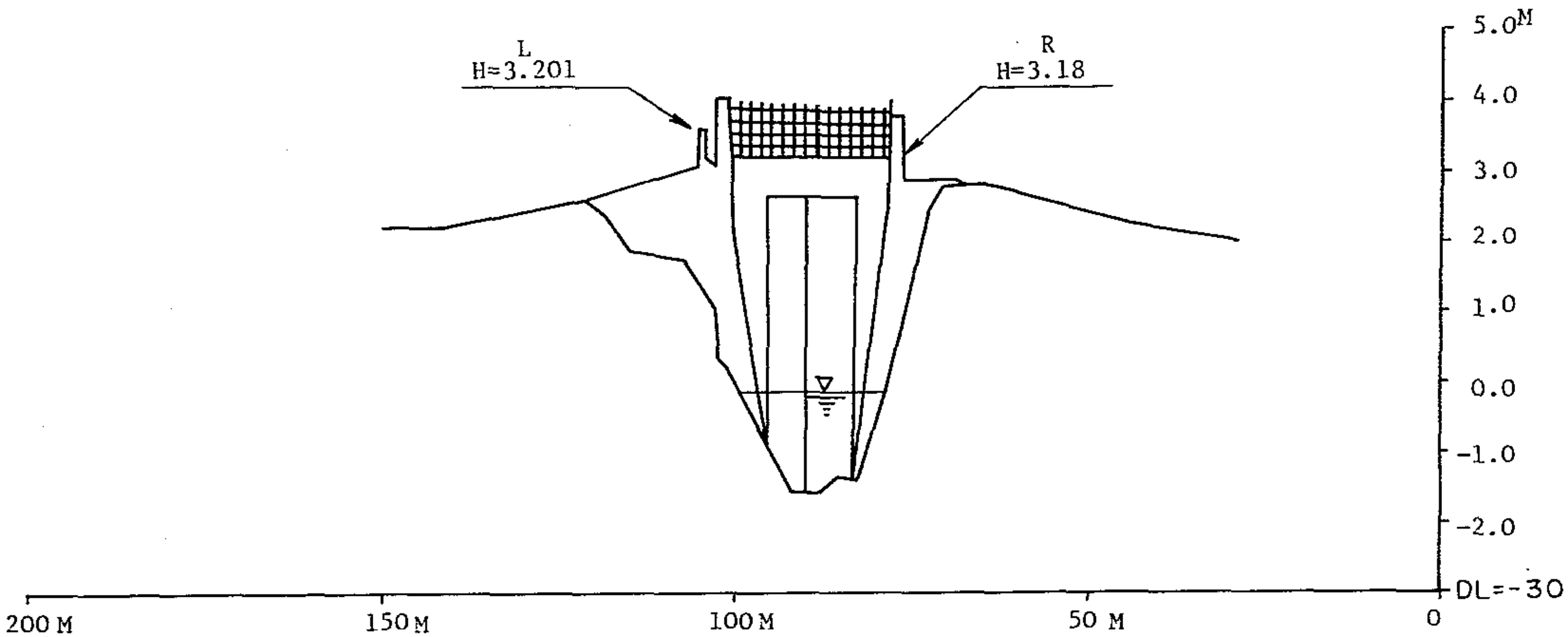
$$V= \frac{1}{0.033} \times (1.1)^{2/3} \times (1/5750)^{1/2} = 0.426m\ /s$$

$$Q= 8.2 \times 0.426 = 3.49m^3/s$$

* H: Water Stage, A: Flow Area, R: Hydraulic Depth (A/S), i: Longitudinal Graident
n: Manning's Roughness Coefficient, V: Velocity, Q: Discharge.

Fig. 1-2 (3) CROSS-SECTION OF EXISTING CHANNEL

---- Pampang (No.T2-3.0) -----



H=2.0m $A=41.3m^2$ S=16.6m R=2.48m i=1/10,000 n=0.033

$$V=\frac{1}{0.033} \times (2.48)^{2/3} \times (1/10,000)^{1/2} = 0.555 m/s$$

$$Q=41.3 \times 0.555 = 22.9 m^3/s$$

* H: Water Stage, A: Flow Area, R: Hydraulic Depth (A/S), I: Longitudinal Graident
n: Manning's Roughness Coefficient, V: Velocity, Q: Discharge.

Fig. I-3 INUNDATION MAP OF THE FLOOD OF JAN. 1976

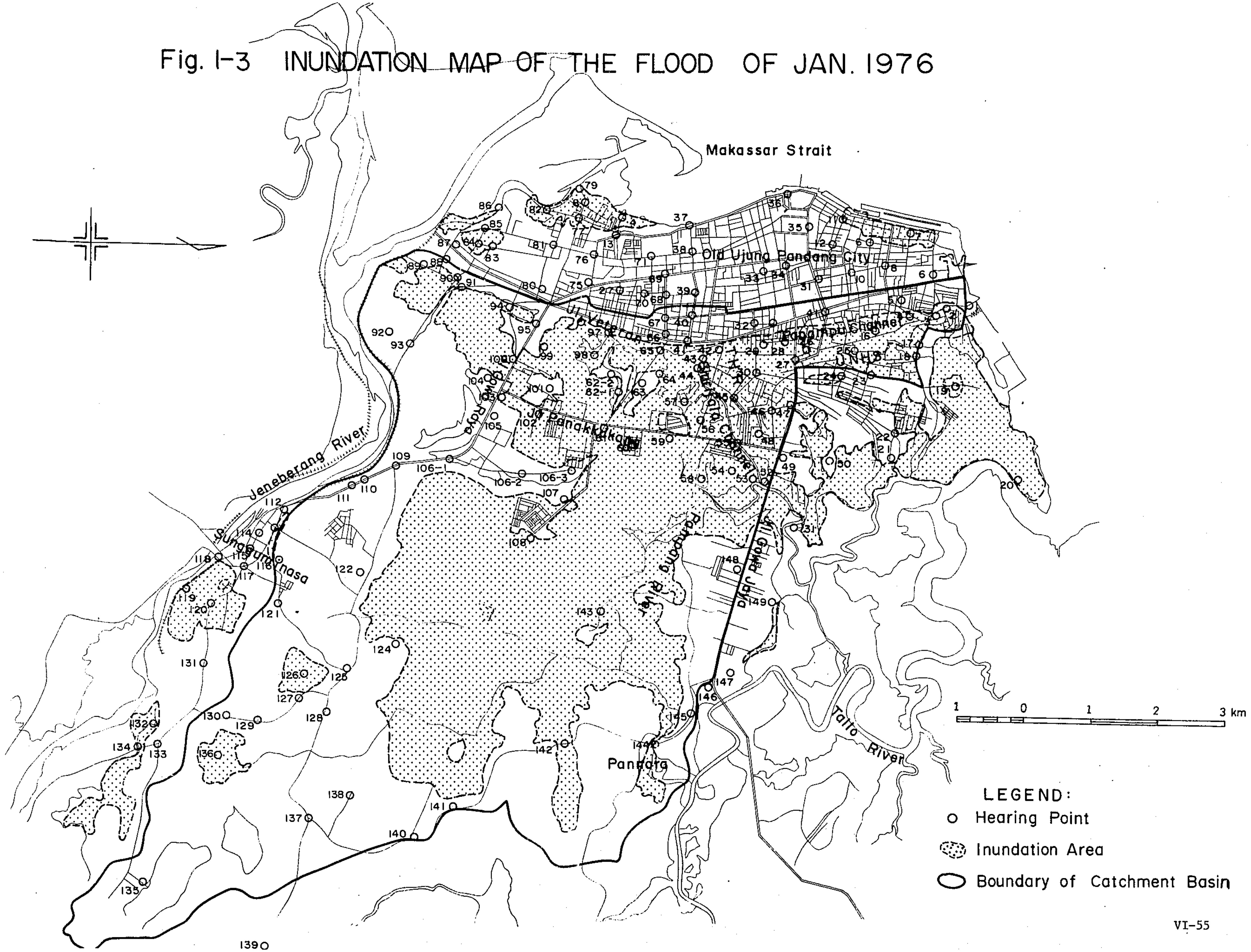


Fig.2-1 POSSIBLE DRAINAGE SYSTEM FOR OVERALL PLAN

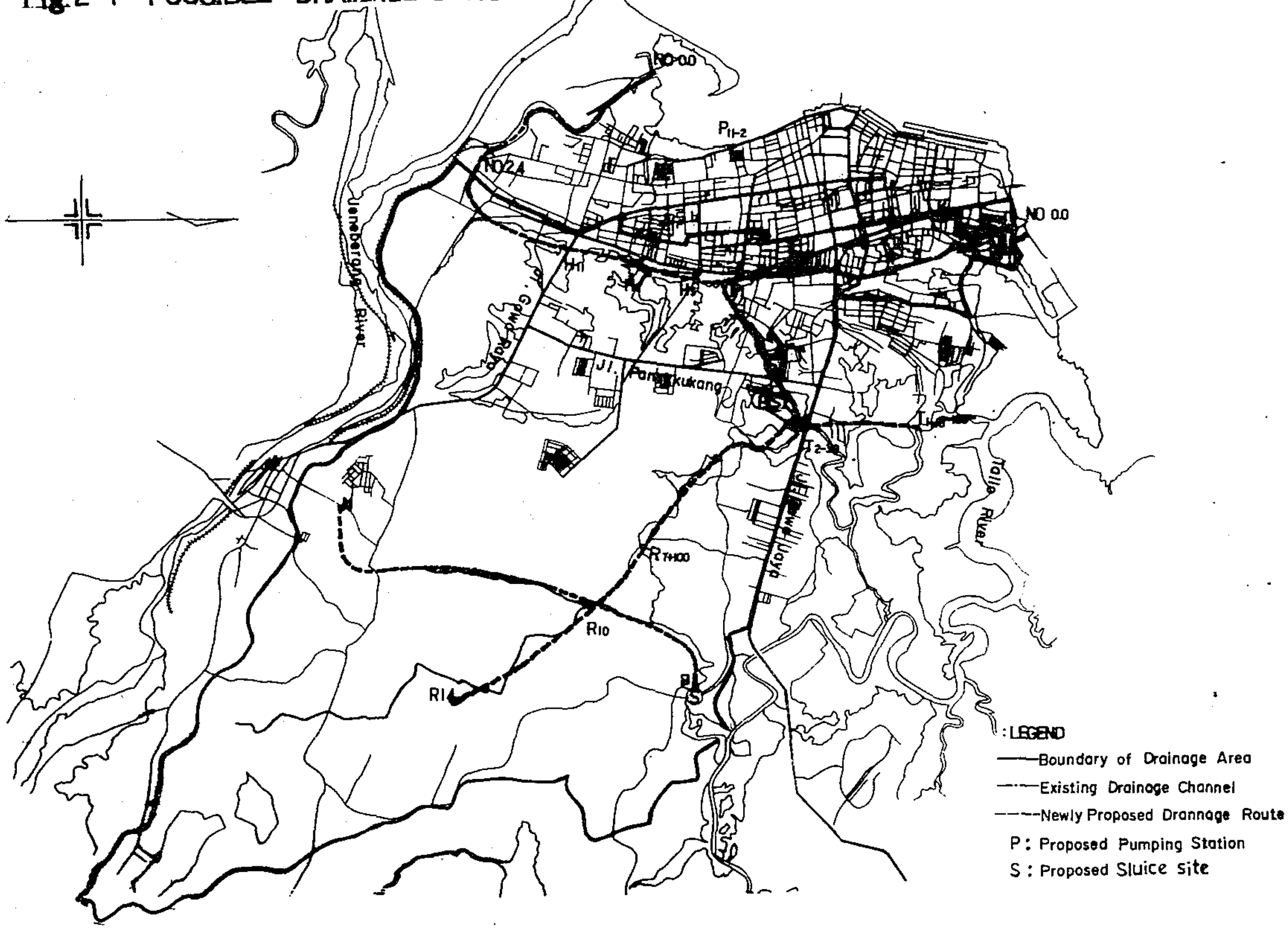


Fig.2-2 PROCEDURE FOR ECONOMIC EVALUATION OF POSSIBLE DRAINAGE SYSTEM

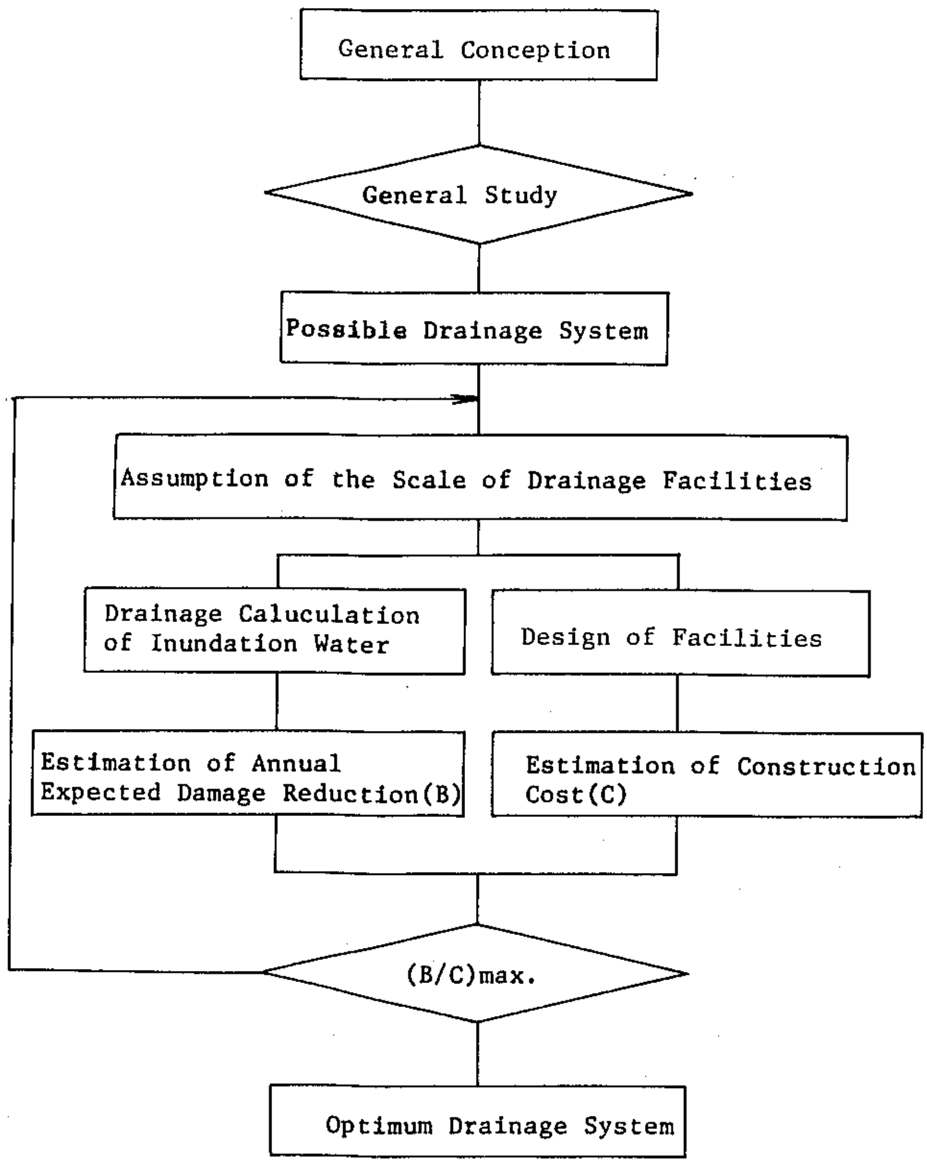


Fig.2-3 CALCULATION MODEL FOR OVERALL PLAN

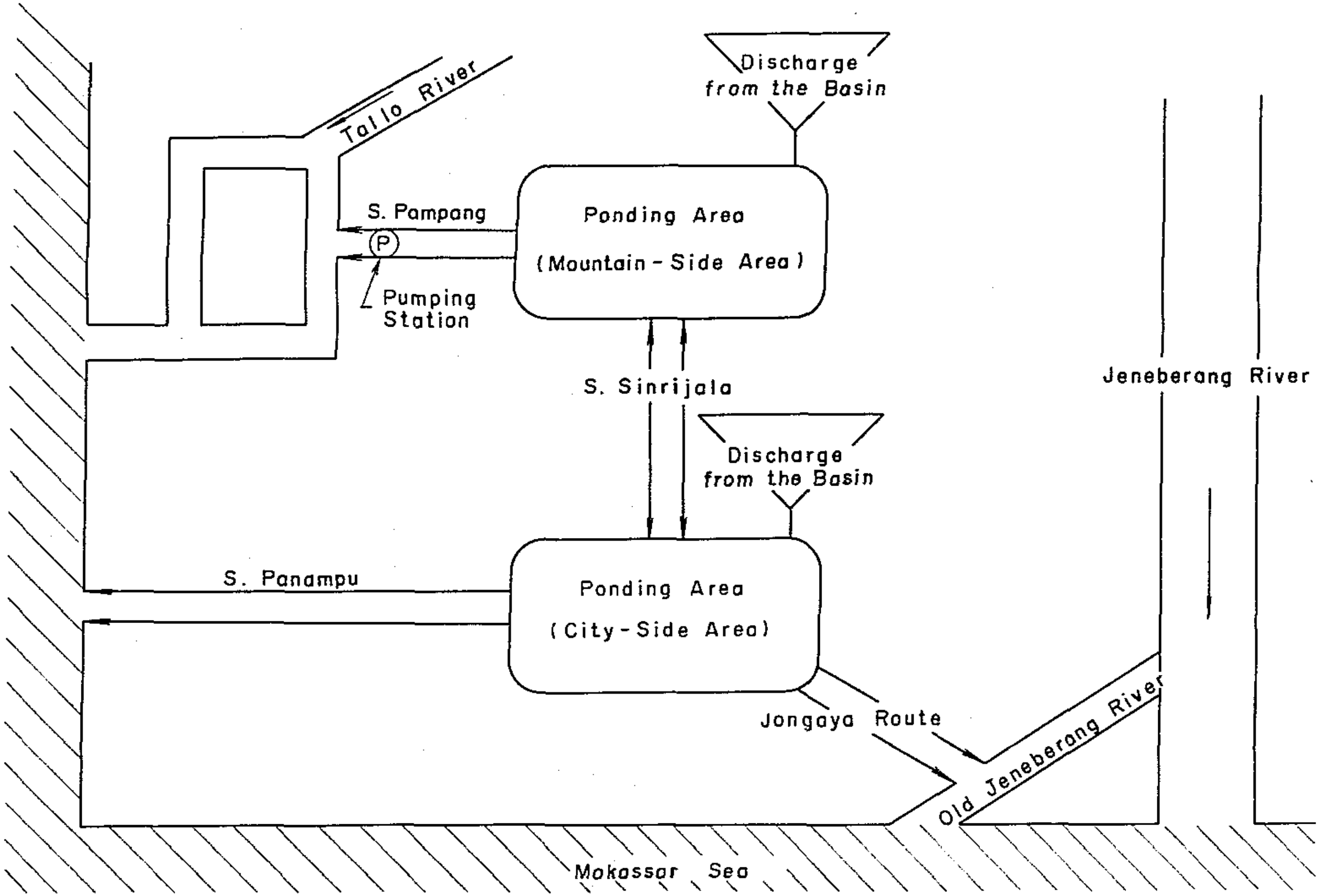


Fig.2-4 DAILY RAINFALL DISTRIBUTION

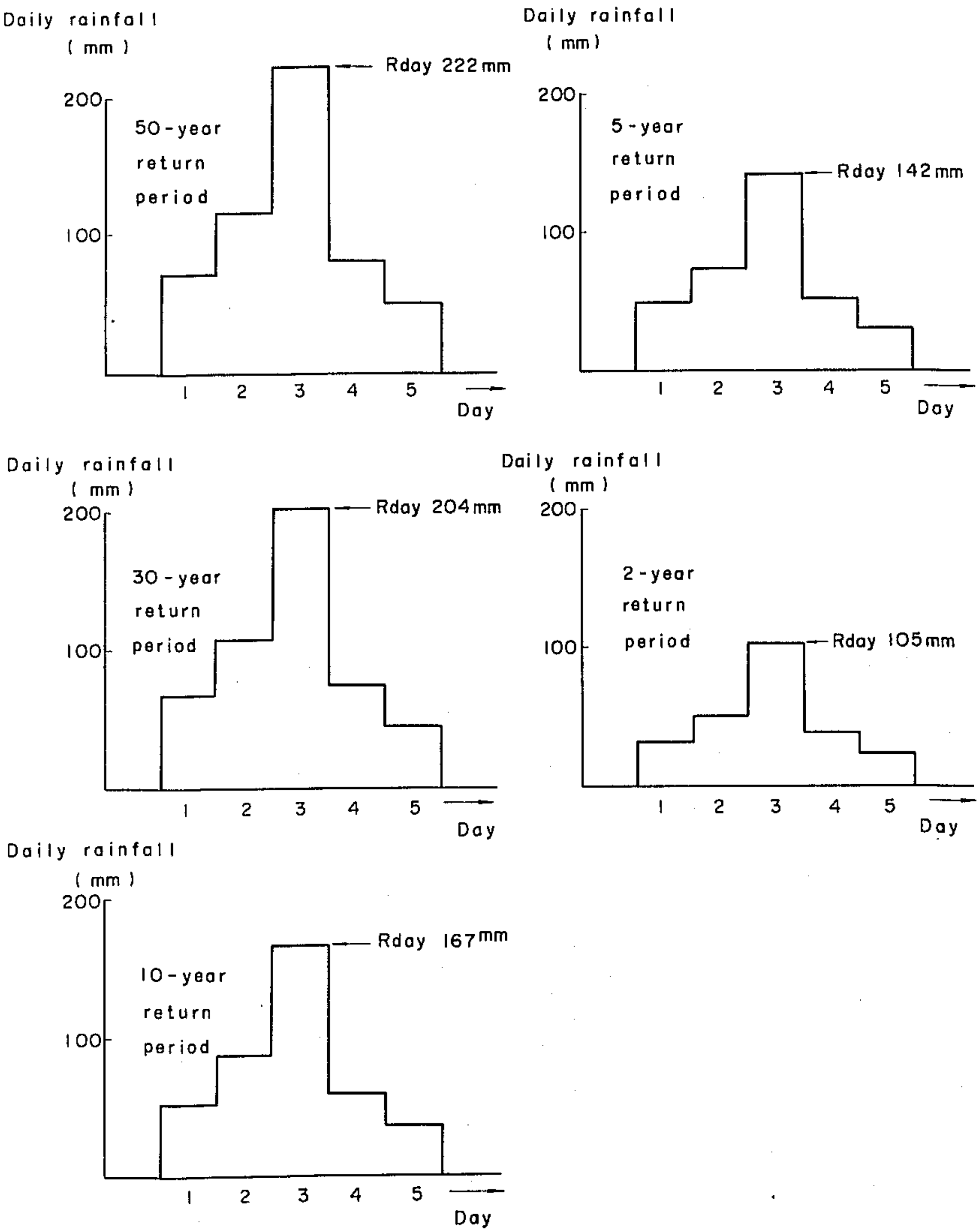


Fig.2-5 OUTLET WATER STAGE FOR OVERALL PLAN

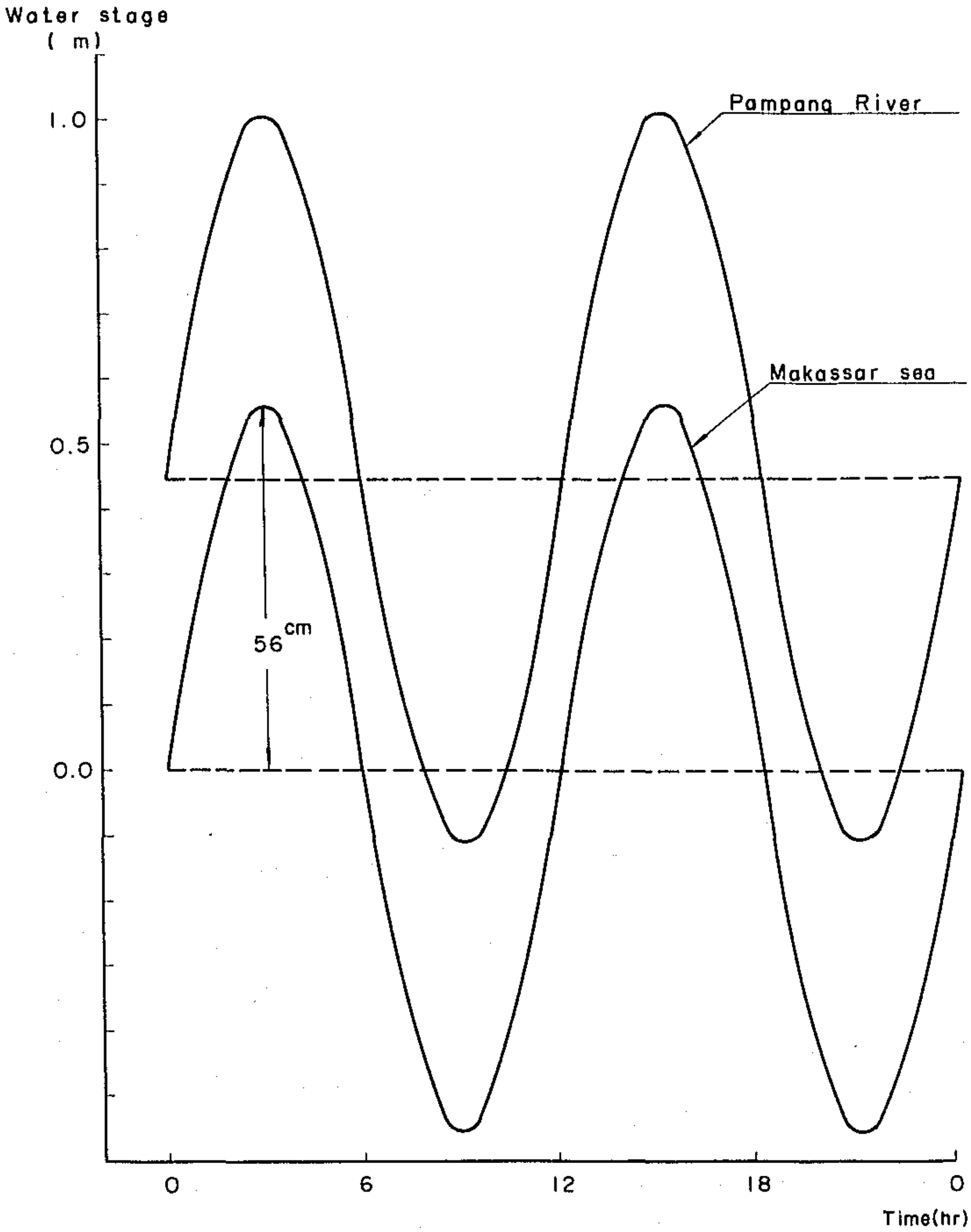


Fig. 2 - 6 CONSTRUCTION COST BY CHANNEL CAPACITY FOR OVERALL PLAN

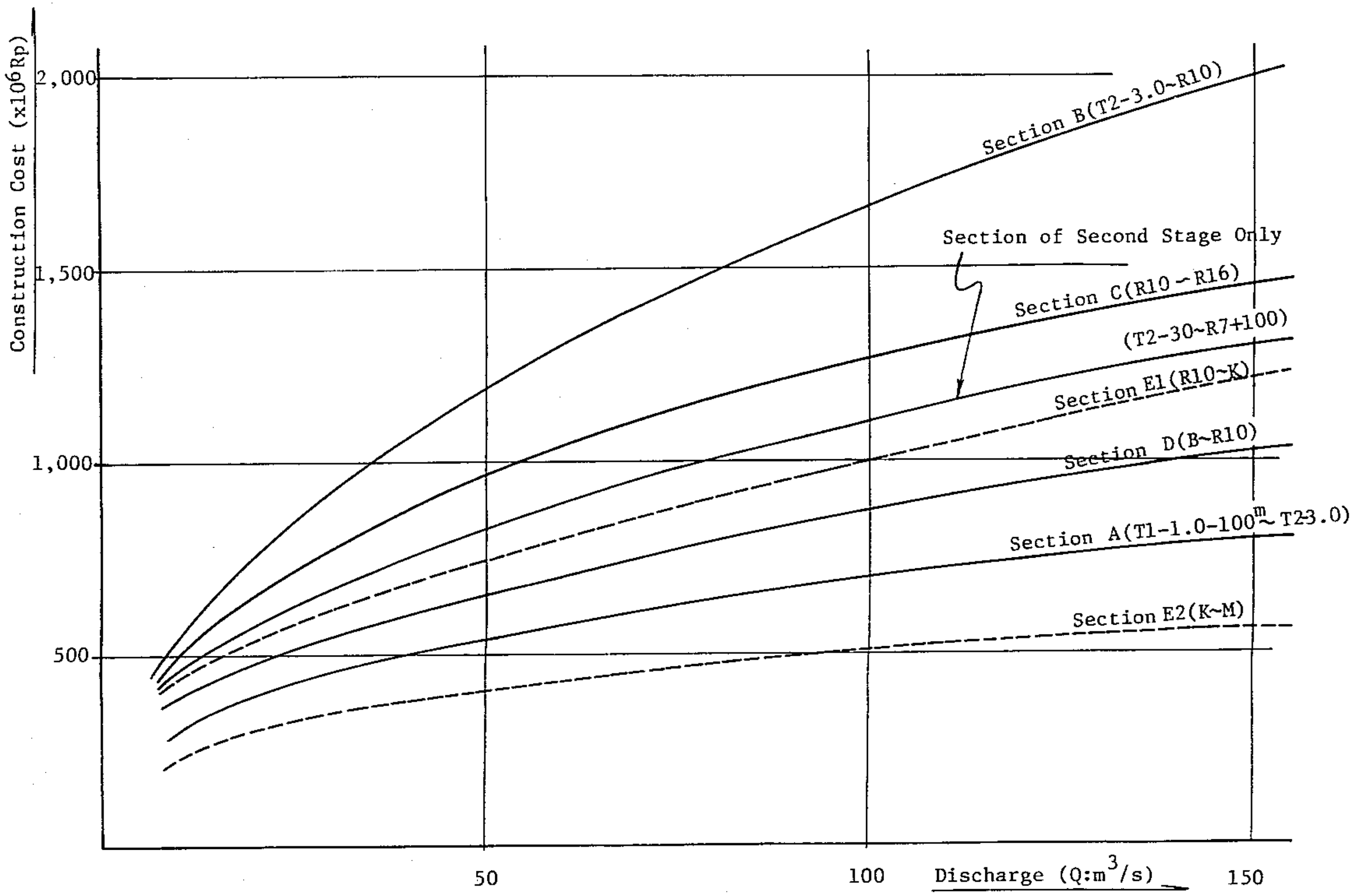


Fig.2-7 CONSTRUCTION COST BY PUMP CAPACITY FOR OVERALL PLAN

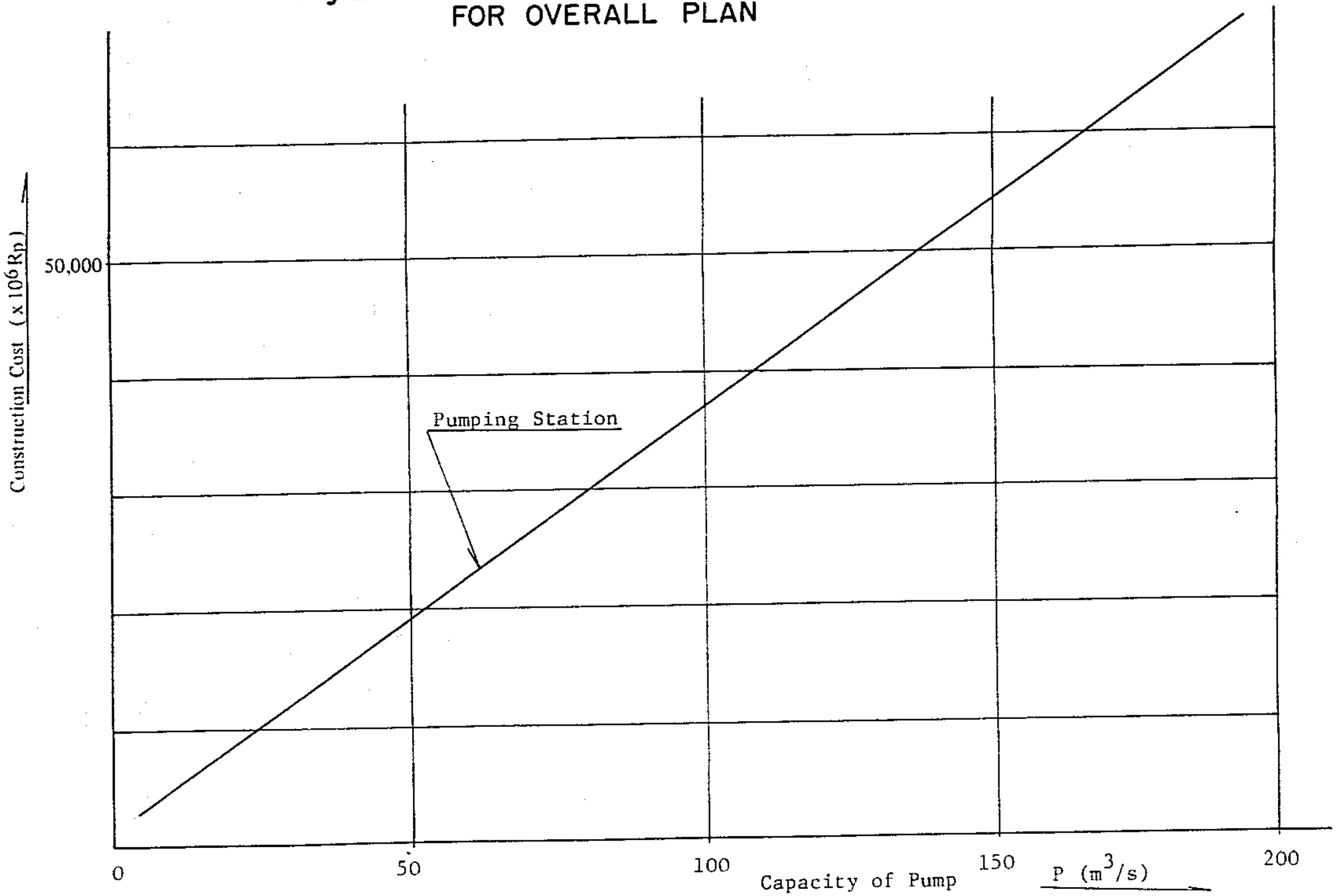
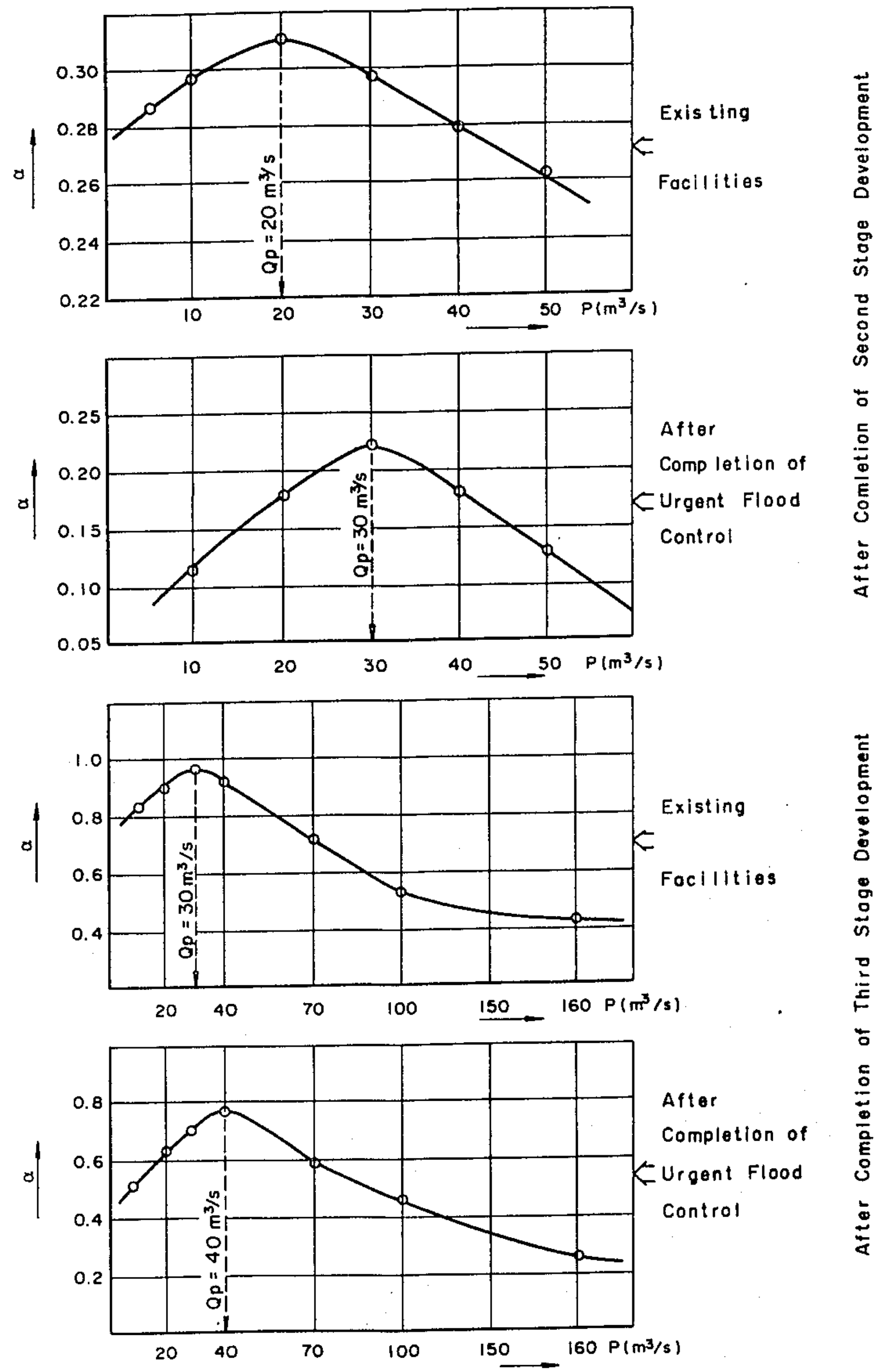


Fig. 2-8 ECONOMICAL JUSTIFICATION OF DRAINAGE SYSTEM FOR OVERALL PLAN



*α : Ratio between annual flood damage reduction and construction cost

P : Capacity of pump

Fig. 2-9 LOCATION OF OPTIMUM DRAINAGE SYSTEM FOR OVERALL PLAN

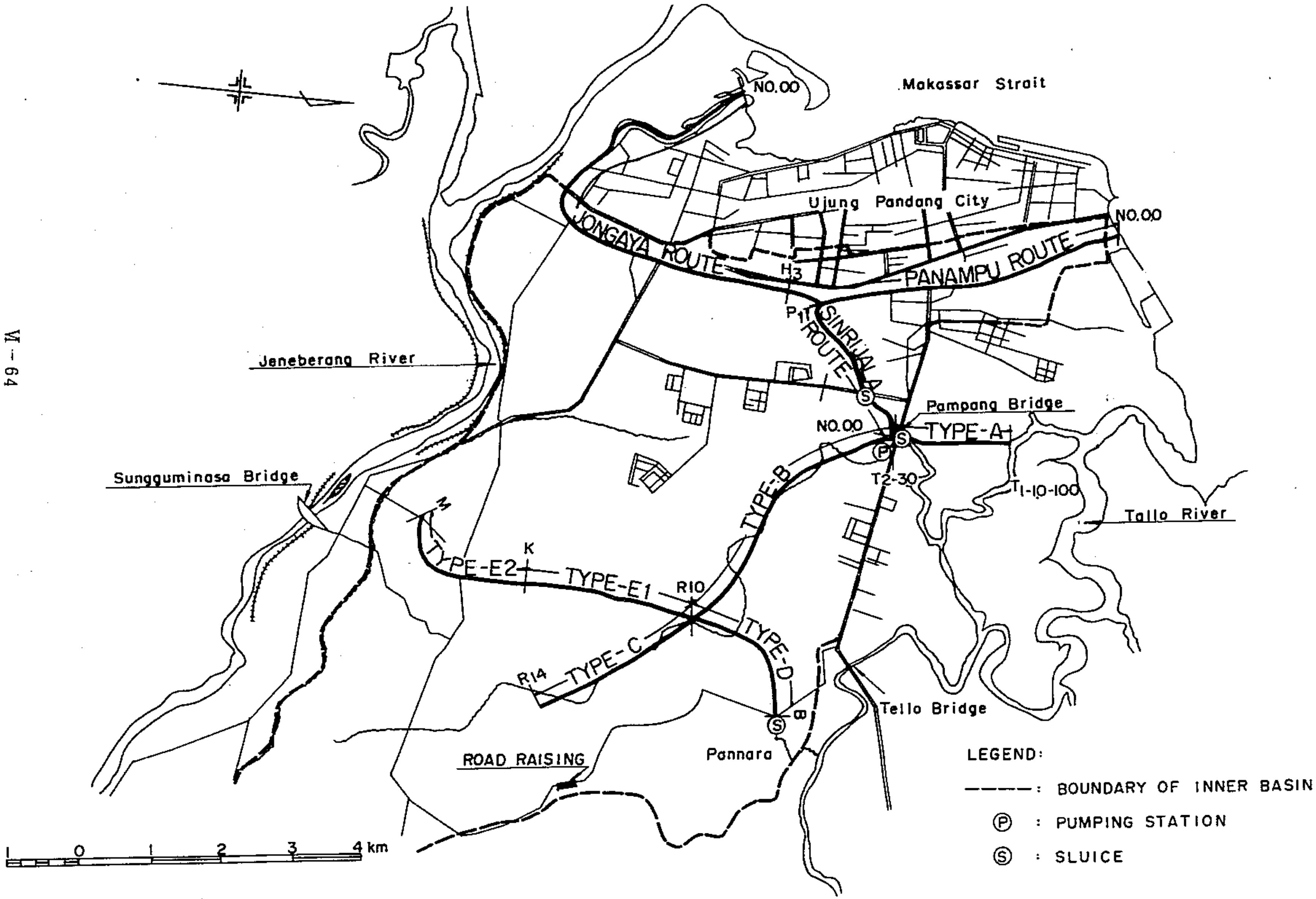
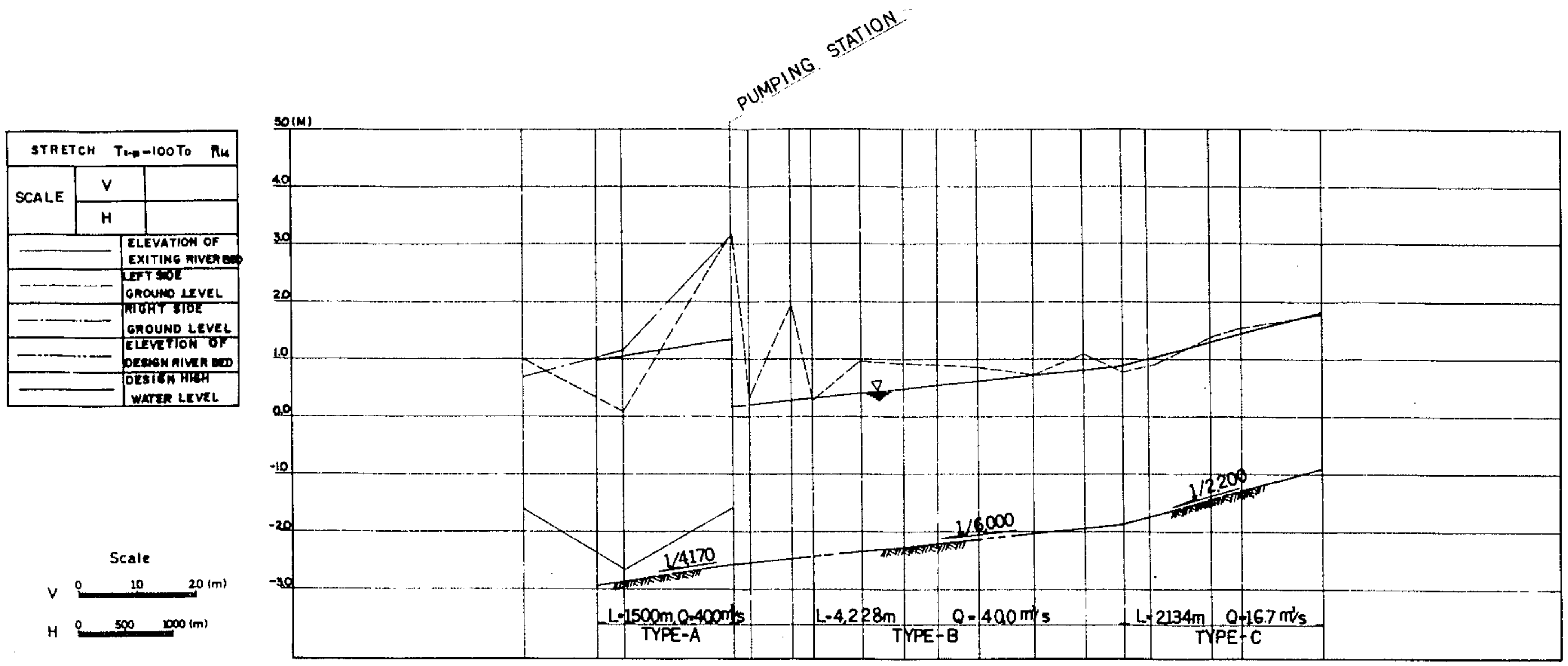
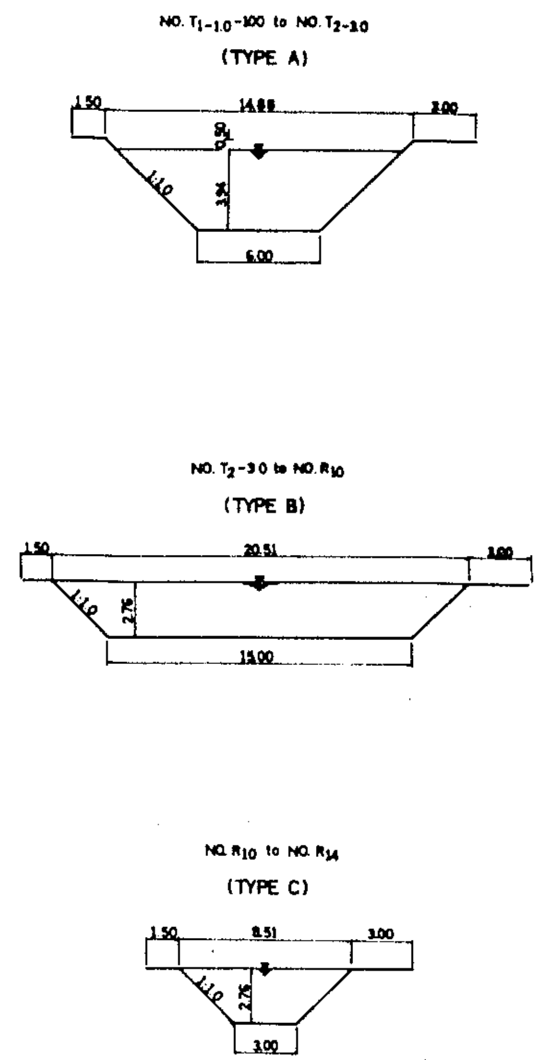


Fig. 2-10 (1) LONGITUDINAL PROFILE OF OPTIMUM DRAINAGE CHANNEL FOR OVERALL PLAN
PAMPANG NO. T1.0-100 – No. R14





		STATION NO.	T_2-1.0	T_1-0-100	T_2-2.0	T_2-3.0	R.1	R.2	R.3	R.4	R.5	R.6	R.7	R.8	R.9	R.10	R.11	R.12	R.13	R.14
DESIGN		DESIGN HIGH WATER LEVEL		1.000	1.075	1.380	0.174 0.207	0.283	0.324	0.411	0.482	0.543	0.620	0.721	0.810	0.880	1.035	1.321	1.463	1.850
		ELEVATION DESIGN RIVER BED		-2.940	-2.855	-2.580	-2.548	-2.472	-2.431	-2.344	-2.273	-2.212	-2.138	-2.054	-1.945	-1.875	-1.720	-1.434	-1.292	-0.908
EXISTING CONDITION		RIGHT SIDE GROUND LEVEL	0.66		1.12	3.18														
		LEFT SIDE GROUND LEVEL	1.00		0.08	3.20	0.36	1.90	0.42	0.99	0.93	0.92	0.88	0.76	1.08	0.79	0.94	1.39	1.50	1.79
		ELEVATION OF EXISTING RIVER BED	-1.61		-2.67	1.58														
DISTANCE		ACCUMULATIVE DISTANCE	0.00	784.00	1076.00	2264.00	2454.00	2914.00	3156.00	3682.00	4109.00	4473.00	4936.00	5541.00	6083.00	6492.00	6823.00	7463.00	7774.00	8628.00
		DITANCE	0.00	784.00	312.00	1180.00	190.00	460.00	242.00	526.00	427.00	364.00	463.00	605.00	542.00	409.00	331.00	640.00	311.00	853.00

Fig. 2 - 10 (2) LONGITUDINAL PROFILE OF OPTIMUM DRAINAGE CHANNEL FOR OVERALL PLAN
NO. B - NO. R10 - NO. M

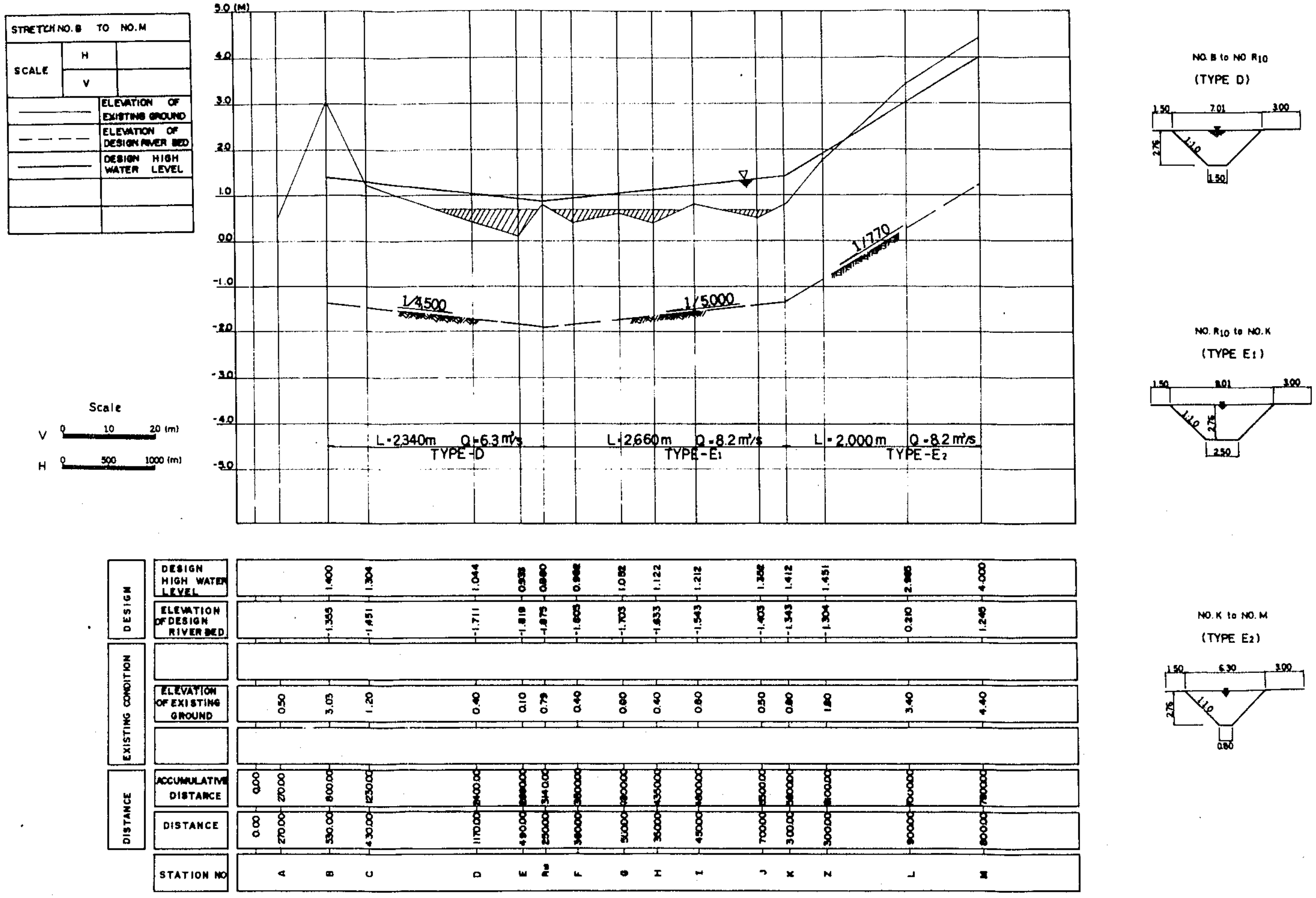
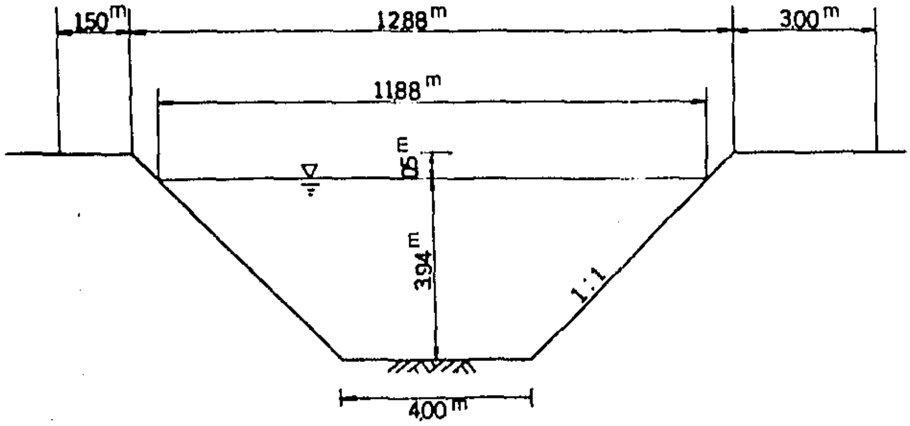


Fig. 2-11 (1) STANDARD CROSS SECTION OF OPTIMUM DRAINAGE CHANNEL FOR OVERALL PLAN

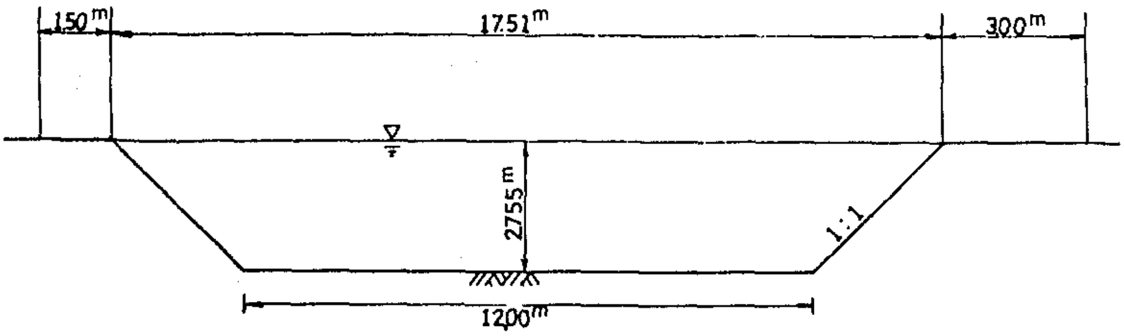
— Second Stage —

Section A : (T1-10 − 100^m ~ T2-30 : L=1500 m)



$$A = \frac{1}{2}(4+4+3.94+3.94)\times 3.94 = 31.3$$
$$P = 4+2\sqrt{3.94^2+3.94^2} = 15.14$$
$$R = 2.06$$
$$V = \frac{1}{0.025}\times(2.06)^{2/3}\times(1/4170)^{1/2} = 1.00$$
$$Q = 31.3 > 30.0$$

Section B : (T2-30 ~ R7+100^m : L=2772 m)



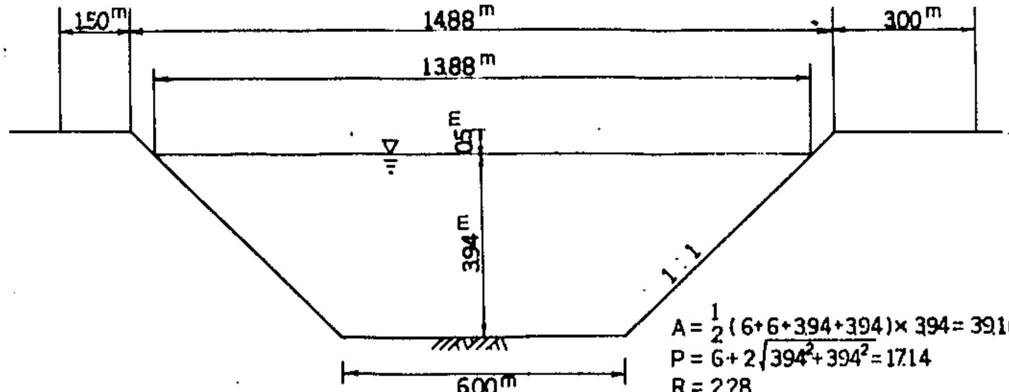
$$A = \frac{1}{2}(12+12+2.755+2.755)\times 2.755 = 40.6$$
$$P = 12+2\sqrt{2.755^2+2.755^2} = 17.79$$
$$R = 2.05$$
$$V = \frac{1}{0.025}\times(2.05)^{2/3}\times(1/6000)^{1/2} = 0.83$$
$$Q = 33.7 > 30.0$$

* A : Flow Section Area, R : Hydraulic Depth(A/P) , V : Velocity (m/s), Q : Discharge(m^3/s)

Fig. 2-11 (2) STANDARD CROSS SECTION OF OPTIMUM DRAINAGE CHANNEL FOR OVERALL PLAN

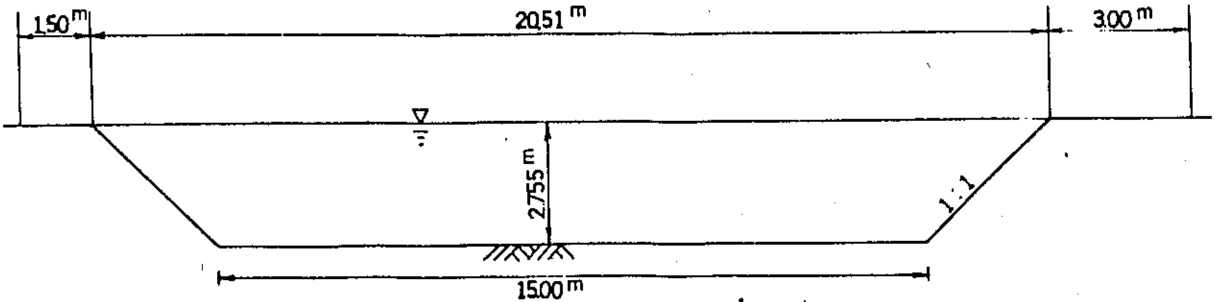
Third Stage

Section A ($T_{1-10-100^m}$ ~ T_{2-30} : L = 1500 m)



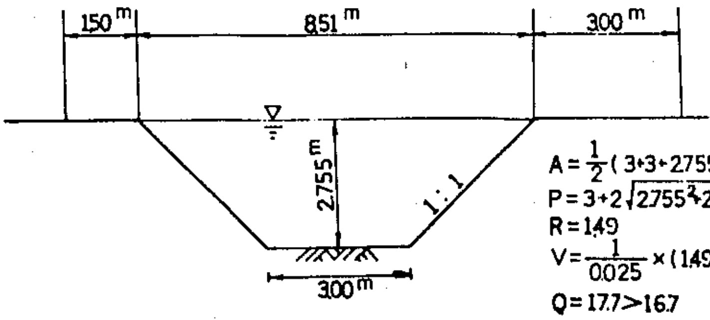
$A = \frac{1}{2}(6+6+3.94+3.94)\times 3.94 = 39.16$
$P = 6+2\sqrt{3.94^2+3.94^2} = 17.14$
$R = 2.28$
$V = \frac{1}{0.025}\times(2.28)^{2/3}\times(1/4170)^{1/2} = 1.07$
$Q = 42.1 > 40.0$

Section B (T_{2-30} ~ R_{10} : L = 4228 m)



$A = \frac{1}{2}(15+15+2.755+2.755)\times 2.755 = 48.9$
$P = 15+2\sqrt{2.755^2+2.755^2} = 22.79$
$R = 2.15$
$V = \frac{1}{0.025}\times(2.15)^{2/3}\times(1/6000)^{1/2} = 0.86$
$Q = 41.96 > 40.0$

Section C (R_{10} ~ R_{16} : L = 4133 m)



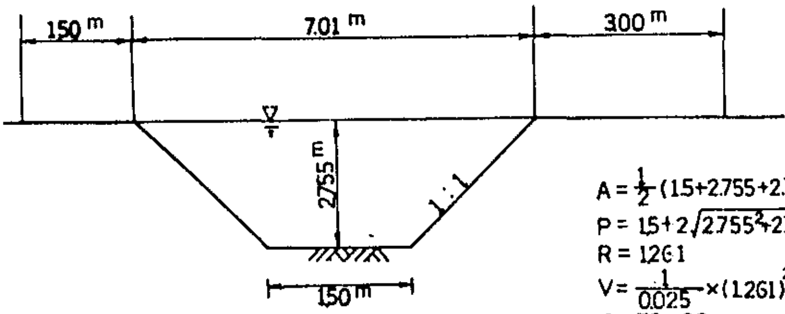
$A = \frac{1}{2}(3+3+2.755+2.755)\times 2.755 = 16.0$
$P = 3+2\sqrt{2.755^2+2.755^2} = 10.79$
$R = 1.49$
$V = \frac{1}{0.025}\times(1.49)^{2/3}\times(1.49)^{1/2} = 1.1$
$Q = 17.7 > 16.7$

* A : Flow Section Area , R : Hydraulic Depth(A/P) , V : Velocity (m/s), Q : Discharge (m^3/s)

Fig. 2−11 (3) STANDARD CROSS SECTION OF OPTIMUM DRAINAGE CHANNEL FOR OVERALL PLAN

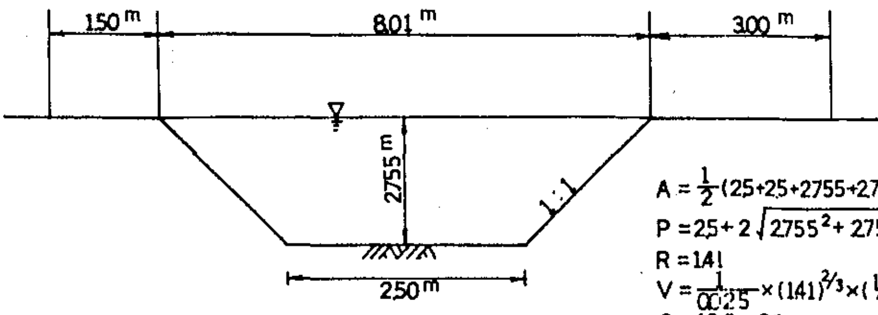
Third Stage

Section D (B～R10 : L= 2340 m)



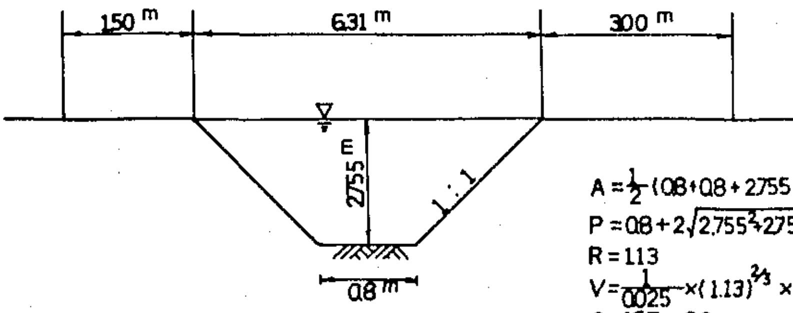
$A = \frac{1}{2}(1.5+2.755+2.755)\times 2.755 = 11.72$
$P = 1.5+2\sqrt{2.755^2+2.755^2} = 9.292$
$R = 1.261$
$V = \frac{1}{0.025}\times(1.261)^{2/3}\times(1/4500) = 0.65$
$Q = 7.6 > 6.3$

Section E1 (R10～K : L=2660 m)



$A = \frac{1}{2}(2.5+2.5+2.755+2.755)\times 2.755 = 14.5$
$P = 2.5+2\sqrt{2.755^2+2.755^2} = 10.29$
$R = 1.41$
$V = \frac{1}{0.025}\times(1.41)^{2/3}\times(1/5000)^{1/2} = 0.7$
$Q = 10.2 > 8.2$

Section E2 (K～M : L = 2000 m)

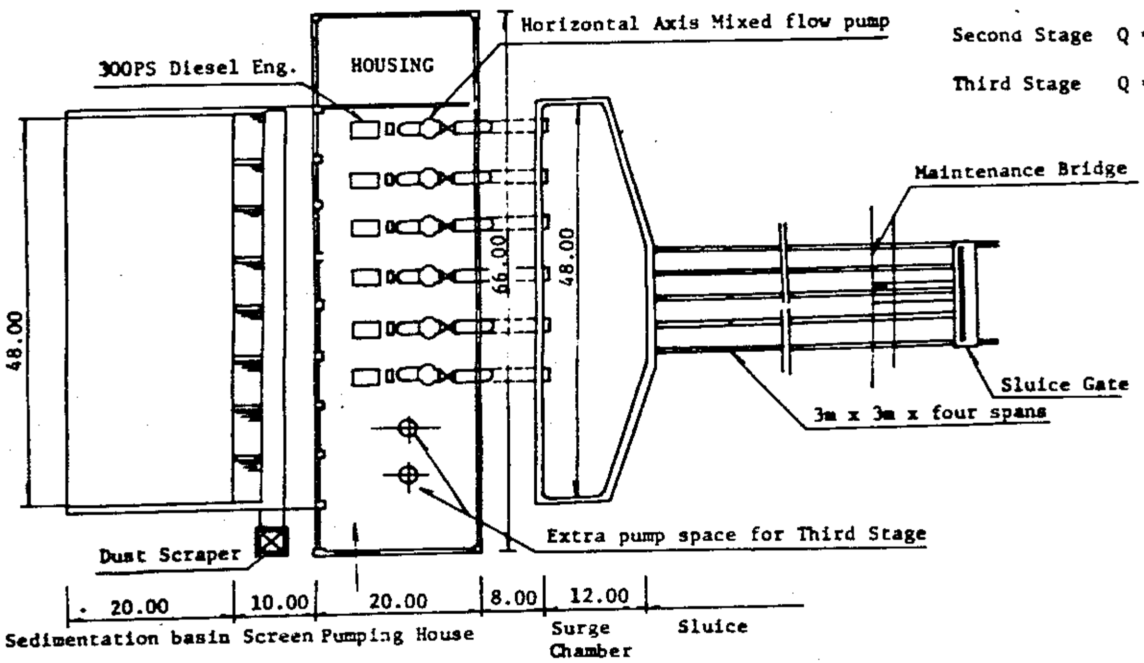


$A = \frac{1}{2}(0.8+0.8+2.755+2.755)\times 2.755 = 9.8$
$P = 0.8+2\sqrt{2.755^2+2.755^2} = 8.592$
$R = 1.13$
$V = \frac{1}{0.025}\times(1.13)^{2/3}\times(1/770)^{1/2} = 1.1$
$Q = 10.7 > 8.2$

* A : Flow Section Area, R : Hydraulic Depth (A/p), V : Velocity (m/s), Q : Discharge (m^3/s)

Fig. 2-12 PUMPING STATION FOR OVERALL PLAN

PLAN

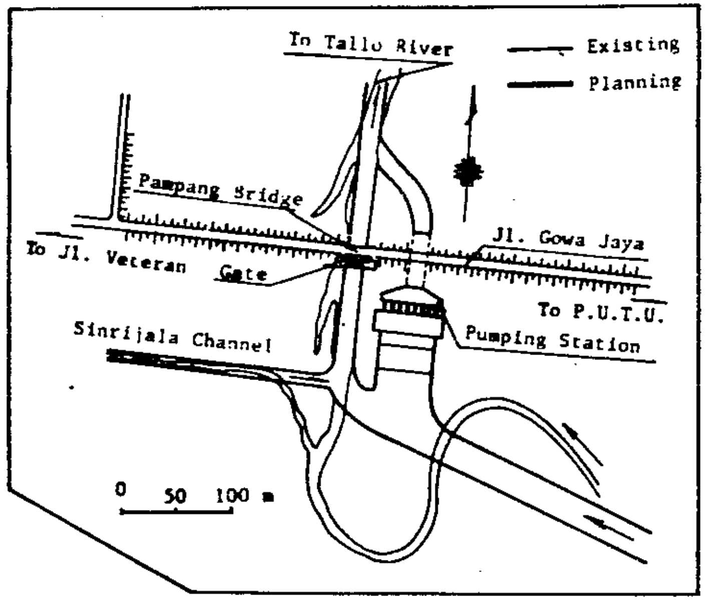


Unit : m

Second Stage $Q = 30\ m^3/s$

Third Stage $Q = 40\ m^3/s$

LOCATION



LONGITUDINAL SECTION

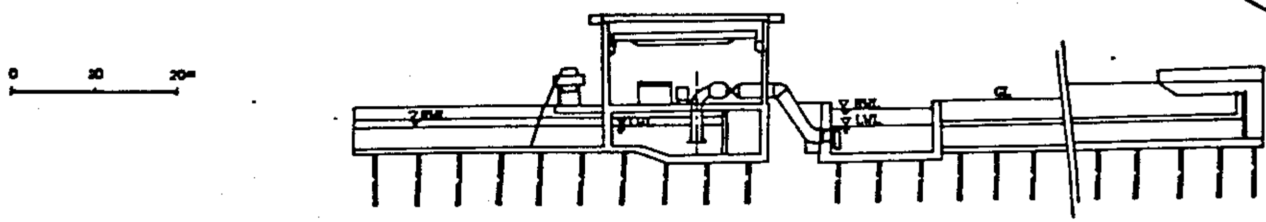
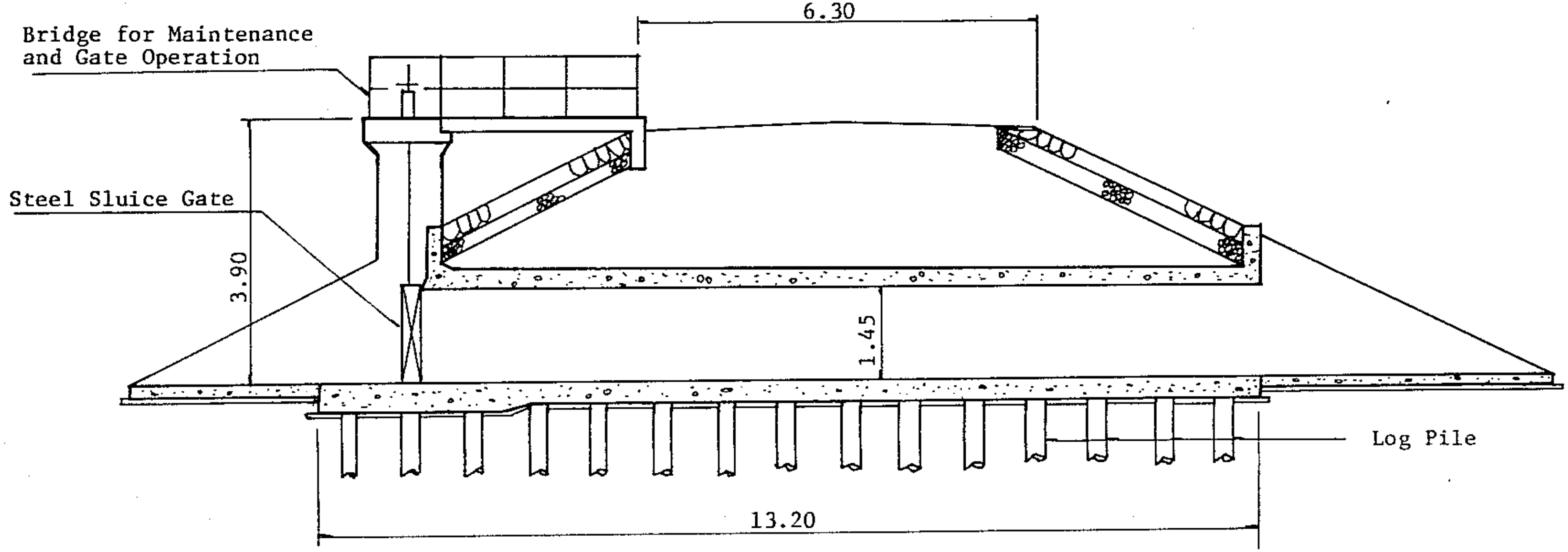


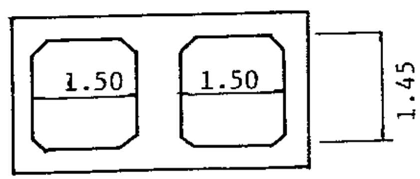
Fig. 2-13 PROPOSED SLUICE AT PANNARA

LONGITUDINAL PROFILE

Unit : m



CROSS SECTION

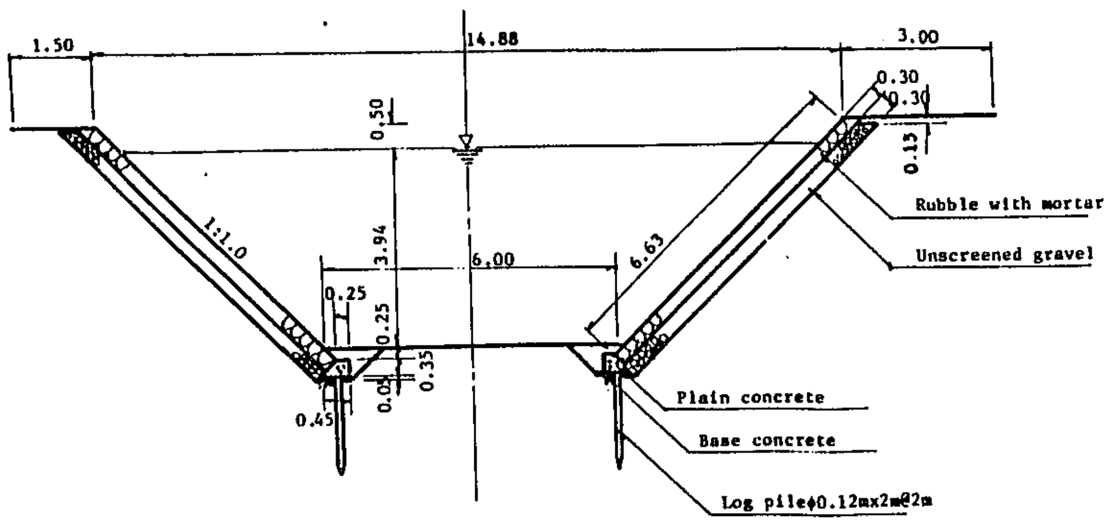


Site	Gate Height	Gate Width	Number
Pampang Bridge	5.0	20.0	1
Pannara	1.5	1.5	2

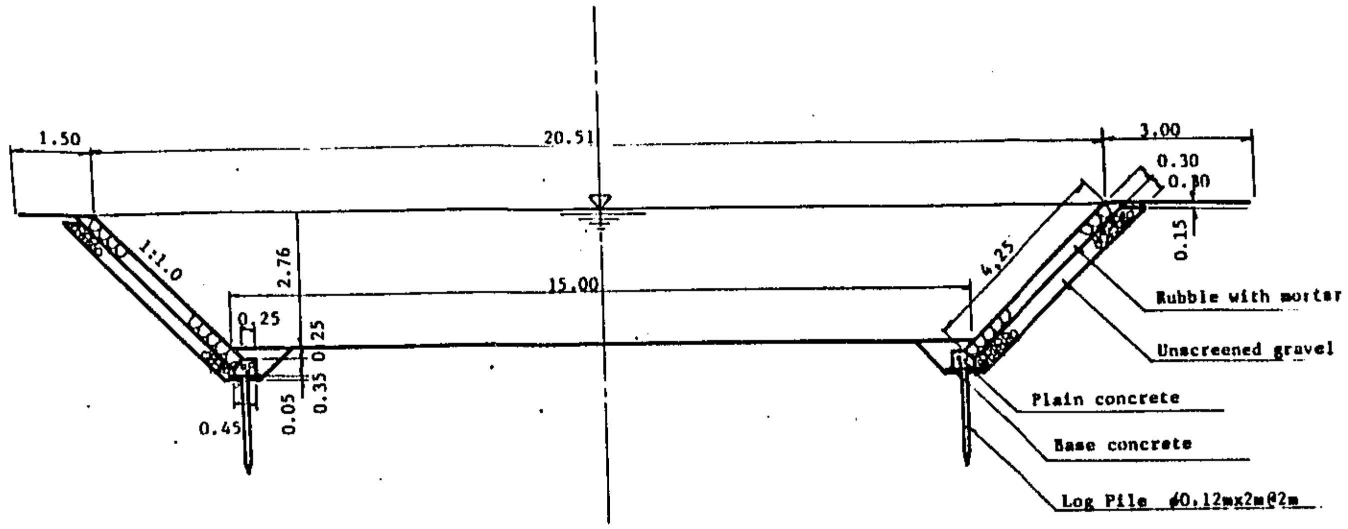
Fig. 2-14 STANDARD STRUCTURE OF DRAINAGE CHANNEL (OVERALL PLAN)

UNIT: M

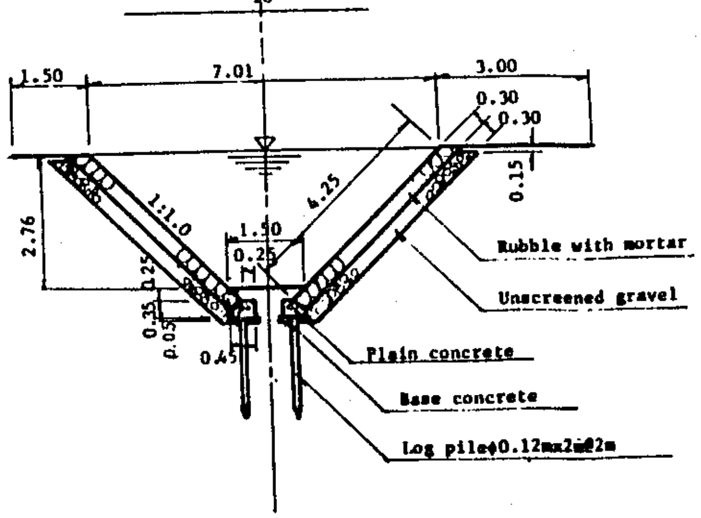
NO.T_1-1.0-100 to NO.T_2-3.0 (TYPE A)



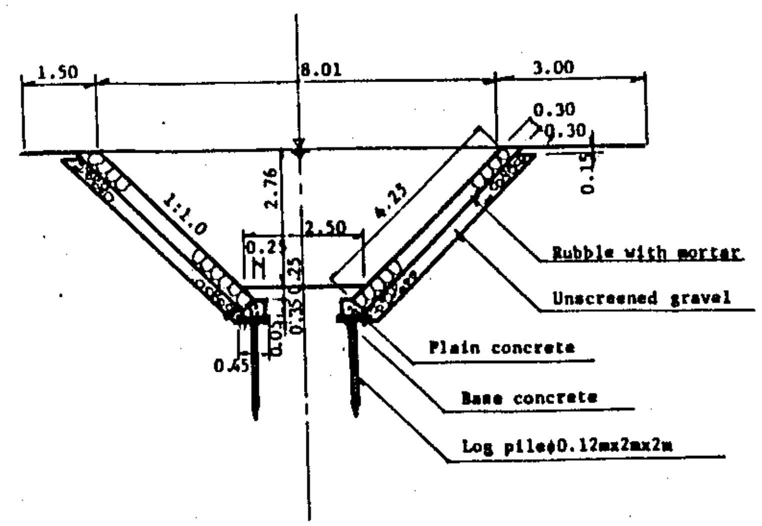
NO.T_2-3.0 to NO.R_{10} (TYPE B)



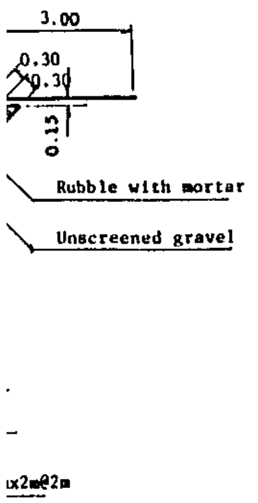
NO.B to NO.R_{10} (TYPE D)



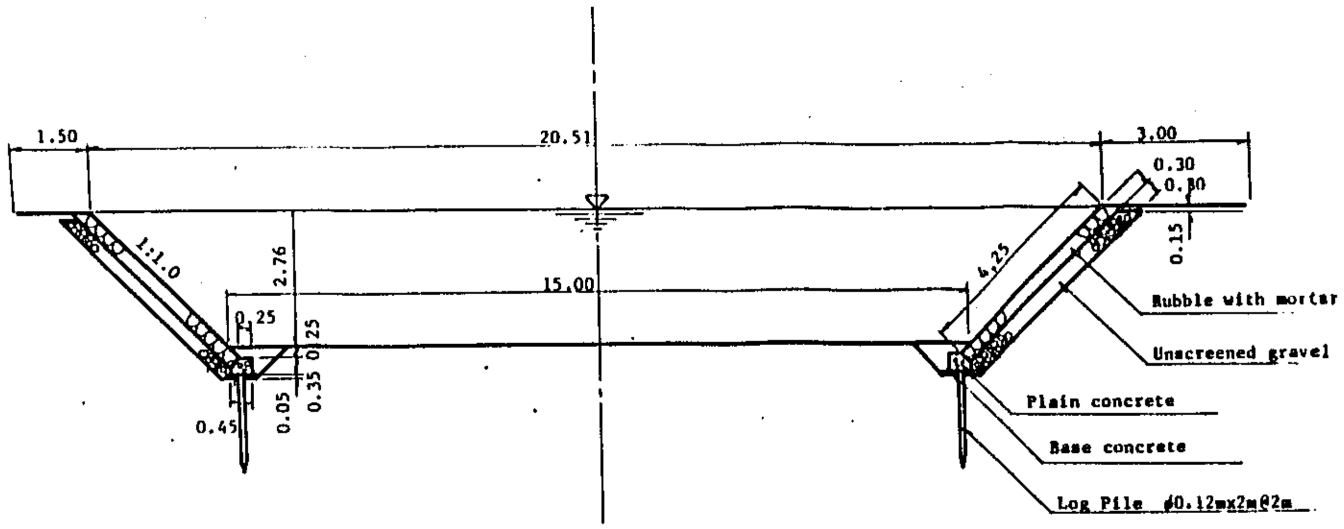
NO.R_{10} to NO.K (TYPE E_1)



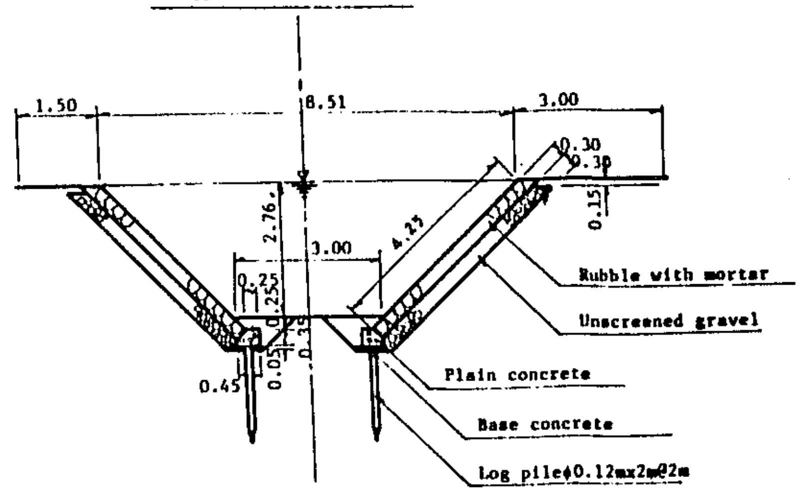
STANDARD STRUCTURE OF DRAINAGE CHANNEL (OVERALL PLAN) UNIT: M



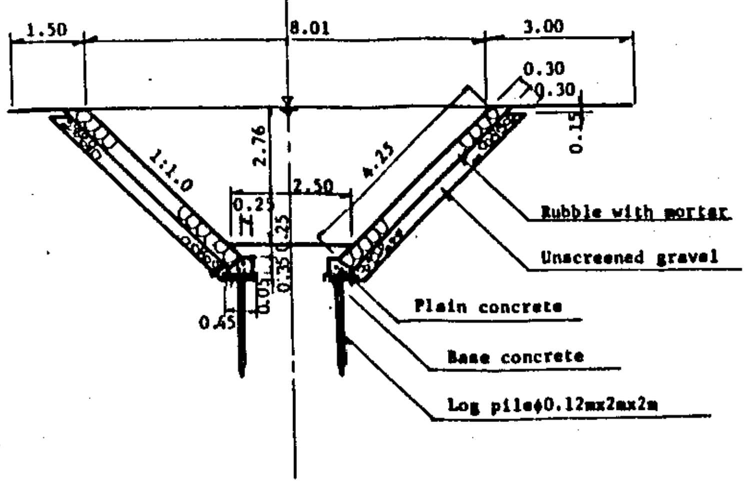
NO.T_2-3.0 to NO.R_{10} (TYPE B)



NO.R_{10} to NO.R_{14} (TYPE C)



NO.R_{10} to NO.K (TYPE E_1)



NO.K to NO.H (TYPE E_2)

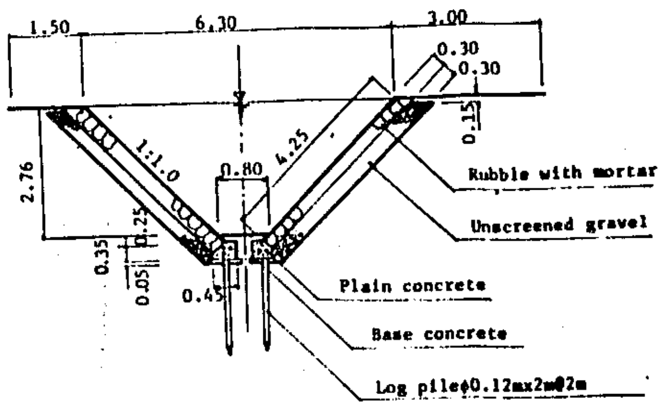


Fig 3-1 POSSIBLE DRAINAGE SYSTEM FOR URGENT PLAN

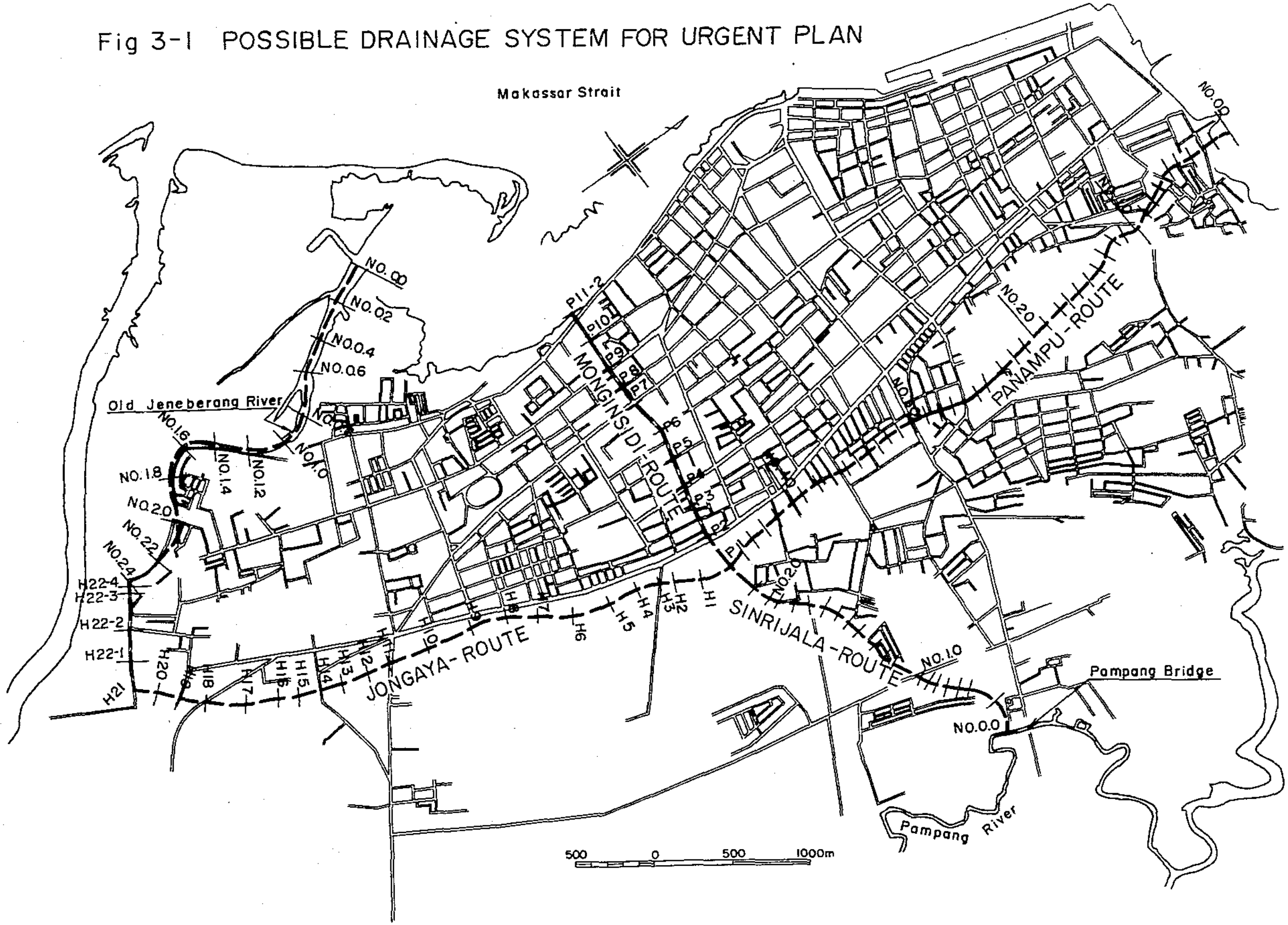


Fig. 3-2 CALCULATION MODEL FOR URGENT PLAN

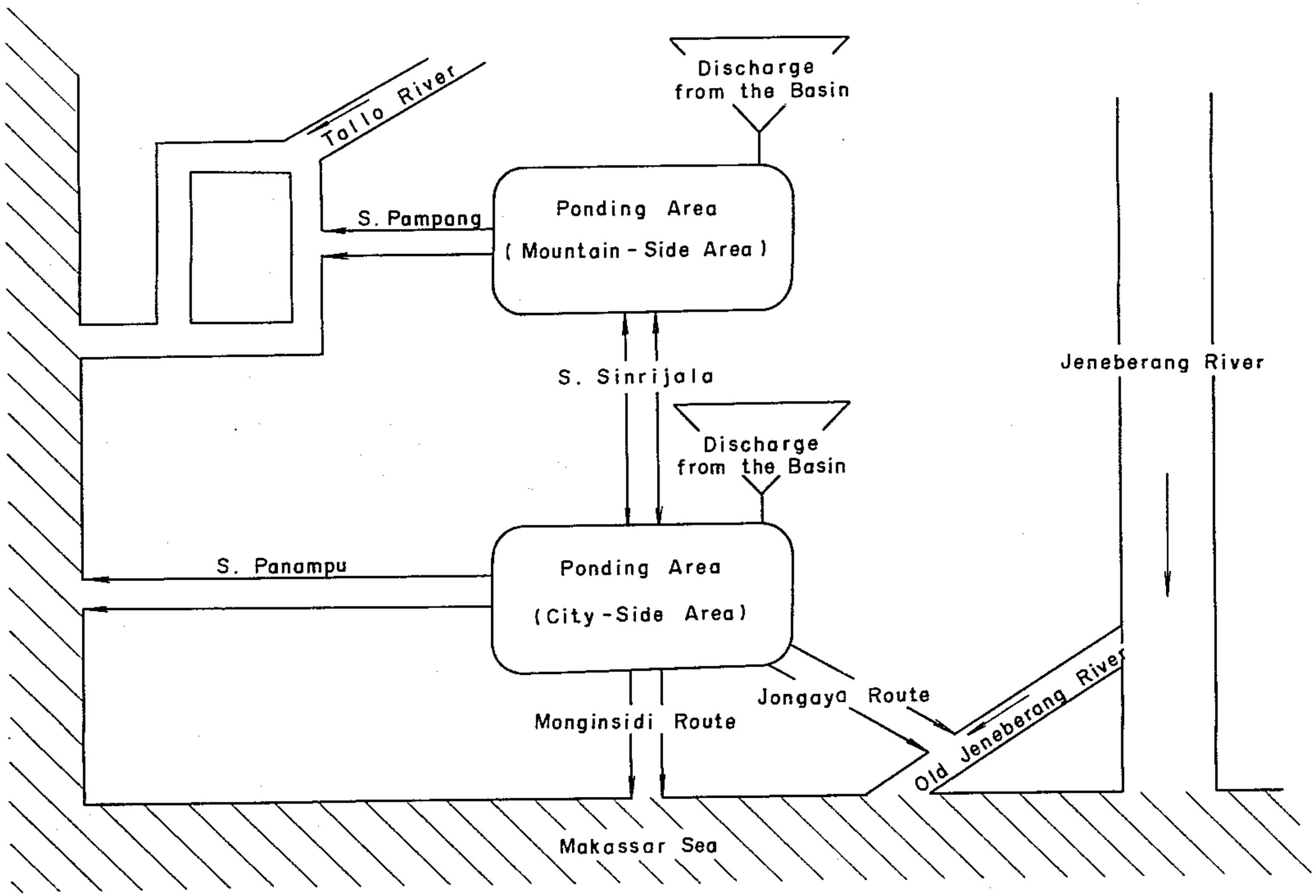


Fig.3-3 OUTLET WATER STAGE FOR URGENT PLAN

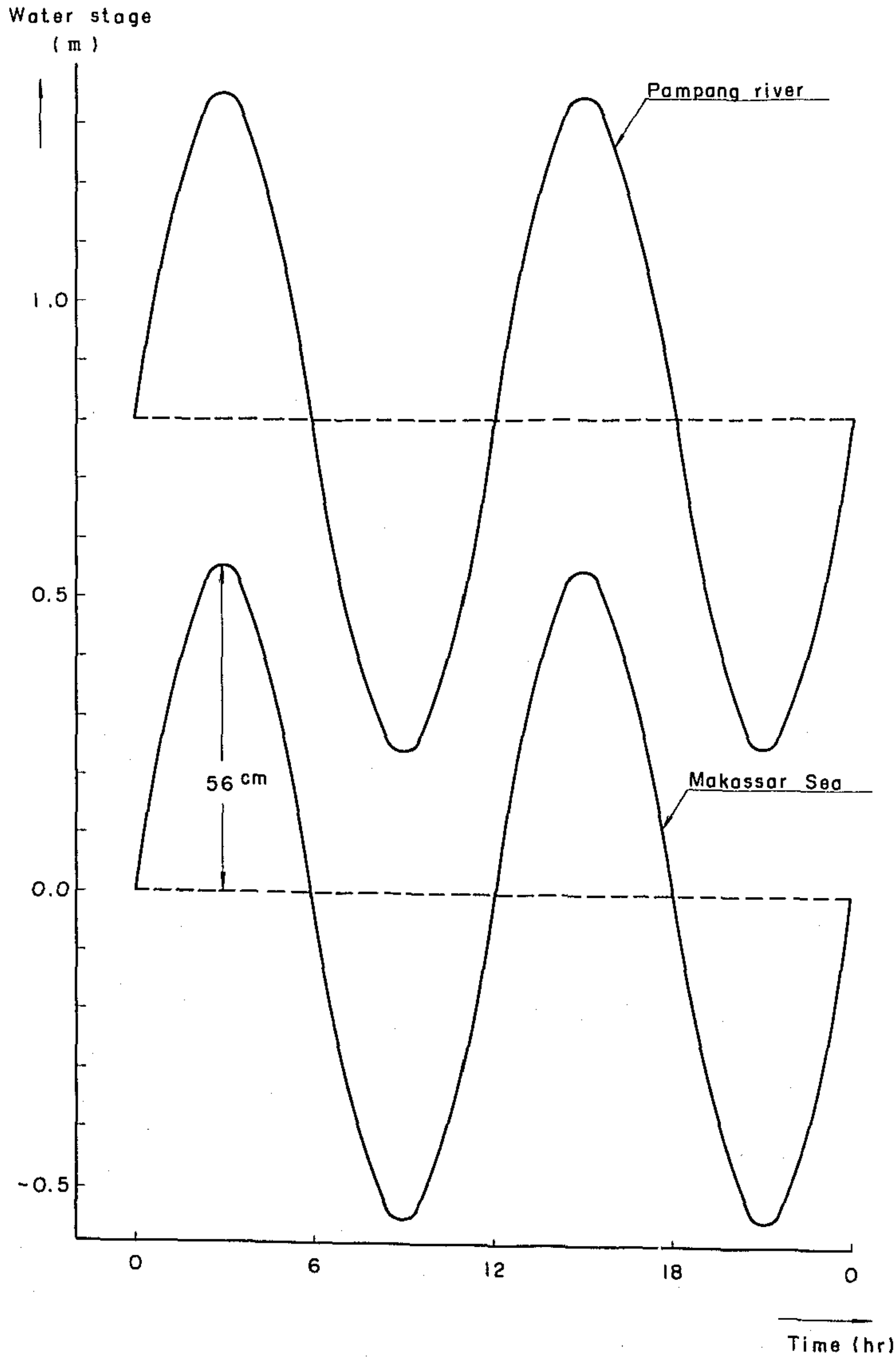


Fig. 3-4 CONSTRUCTION COST BY CHANNEL CAPACITY FOR URGENT PLAN

(Urgent Plan)

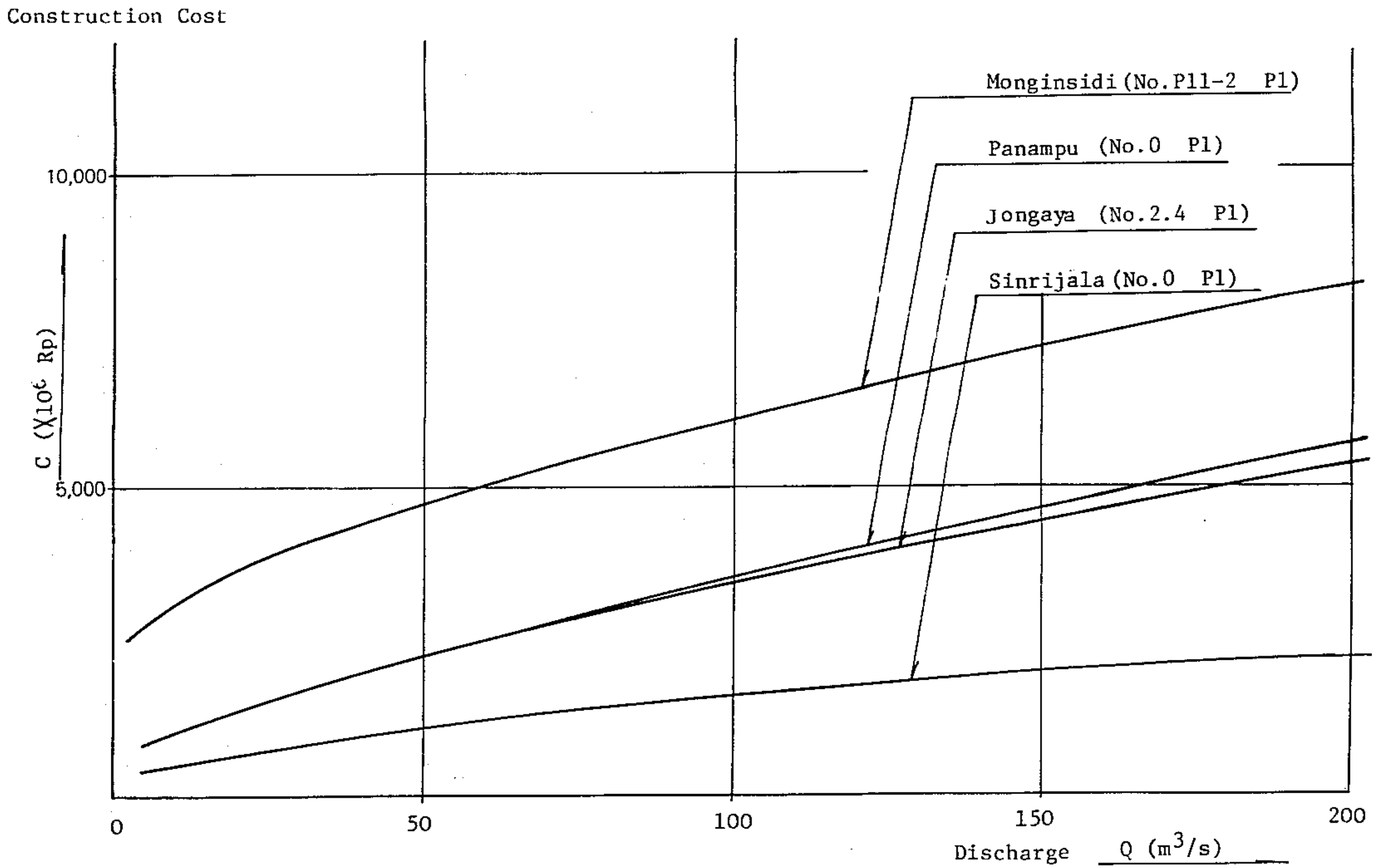
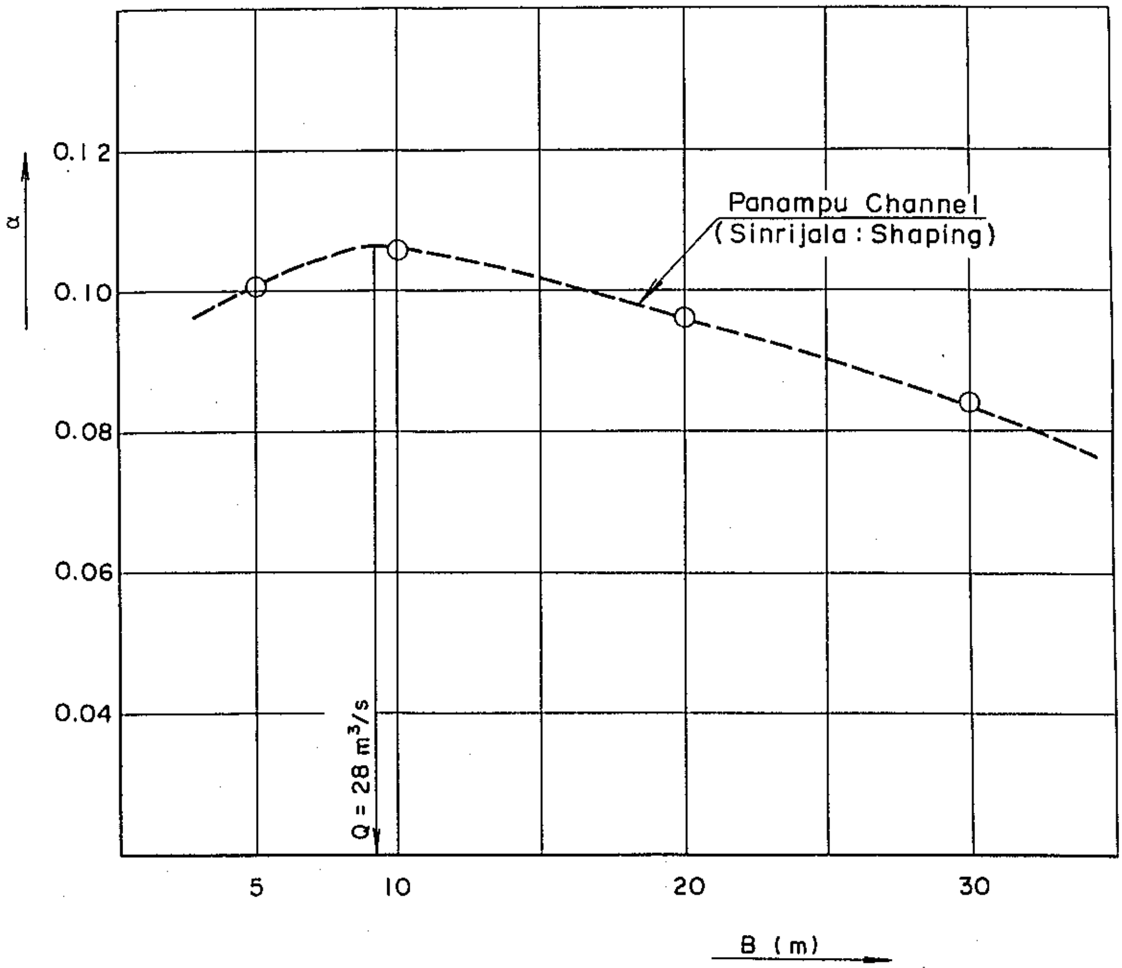


Fig.3-5 OPTIMUM SCALE OF PANAMPU IMPROVEMENT
Existing Stage

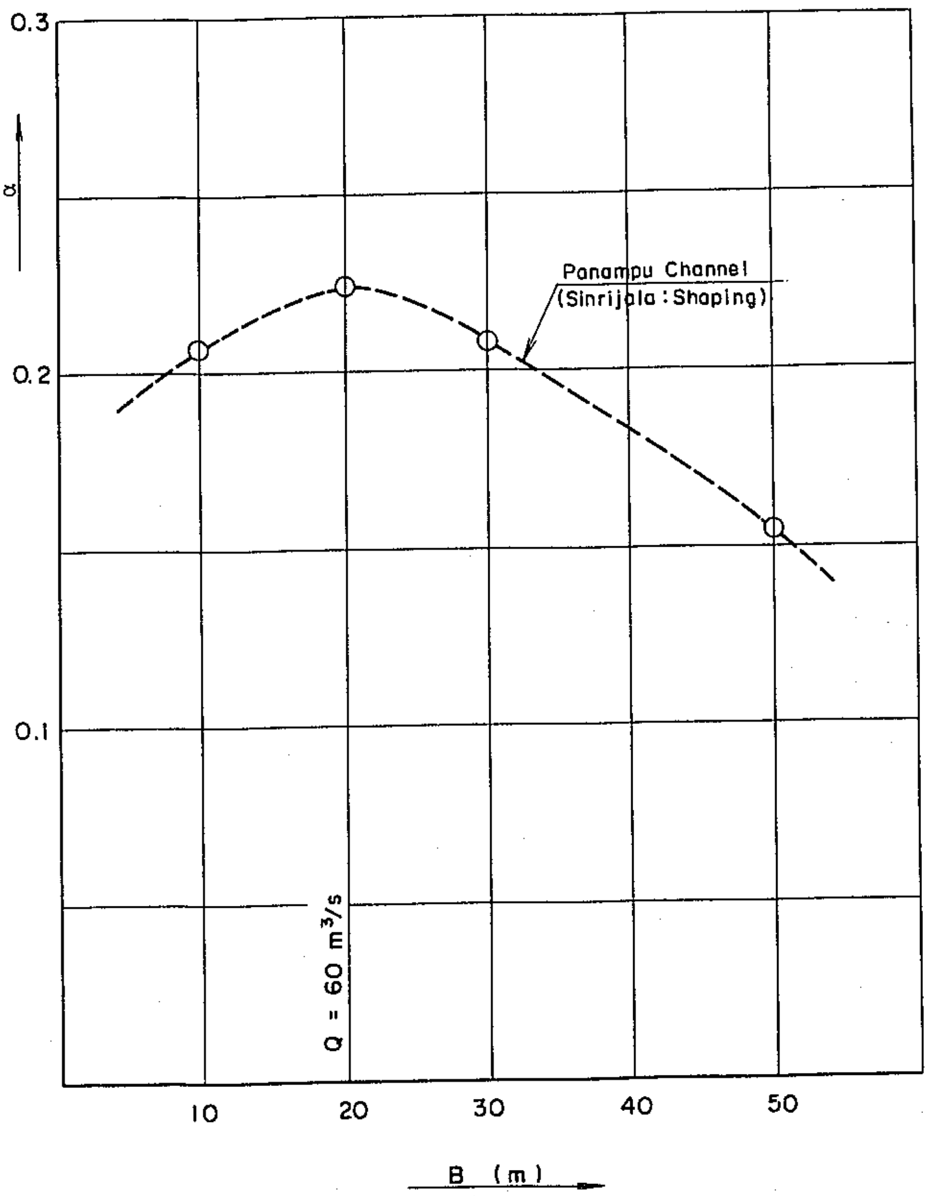


α = Annual Expected Damage Reduction / Construction Cost

B = Bottom Width of Panampu Channel

Q = Dischange of Panampu Channel

Fig. 3-6 OPTIMUM SCALE OF PANAMPU IMPROVEMENT
First Development Stage

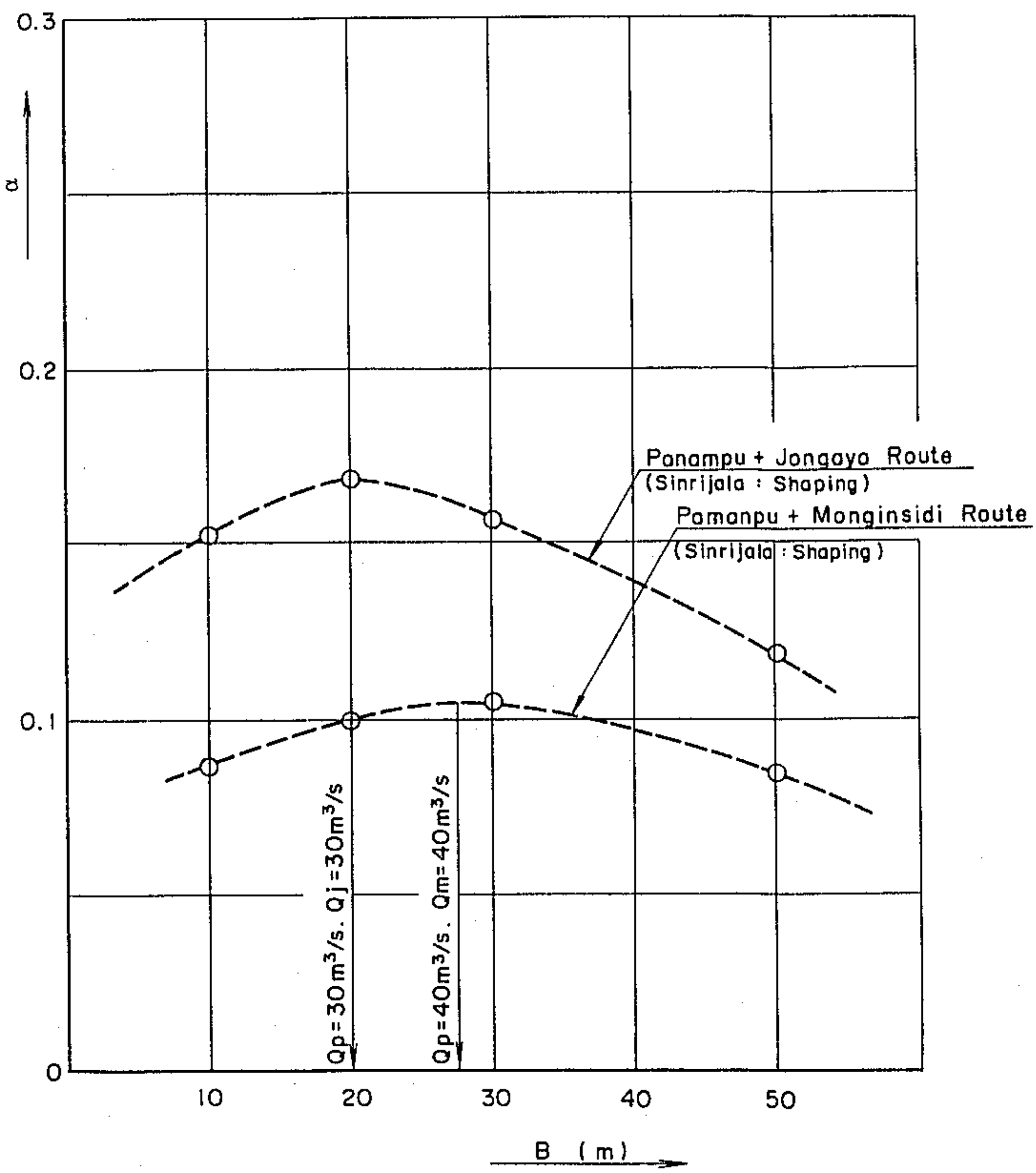


α = Annual Expected Damage Reduction / Construction Cost

B = Bottom Width of Panampu Channel

Q = Dischange of Panampu Channel

Fig. 3-7 OPTIMUM SCALE OF TWO DRAINAGE CHANNELS
First Development Stage



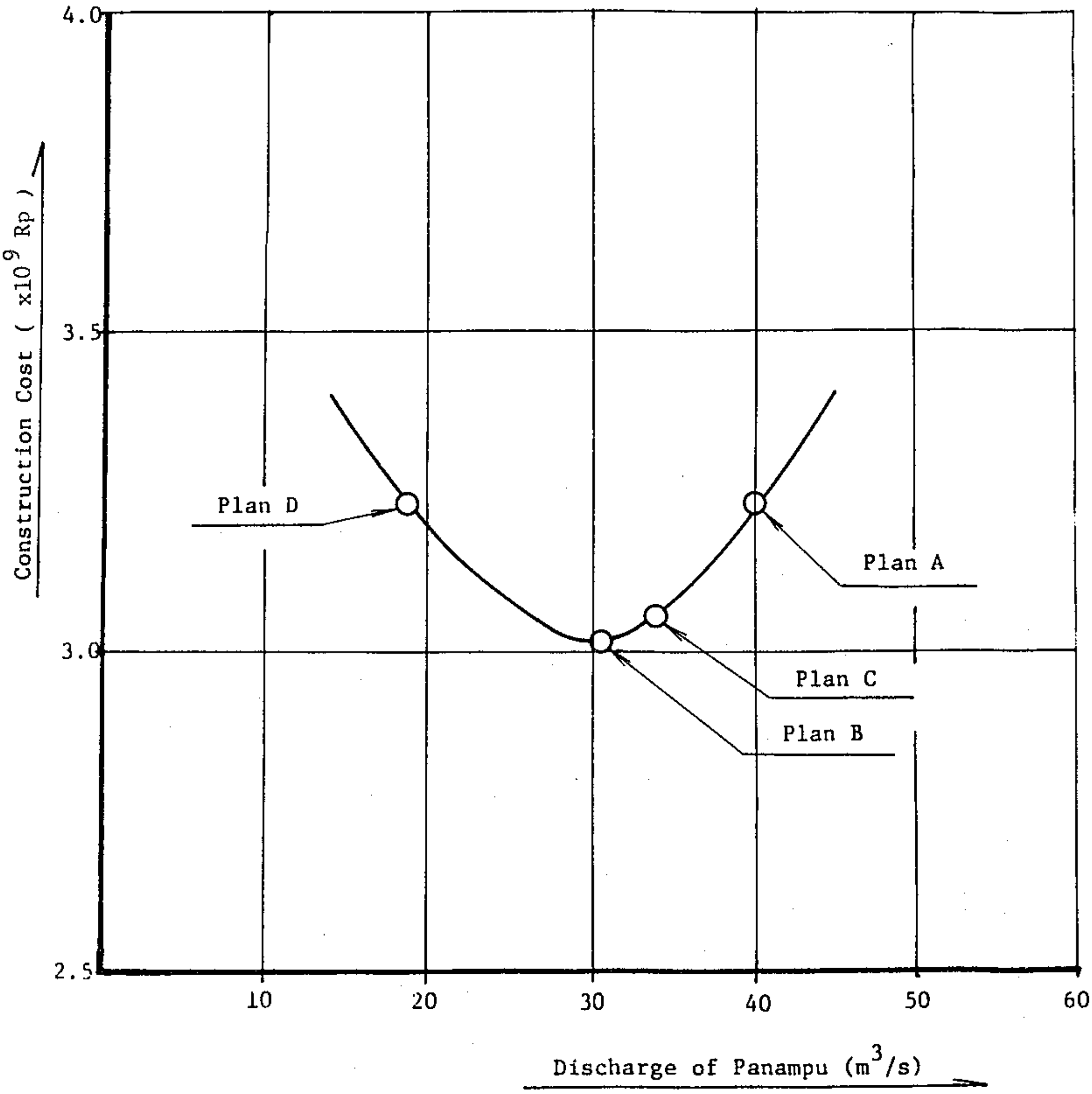
α = Annual Expected Damage Reduction / Construction Cost

B = Bottom Width of Panampu Channel

Q = Dischange (Qp = Panampu, Qj = Jangaya, Qm = Mouginsidi)

Fig.3-8 ALLOCATION OF DRAINAGE CAPACITY

Panampu and Jongaya Channels



Boundary of Panampu and Jongaya Channel

Plan A : No.H_{11}, Plan B : No. H_3, Plan C : No. H_7,
Plan D : No.3.0 (Ref. Table 3-7)

Fig.3-9 LOCATION OF OPTIMUM DRAINAGE SYSTEM FOR URGENT PLAN

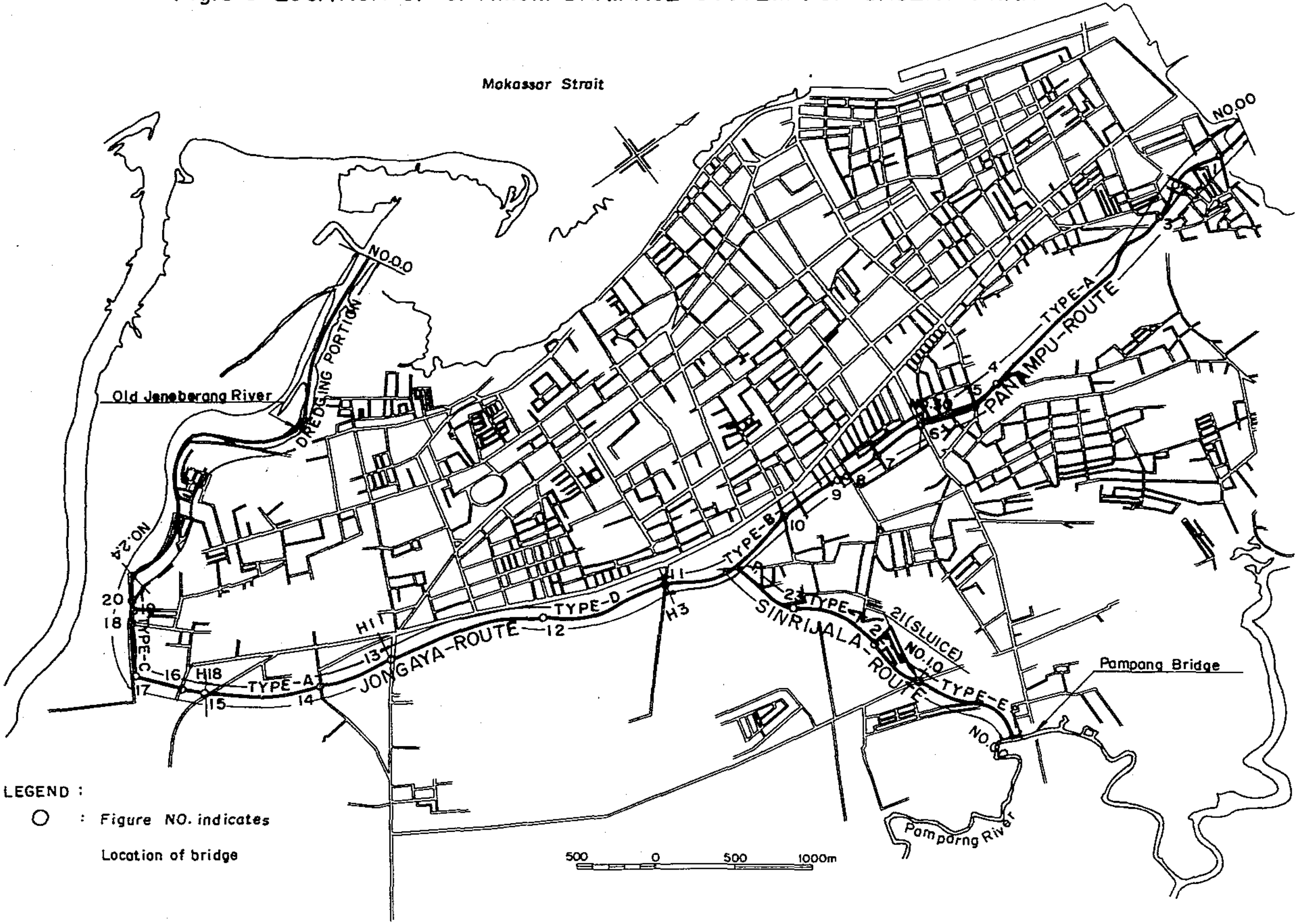
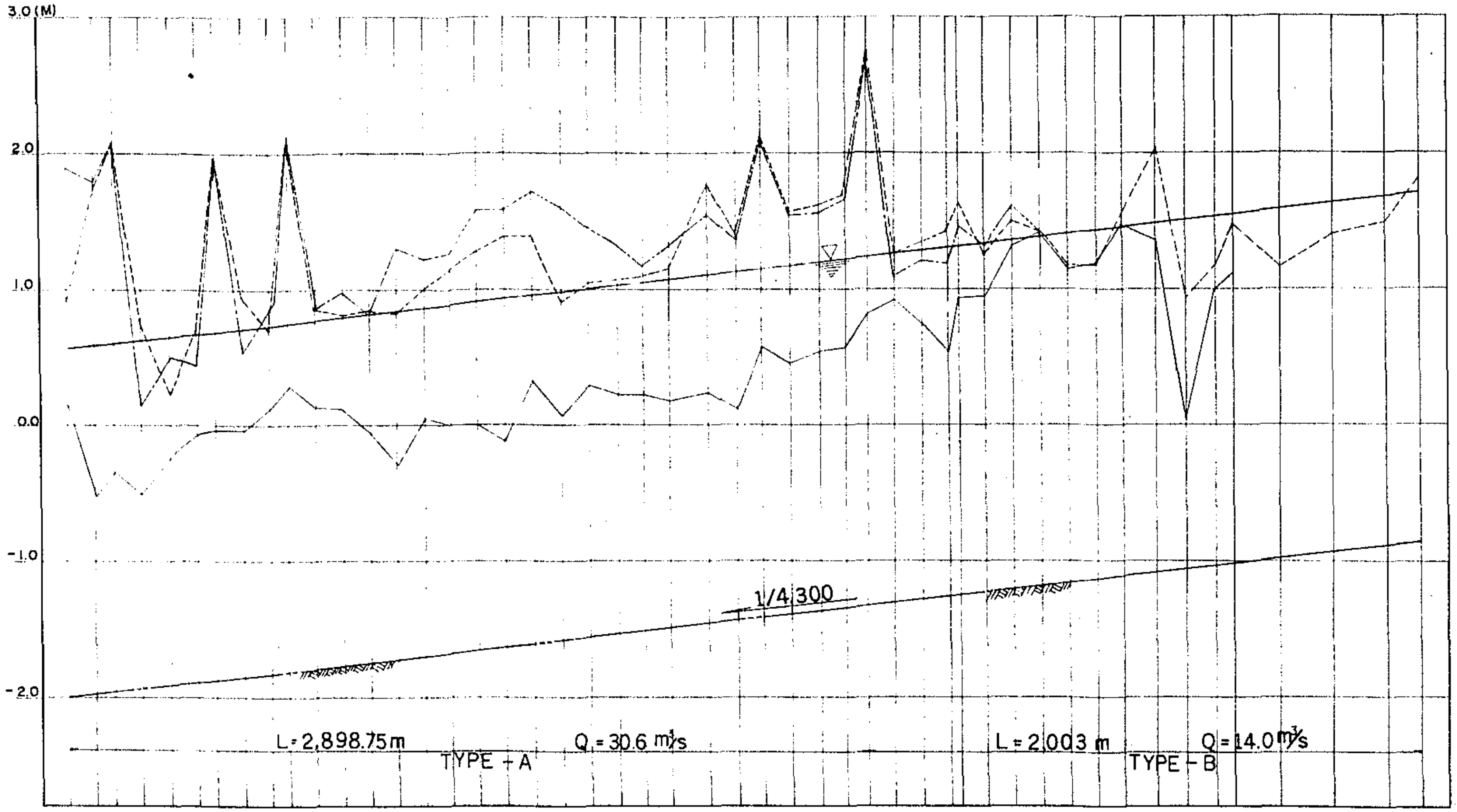


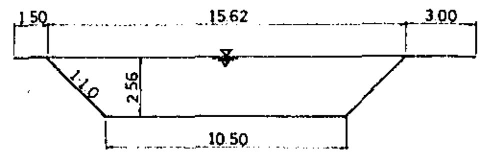
Fig. 3-10 (I) LONGITUDINAL PROFILE OF OPTIMUM DRAINAGE CHANNEL FOR URGENT PLAN
PANAMPU ROUTE NO. 00 – NO. H3

STRETCH NO.00 TO NO. H3	
SCALE V	
SCALE H	
	ELEVATION OF EXISTING RIVER BED
	LEFT SIDE GROUND LEVEL
	RIGHT SIDE GROUND LEVEL
	ELEVATION OF DESIGN RIVER BED
	DESIGN HIGH WATER LEVEL

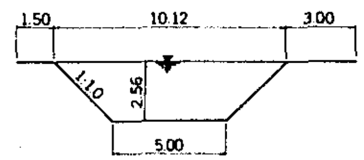
Scale
V 0 0.5 1.0 (m)
H 0 500 (m)



NO 0 to NO.3.0
(TYPE A)

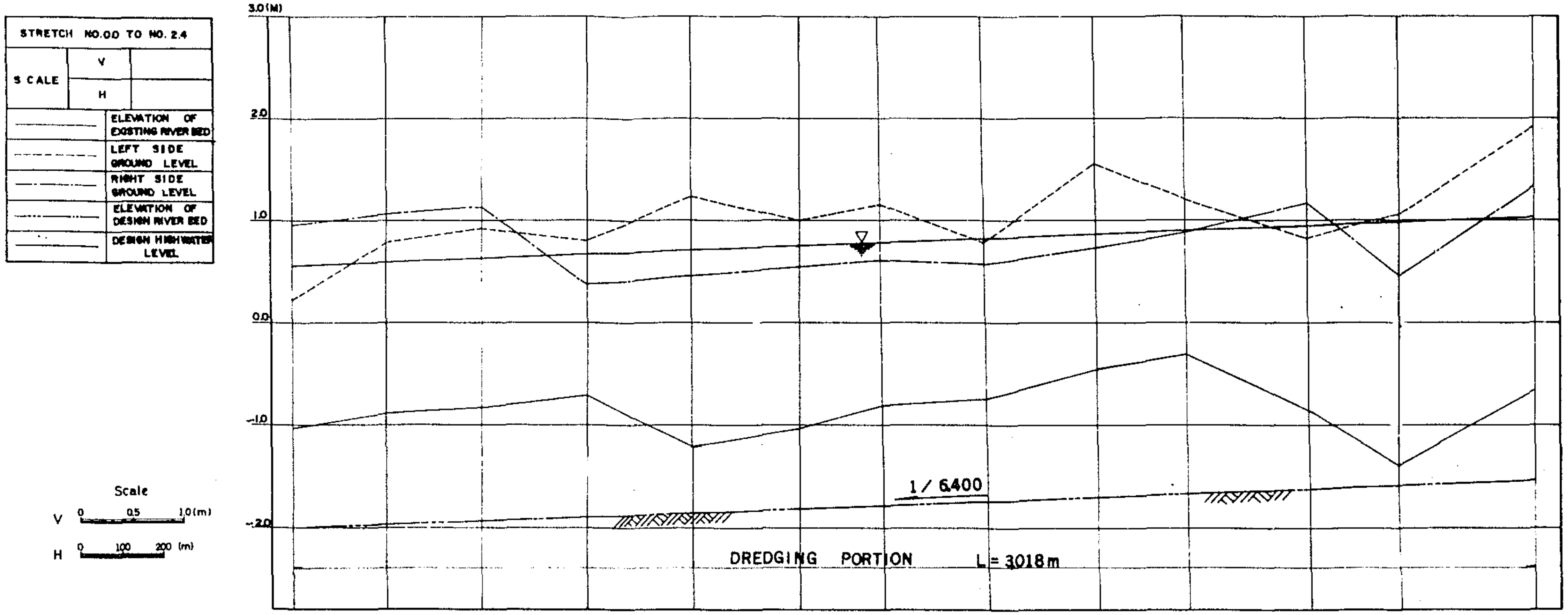


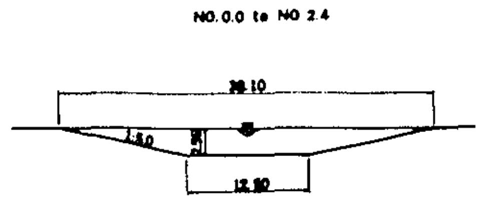
NO 3.0 to NO. H3
(TYPE B)



SECTION NO.	DISTANCE: DISTANCE	DISTANCE: ACCUMULATIVE DISTANCE	EXISTING CONDITION: ELEVATION OF EXISTING RIVER BED	EXISTING CONDITION: LEFT SIDE GROUND LEVEL	EXISTING CONDITION: RIGHT SIDE GROUND LEVEL	DESIGN: ELEVATION OF DESIGN RIVER BED	DESIGN: DESIGN HIGH WATER LEVEL
0.0	0.00	0.00	0.14	0.89	1.89	-2.000	0.560
0.1	99.10	99.10	-0.53	1.73	1.79	-1.977	0.583
0.2	68.15	167.25	-0.34	2.08	2.09	-1.961	0.599
0.3	100.00	267.25	0.51	0.41	0.16	-1.938	0.622
0.4	100.00	367.25	-0.22	0.23	0.52	-1.915	0.645
0.5	100.00	467.25	-0.07	0.70	0.45	-1.891	0.669
0.6	63.40	530.65	-0.03	1.98	1.99	-1.877	0.683
0.7	100.00	630.65	0.04	0.95	0.54	-1.853	0.707
0.8	100.00	730.65	0.14	0.69	0.87	-1.830	0.730
0.9	66.10	796.75	0.29	2.16	2.11	-1.815	0.745
1.0	100.00	896.75	0.13	0.85	0.85	-1.791	0.769
1.1	100.00	996.75	0.12	0.81	0.98	-1.768	0.792
1.2	100.00	1096.75	-0.06	0.84	0.81	-1.745	0.815
1.3	100.00	1196.75	-0.29	0.81	1.31	-1.722	0.838
1.4	100.00	1296.75	0.05	0.99	1.23	-1.698	0.862
1.5	100.00	1396.75	0.00	1.14	1.27	-1.675	0.885
1.6	100.00	1496.75	0.00	1.28	1.58	-1.652	0.908
1.7	100.00	1596.75	-0.12	1.38	1.59	-1.629	0.931
1.8	100.00	1696.75	0.33	1.39	1.72	-1.605	0.955
1.9	100.00	1796.75	0.06	0.89	1.61	-1.582	0.978
2.0	100.00	1896.75	0.29	1.03	1.46	-1.559	1.001
2.1	100.00	1996.75	0.23	1.06	1.34	-1.536	1.024
2.2	100.00	2096.75	0.23	1.09	1.17	-1.512	1.048
2.3	100.00	2196.75	0.18	1.15	1.32	-1.489	1.071
2.4	132.00	2328.75	0.24	1.78	1.56	-1.458	1.102
2.5	100.00	2428.75	0.11	1.34	1.38	-1.435	1.125
2.6	90.00	2518.75	0.58	2.13	2.13	-1.414	1.146
2.7	100.00	2618.75	0.45	1.56	1.54	-1.391	1.169
2.8	100.00	2718.75	0.55	1.61	1.56	-1.368	1.192
2.9	100.00	2818.75	0.57	1.68	1.66	-1.344	1.216
3.0	80.00	2898.75	0.82	2.75	2.73	-1.326	1.234
3.1	100.00	2998.75	0.92	1.27	1.10	-1.303	1.257
3.2	100.00	3098.75	0.76	1.36	1.22	-1.279	1.281
3.3	100.00	3198.75	0.55	1.43	1.19	-1.256	1.304
3.4	31.00	3229.75	0.93	1.70	1.49	-1.249	1.311
3.5	100.00	3329.75	0.95	1.26	1.32	-1.226	1.334
3.6	100.00	3429.75	1.32	1.50	1.62	-1.202	1.358
3.7	100.00	3529.75	1.41	1.43	—	-1.179	1.381
3.8	100.00	3629.75	1.15	1.18	—	-1.156	1.404
3.9	100.00	3729.75	1.19	1.19	—	-1.133	1.427
4.0	100.00	3829.75	1.47	1.62	—	-1.109	1.451
4.1	122.50	3952.25	1.36	2.05	—	-1.091	1.479
4.2	100.00	4052.25	0.02	0.95	—	-1.058	1.502
4.3	100.00	4152.25	0.99	1.18	—	-1.034	1.526
4.4	70.00	4222.25	1.12	1.47	—	-1.018	1.542
P.I	160.00	4382.25	—	1.16	—	-0.980	1.580
H.1	200.00	4582.25	—	1.40	—	-0.934	1.626
H.2	200.00	4782.25	—	1.48	—	-0.888	1.672
H.3	119.50	4901.75	—	1.81	—	-0.860	1.700

Fig. 3-10 (2) LONGITUDINAL PROFILE OF OPTIMUM DRAINAGE CHANNEL FOR URGENT PLAN
JONGAYA ROUTE NO. 0.0 - NO. 2.4 (1/2)





DESIGN	DESIGN HIGH WATER LEVEL	0.560	0.596	0.631	0.672	0.712	0.752	0.794	0.823	0.866	0.898	0.945	0.981	1.036
	ELEVATION OF DESIGN RIVER BED	-2.000	-1.965	-1.929	-1.888	-1.848	-1.808	-1.776	-1.737	-1.695	-1.661	-1.615	-1.579	-1.523
EXISTING CONDITION	RIGHT SIDE GROUND LEVEL	0.95	1.06	1.14	0.38	0.45	0.55	0.61	0.57	0.73	0.88	1.17	0.46	1.33
	LEFT SIDE GROUND LEVEL	0.22	0.79	0.92	0.80	1.23	1.00	1.16	0.78	1.55	1.21	0.84	1.08	1.91
	ELEVATION OF EXISTING RIVER BED	1.03	0.90	0.83	0.71	-1.21	-1.03	-0.81	-0.74	-0.45	-0.30	-0.84	-1.38	-0.66
DISTANCE	ACCUMULATIVE DISTANCE	0.00	226.00	456.00	718.00	974.00	1231.00	1431.00	1682.00	1952.00	2171.00	2465.00	2693.00	3018.00
	DISTANCE	0.00	226.00	230.00	262.00	256.00	257.00	200.00	251.00	270.00	219.00	294.00	228.00	325.00
	SECTION NO.	0.0	0.2	0.4	0.6	0.8	1.0	1.2	1.4	1.6	1.8	2.0	2.2	2.4

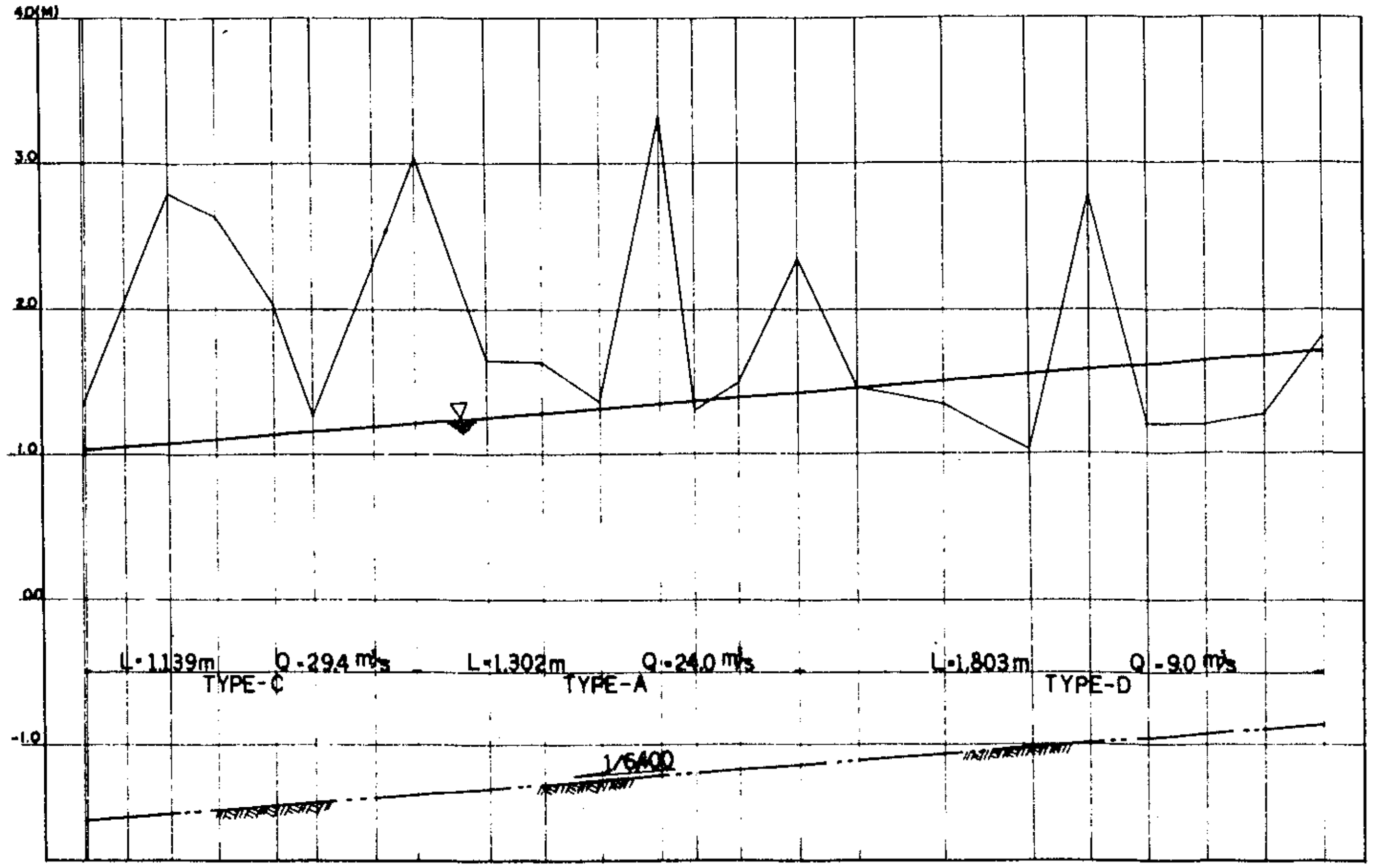
Fig.3-10 (3) LONGITUDINAL PROFILE OF OPTIMUM DRAINAGE CHANNEL FOR URGENT PLAN JONGAYA ROUTE NO. 2.4 - NO. H3 (2/2)

STRETCH NO.2.4 TO NO. H3		
SCALE	V	
	H	
————	ELEVATION OF EXISTING GROUND	
— · — · —	ELEVATION OF DESIGN RIVER BED	
————	DESIGN HIGH WATER LEVEL	

Scale

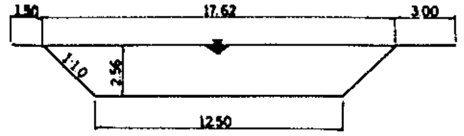
V 0 0.5 1.0 (m)

H 0 500 (m)



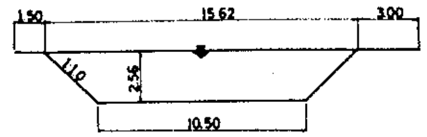
NO H18 to NO. 2.4

(TYPE C)



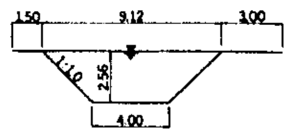
NO. H11 to NO. H18

(TYPE A)



NO H3 to NO H11

(TYPE D)

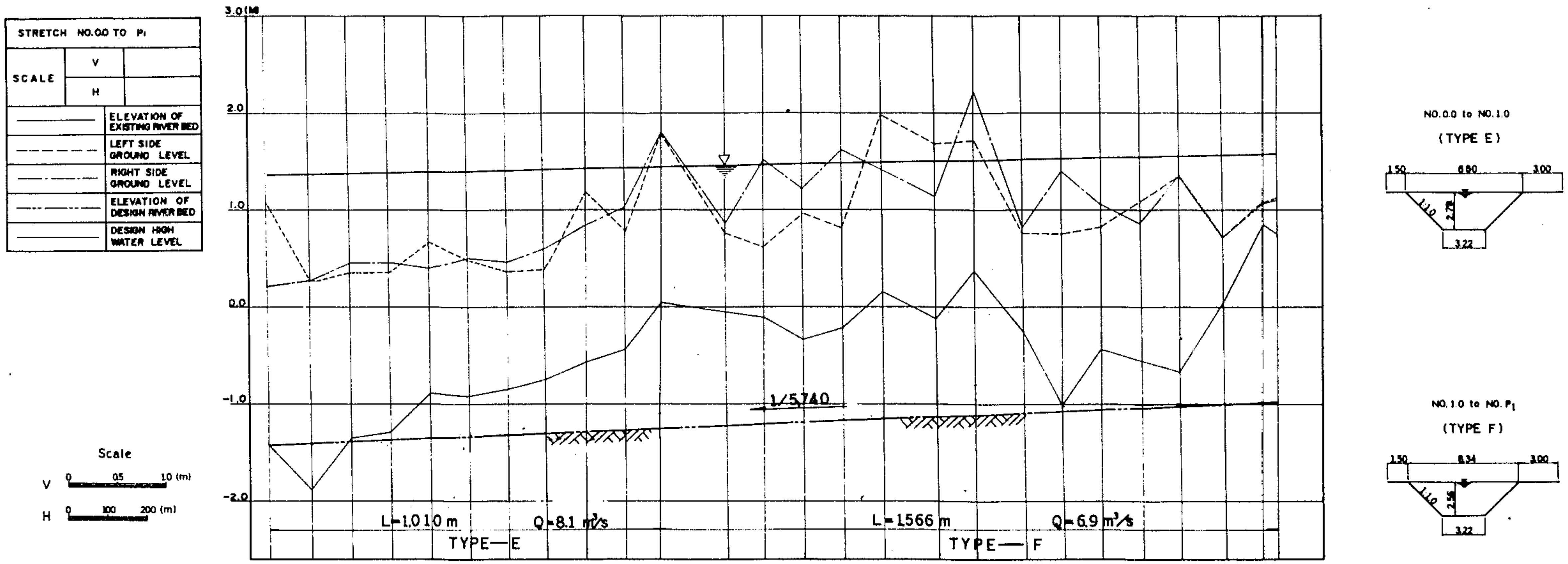


		24	H21.4	H21.3	H21.2	H21.1	H21	H20	H19	H18	H17	H16	H15
DESIGN	DESIGN HIGH WATER LEVEL	1.037	1.038	1.059	1.082	1.108	1.140	1.161	1.192	1.215	1.253	1.283	1.314
	ELEVATION OF DESIGN RIVER BED	-1.523	-1.522	-1.501	-1.476	-1.451	-1.420	-1.399	-1.368	-1.345	-1.307	-1.277	-1.246
EXISTING CONDITION													
	ELEVATION OF EXISTING GROUND	1.33	1.38	2.08	2.81	2.64	2.05	1.28	2.32	3.03	1.63	1.61	1.35
DISTANCE	ACCUMULATIVE DISTANCE	3018.00	3025.50	3159.50	3306.50	3480.50	3680.50	3813.50	4013.50	4157.00	4402.00	4592.00	4792.00
	DISTANCE	328.00	7.50	134.00	147.00	174.00	200.00	133.00	200.00	143.50	245.00	190.00	200.00
	SECTION NO.	24	H21.4	H21.3	H21.2	H21.1	H21	H20	H19	H18	H17	H16	H15

		H14	H13	H12	H11	H10	H9	H8	H7	H6	H5	H4	H3
DESIGN	DESIGN HIGH WATER LEVEL	1.346	1.363	1.387	1.418	1.451	1.498	1.545	1.576	1.606	1.638	1.669	1.700
	ELEVATION OF DESIGN RIVER BED	-1.215	-1.197	-1.173	-1.142	-1.109	-1.062	-1.015	-0.984	-0.954	-0.923	-0.891	-0.860
EXISTING CONDITION													
	ELEVATION OF EXISTING GROUND	3.28	1.29	1.49	2.35	1.46	1.34	1.02	2.81	1.20	1.20	1.27	1.81
DISTANCE	ACCUMULATIVE DISTANCE	4992.00	5104.00	5259.00	5459.00	5669.00	5969.00	6269.00	6469.00	6662.00	6862.00	7062.00	7262.00
	DISTANCE	200.00	112.00	155.00	200.00	210.00	300.00	300.00	200.00	193.00	200.00	200.00	200.00
	SECTION NO.	H14	H13	H12	H11	H10	H9	H8	H7	H6	H5	H4	H3

Fig.3-10 (4) LONGITUDINAL PROFILE OF OPTIMUM DRAINAGE CHANNEL FOR URGENT PLAN

SINRIJALA ROUTE NO. 0.0 - NO. P.



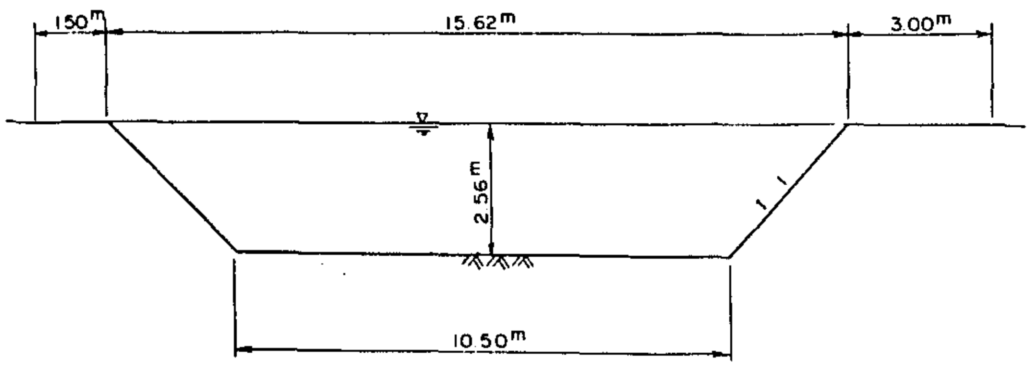
		SECTION NO.	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2
DESIGN	DESIGN HIGH WATER LEVEL		1.350	1.369	1.378	1.386	1.395	1.404	1.412	1.421	1.429	1.438	1.446	1.460	1.468
DESIGN	ELEVATION OF DESIGN RIVER BED		-1.420	-1.410	-1.392	-1.375	-1.358	-1.340	-1.323	-1.305	-1.288	-1.271	-1.253	-1.226	-1.208
EXISTING CONDITION	RIGHT SIDE GROUND LEVEL		1.06	0.26	0.36	0.36	0.67	0.49	0.37	0.39	1.19	0.79	1.79	0.76	0.62
EXISTING CONDITION	LEFT SIDE GROUND LEVEL		0.22	0.27	0.46	0.46	0.40	0.51	0.47	0.61	0.84	1.02	1.80	0.86	1.51
EXISTING CONDITION	ELEVATION OF EXISTING RIVER BED		-1.42	-1.87	-1.34	-1.28	-0.88	-0.91	-0.84	-0.73	-0.56	-0.43	0.05	-0.05	-0.10
DISTANCE	ACCUMULATIVE DISTANCE		0.00	110.00	210.00	310.00	410.00	510.00	610.00	710.00	810.00	910.00	1010.00	1168.00	1268.00
DISTANCE	DISTANCE		0.00	110.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	158.00	100.00

		SECTION NO.	1.3	1.4	1.5	1.6	1.7	1.8	1.9	2.0	2.1	2.2	2.3	2.4	R
DESIGN	DESIGN HIGH WATER LEVEL		1.477	1.486	1.494	1.505	1.514	1.524	1.533	1.542	1.550	1.559	1.568	1.577	1.580
DESIGN	ELEVATION OF DESIGN RIVER BED		-1.191	-1.173	-1.156	-1.131	-1.113	-1.094	-1.076	-1.057	-1.041	-1.024	-1.004	-0.986	-0.980
EXISTING CONDITION	RIGHT SIDE GROUND LEVEL		0.97	0.82	1.98	1.68	1.72	0.77	0.78	0.82	1.09	1.35	0.71	1.08	
EXISTING CONDITION	LEFT SIDE GROUND LEVEL		1.21	1.62	1.41	1.14	2.22	0.82	1.40	1.06	0.86	1.35	0.71	1.08	
EXISTING CONDITION	ELEVATION OF EXISTING RIVER BED		-0.33	-0.21	0.16	-0.12	0.36	-0.24	-1.00	-0.42	-0.55	-0.66	0.06	0.85	
DISTANCE	ACCUMULATIVE DISTANCE		1368.00	1468.00	1568.00	1702.00	1802.00	1926.00	2026.00	2126.00	2226.00	2326.00	2441.00	2541.00	2576.00
DISTANCE	DISTANCE		100.00	100.00	100.00	134.00	100.00	124.00	100.00	100.00	100.00	100.00	115.00	100.00	35.00

Fig. 3-11 (1) STANDARD CROSS-SECTION OF OPTIMUM DRAINAGE CHANNEL FOR URGENT PLAN

Panampu

◦ No. 00 ~ No. 3.0 ($Q = 30.6\ m^3/s$)



$$A = \frac{1}{2}(10.5 + 10.5 + 2.56 + 2.56) \times 2.56 = 33.43$$

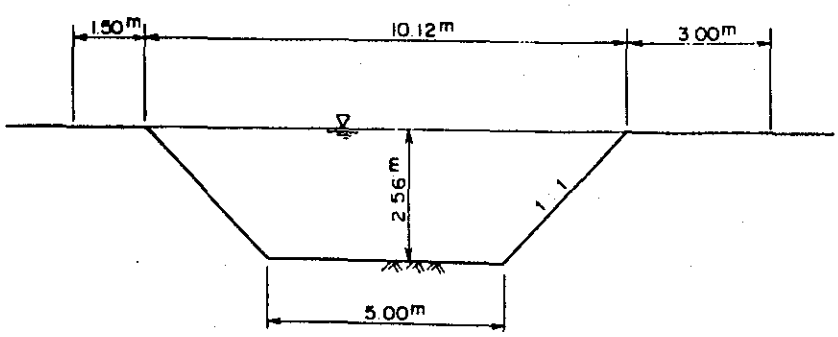
$$P = 10.5 + 2\sqrt{2.56^2 + 2.56^2} = 17.74$$

$$R = 33.43 / 17.74 = 1.88$$

$$V = \frac{1}{0.025} \times (1.88)^{2/3} \times (1/4300)^{1/2} = 0.926$$

$$Q = 33.43 \times 0.926 = 30.9 > 30.6$$

◦ No. 3.0 ~ No. H8 ($Q = 14.3\ m^3/s$)



$$A = \frac{1}{2}(5 + 5 + 2.56 + 2.56) \times 2.56 = 19.35$$

$$P = 5 + 2\sqrt{2.56^2 + 2.56^2} = 12.24$$

$$R = 19.35 / 12.24 = 1.56$$

$$V = \frac{1}{0.025} \times (1.56)^{2/3} \times (1/4300)^{1/2} = 0.818$$

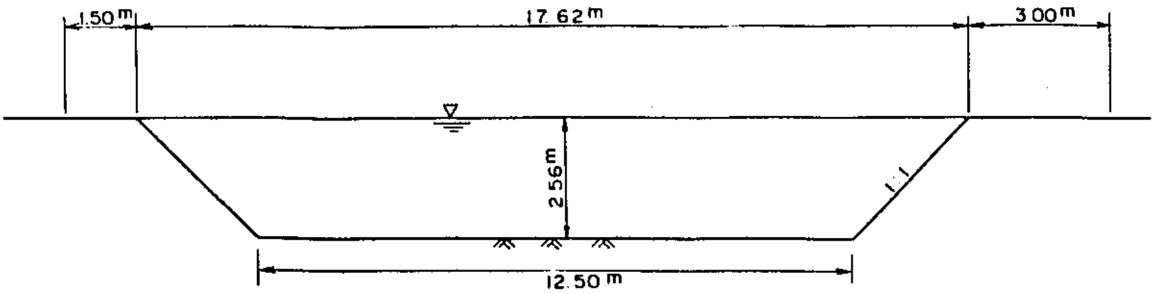
$$Q = 19.35 \times 0.818 = 15.8 > 14.3\ m^3/s$$

* A : Flow Section Area, R : Hydraulic Depth (A/p), V : Velocity (m/s), Q : Discharge (m^3/s)

Fig. 3-11 (2) STANDARD CROSS-SECTION OF OPTIMUM DRAINAGE CHANNEL FOR URGENT PLAN

Jongaya Channel

◦ No. 2.4^k ~ No. H16



$A = \frac{1}{2}(12.5 + 12.5 + 2.56 + 2.56) \times 2.56 = 38.55$

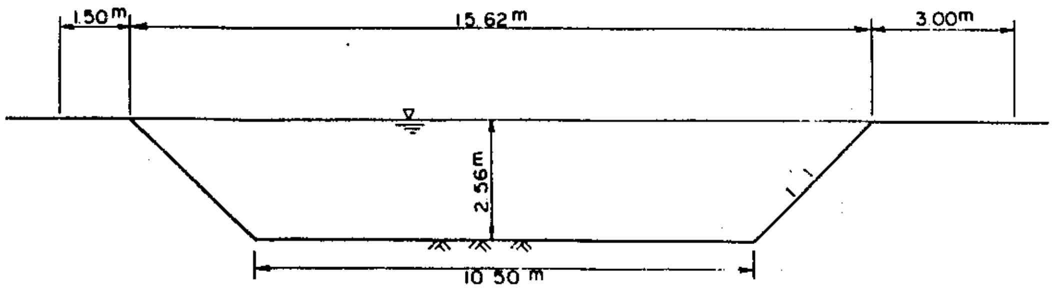
$P = 12.5 + 2\sqrt{2.56^2 + 2.56^2} = 19.74$

$R = 38.55 / 19.74 = 1.95$

$V = \frac{1}{0.025} \times (1.95)^{2/3} \times (1/6400)^{1/2} = 0.780$

$Q = 38.55 \times 0.780 = 30.1 > 29.4$

◦ No. H16 ~ No. H11



$A = \frac{1}{2}(10.50 + 10.50 + 2.56 + 2.56) \times 2.56 = 33.43$

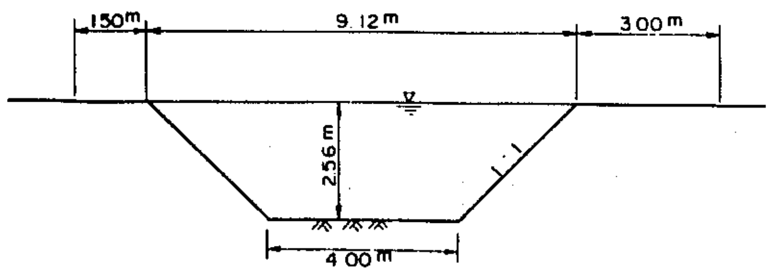
$P = 10.50 + 2\sqrt{2.56^2 + 2.56^2} = 17.74$

$R = 33.43 / 17.74 = 1.88$

$V = \frac{1}{0.025} \times 1.88^{2/3} \times (1/6400)^{1/2} = 0.763$

$Q = 33.43 \times 0.763 = 25.5 > 24.0$

◦ No. H11 ~ No. H3 ($Q = 9.0\ m^3/s$)



$A = \frac{1}{2}(4 + 4 + 2.56 + 2.56) \times 2.56 = 16.70$

$P = 4.0 + 2\sqrt{2.56^2 + 2.56^2} = 11.24$

$R = 16.7 / 11.24 = 1.48$

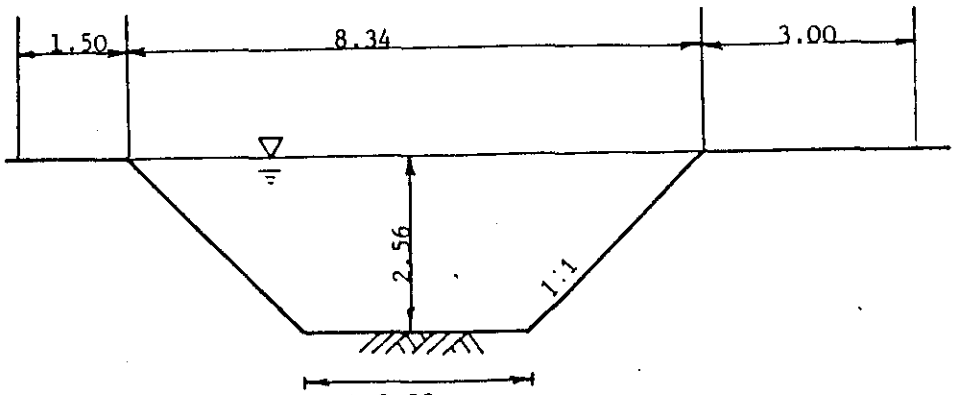
$V = \frac{1}{0.025} \times (1.48)^{2/3} \times (1/6400)^{1/2} = 0.640$

$Q = 16.70 \quad 0.640 = 10.6 > 9.0$

* A : Flow Section Area, R : Hydraulic Depth(A/P), V : Velocity (m/s), Q : Discharge (m^3/s)

Fig. 3-11 (3) STANDARD CROSS-SECTION OF OPTIMUM DRAINAGE CHANNEL FOR URGENT PLAN
Sinrijala Channel

Pl— No.1.0



$$A=\frac{1}{2}(3.22+3.22+2.56+2.56)\times 2.56=14.80$$

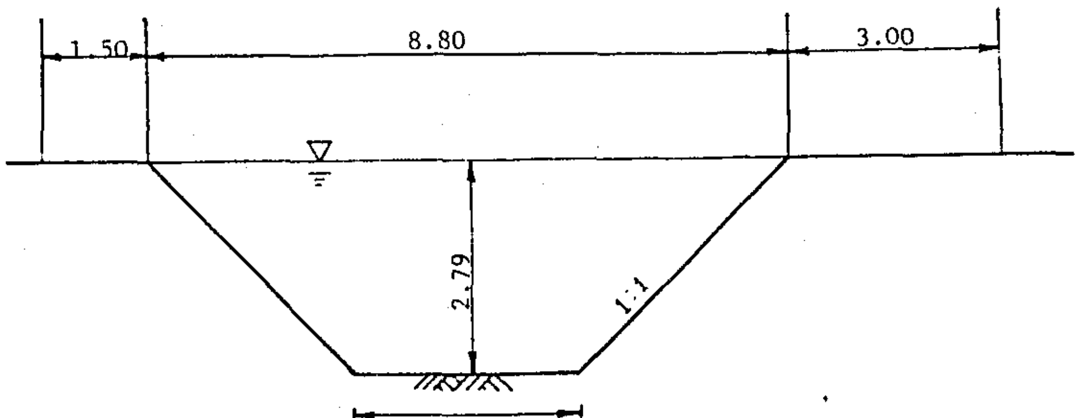
$$P=3.22+2\sqrt{2.56^2+2.56^2}=10.46$$

$$R=1.41$$

$$V=\frac{1}{0.025}\times(1.41)^{2/3}\times(1/11710)^{1/2}=0.466$$

$$Q=6.9m^3/s$$

No.0.0— No.1.0



$$A=\frac{1}{2}(3.22+3.22+2.79+2.79)\times 2.79=16.77$$

$$P=3.22+2\sqrt{2.79^2+2.79^2}=11.11$$

$$R=1.51$$

$$V=\frac{1}{0.025}\times(1.51)^{2/3}\times(1/11710)^{1/2}=0.486$$

$$Q=8.1m^3/s$$

* A : Flow Section Area, R : Hydraulic Depth(A/P), V : Velocity (m/s), Q : Discharge (m^3/s)

Fig. 3-12 PROPCSED SLUICE UNDER PANAKKUKANG ROAD

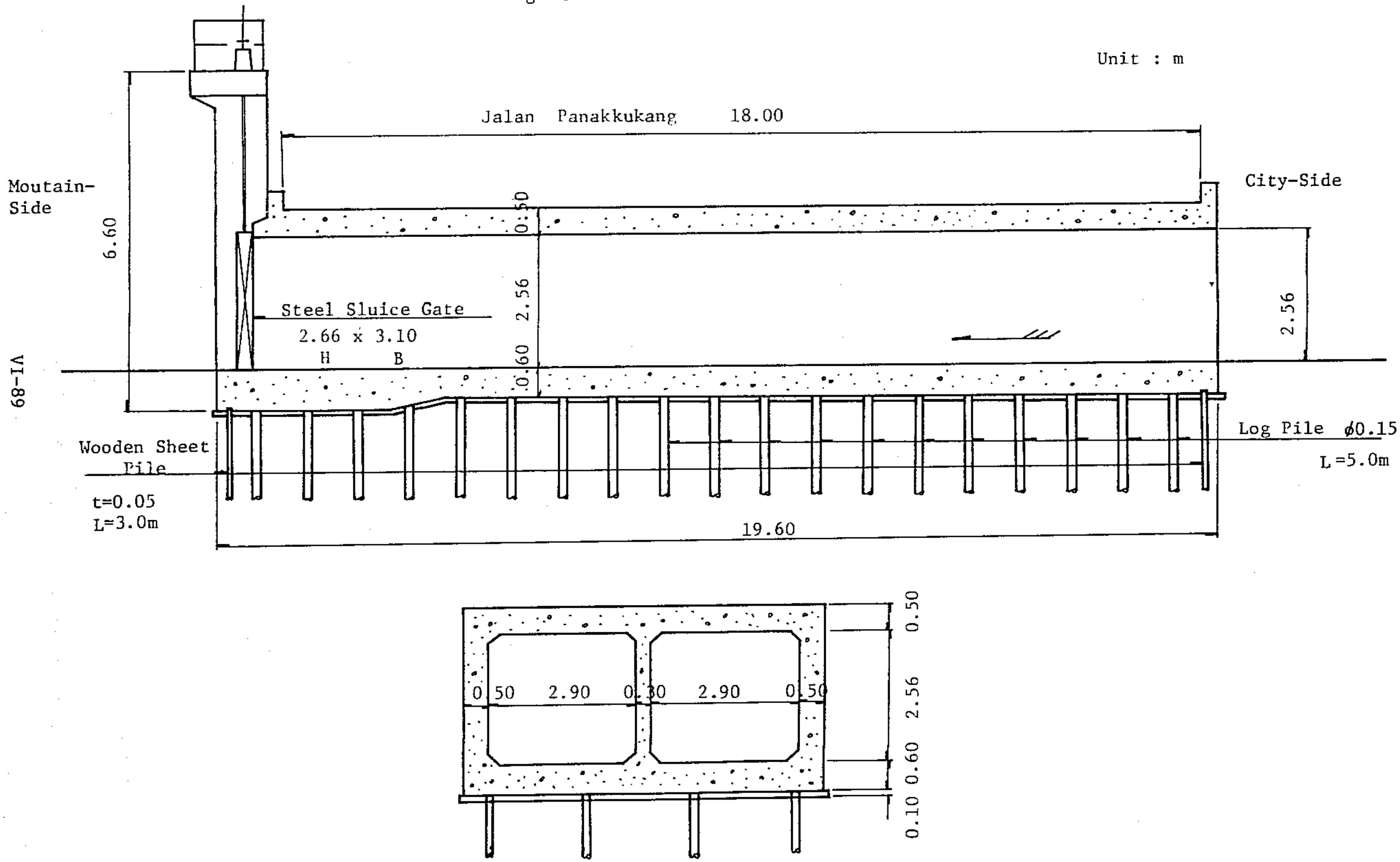


Fig. 3-13 STANDARD DESIGN OF PROPOSED BRIDGE

Unit: m

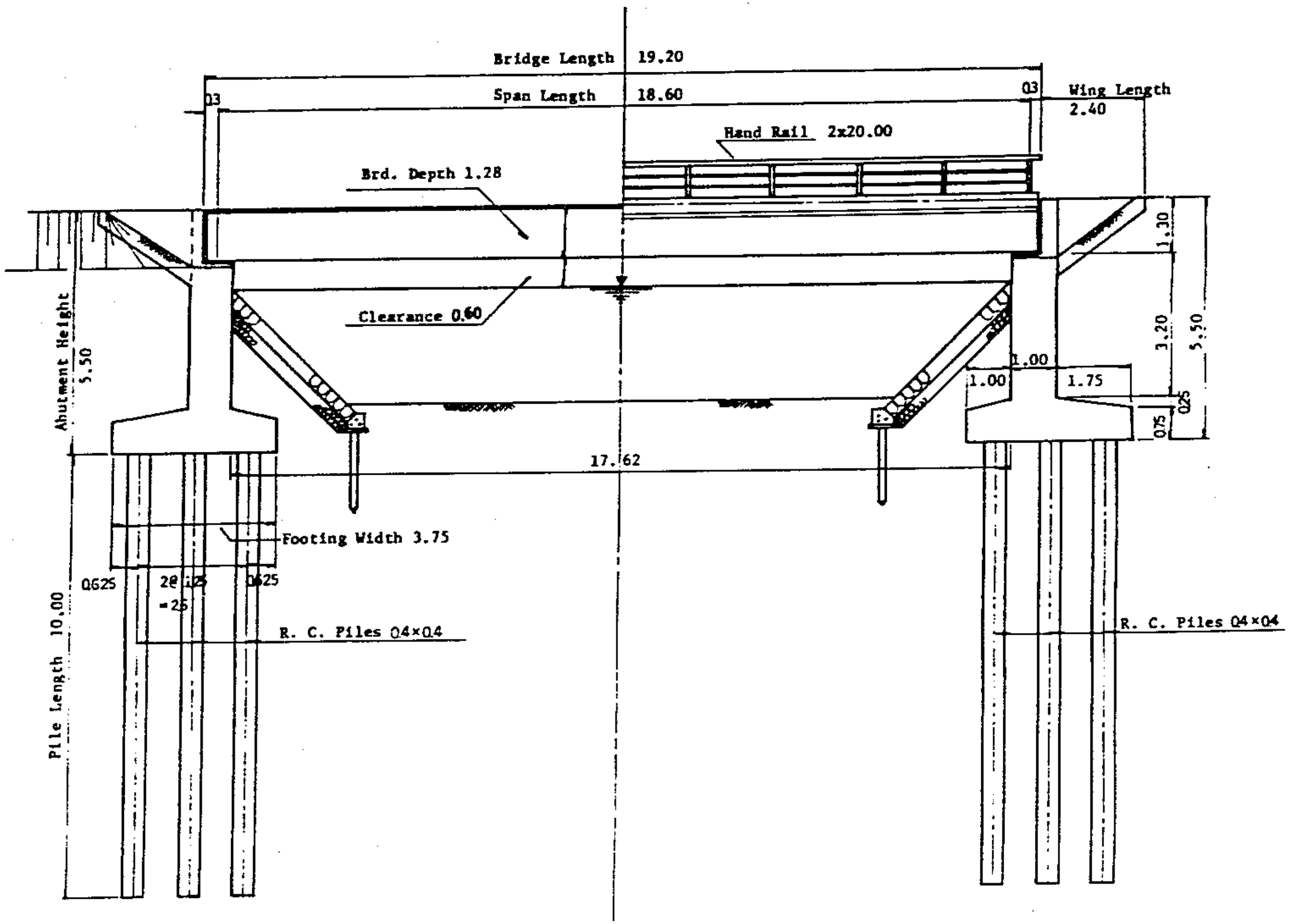
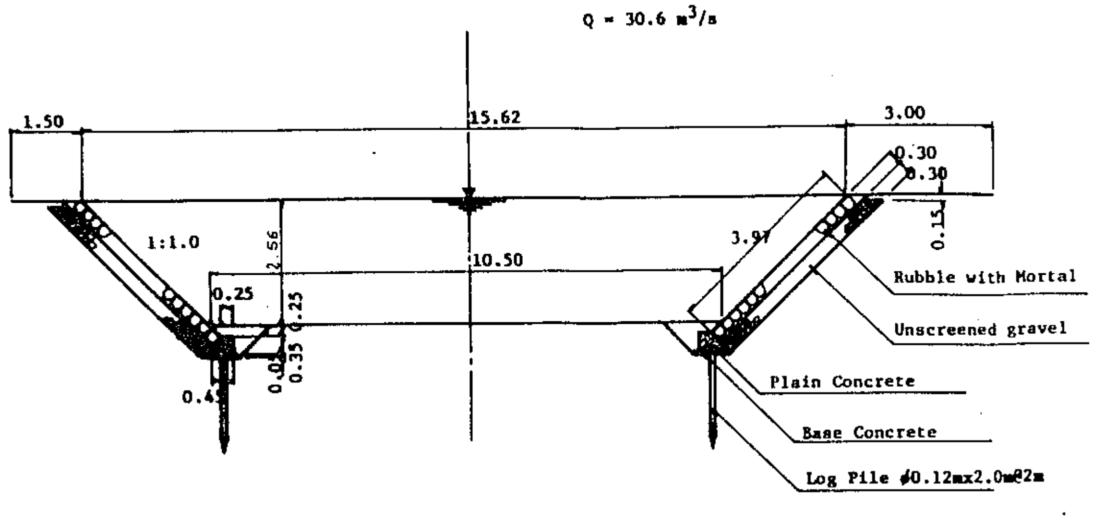


Fig.3-14 (1) STANDARD STRUCTURE OF DRAINAGE CHANNEL (URGENT PLAN)

Unit : m

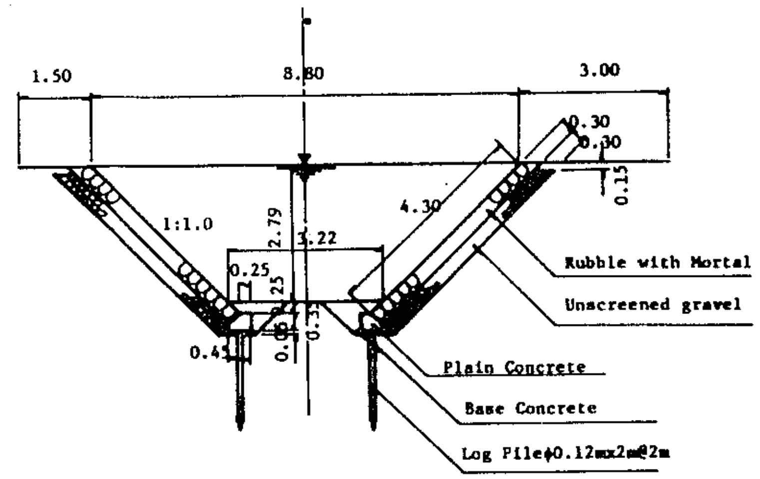
PANAMPU ROUTE
No.0 to No.3.0
(TYPE A)

Q = 30.6 m^3/s



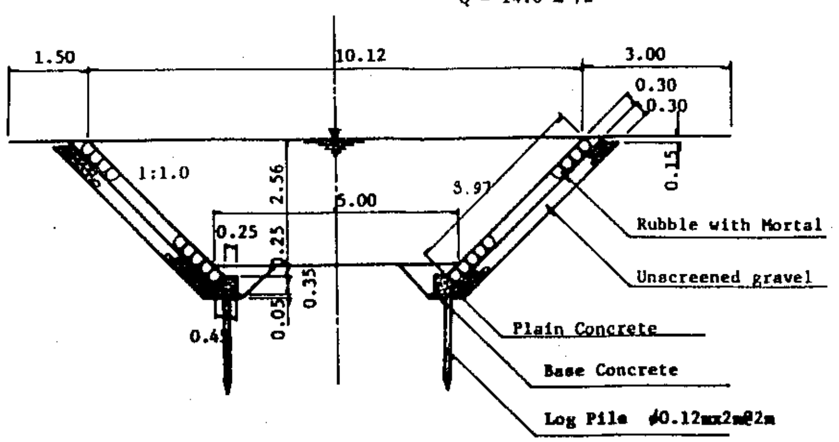
SINRIJALA ROUTE
NO. 0.0 to NO. 1.0
(TYPE E)

Q = 8.1 m^3/s



PANAMPU ROUTE
No.3.0 to No.H_3
(TYPE B)

Q = 14.0 m^3/s



SINRIJALA ROUTE
No. 1.0 to No. P_1
(TYPE F)

Q = 6.9 m^3/s

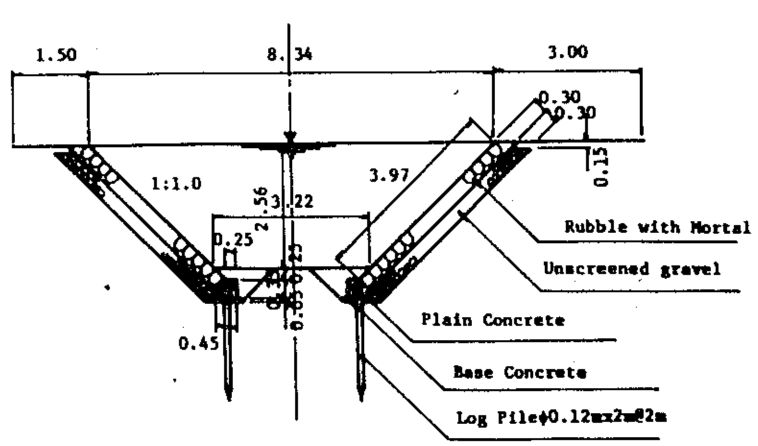
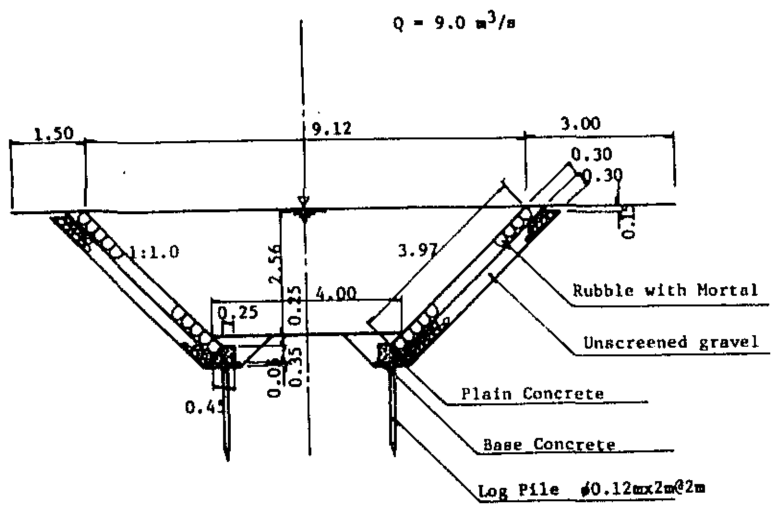


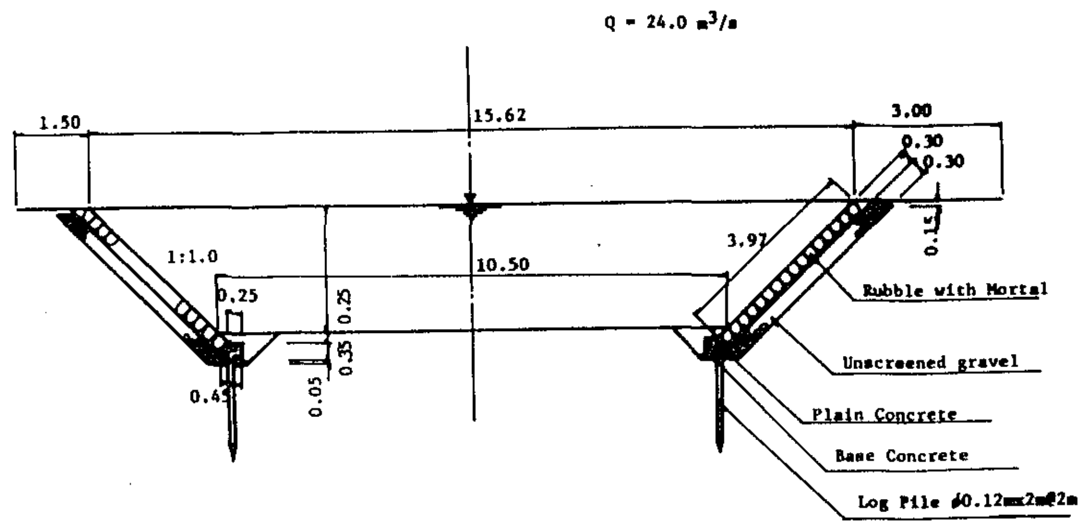
Fig. 3-14 (2) STANDARD STRUCTURE OF DRAINAGE CHANNEL (URGENT PLAN)

Unit: m

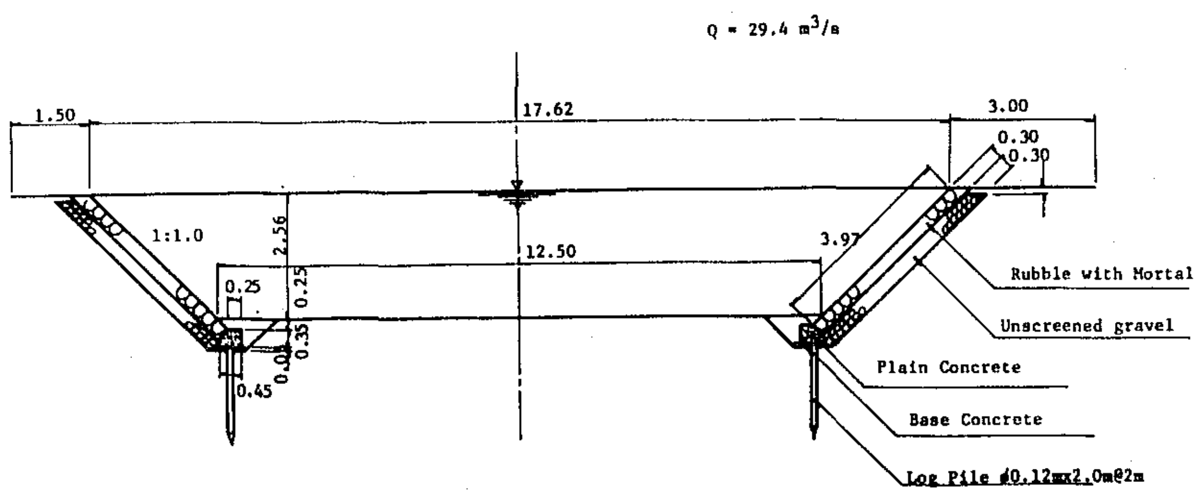
JONGAYA ROUTE
No. H_3 to No. H_{11}
(TYPE D)



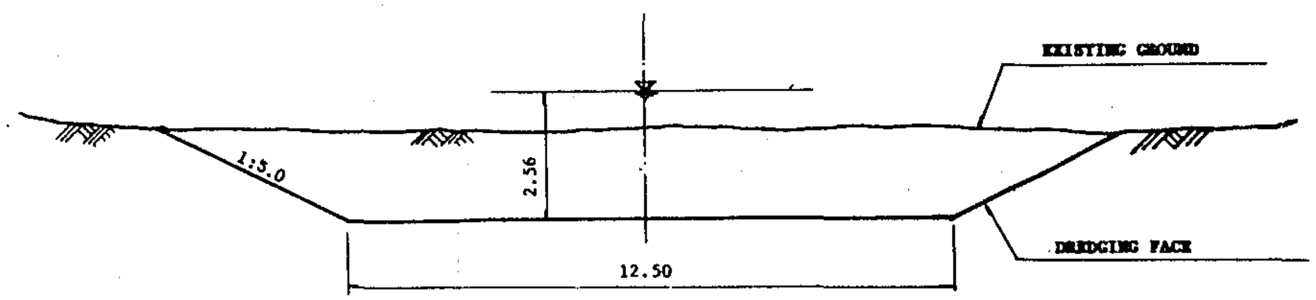
JONGAYA ROUTE
No. H_{11} to No. H_{18}
(TYPE A)



JONGAYA ROUTE
No.H_{18} to No.2.4
(TYPE C)



OLD JENEBERANG
NO. 0.0 to NO. 2.4



VII CONSTRUCTION PLANNING

C O N T E N T S

LIST OF TABLES

LIST OF FIGURES

1. GENERAL

Urgent Flood Control Project consists of the channel improvement of the Jeneberang river and improvement of the drainage system of the city area of Ujung Pandang. The construction work of the project is scheduled for five years, commencing in 1981, including one year for preparation and detailed engineering work.

The reason why the construction work period was scheduled for four years is that the last year, 1985, of the scheduled construction period coincides with the year of completion of the First Development Stage of Urban Development Plan, and also that the residual value of the machinery and equipment to be used in the construction work can be minimized.

The river and drainage system improvement work consist mainly of earthwork, which is scheduled in 7 months of a dry season from April to October, since the earthwork in a rainy season has much difficulties to be conducted (refer to Fig. 2-9 in this Supporting Report "Hydrology").

2. RIVER IMPROVEMENT WORK

2.1 Work Order

The river improvement work consists of dredging and excavation as well as construction of revetment, groyne, sluice and other structures. In the Jeneberang river, dredging and excavation works will be excuted from down toward up-stream along the river course in the stretch from the estuary to the Sungguminasa bridge. The river channel will be dredged in the stretch from estuary to 2.4 K, and excavation above 2.4 K. Since the water depth is enough to dredge between estuary and 2.4 K, the dredger work is viable only in the above mentioned stretch.

Of the above mentioned works, the earthwork is the greatest in volume. Therefore, the critical path is determined by the progress of earthwork. And other works should be scheduled in accordance with the earthwork schedule.

Since various kinds of heavy machinery will be used for earthwork, the earthwork volume in the first year of construction period is made less than those of other years, considering the training period of operators.

The work sections of the urgent river improvement are presented in Table 2-1 and illustrated in Fig. 2-1, and its construction schedule is shown in Fig. 2-2. The breakdown of the total volume is given in Table 2-2 on annual basis.

The improvement work will be divided into four work sections.

The construction work of the dike will be performed from the down stream up on the right bank first then on the left bank so that a bad influence may not be exerted upon the project area, especially upon Ujung Pandang city.

Only after the completion of excavation of the low water channel, construction of the low water revetment and groyne will begin. Immediately upon completion of the embankment, construction of high water revetment and sodding will be started.

The new sluices are to be constructed prior to the embankment.

The road raising work of Jl. Malino located in the upper reaches of the Sungguminasa bridge is scheduled to be executed on the final year of the proposed construction period.

2.2 Selection and Combination of Major Construction Equipment

Conditions

1) Annual working days

Annual working days, excepting average 4 holidays per month, are as follows.

7 months x 26 days = 182 days

2) Annual work volume

About 50% of the excavation work is land excavation by bulldozers and the remaining is underwater excavation by back hoes.

The annual volume of each work are given below.

Dredging :	79,200 m^3
Land Excavation :	79,500 m^3
Underwater Excavation :	79,500 m^3
Embankment :	26,500 m^3

Major Construction Equipment

Selection of construction equipment is based on the annual maximum work volume of the respective work item. The major construction equipment required for the river improvement is listed below.

Dredging :	Dredger
Excavation :	Bulldozers
Excavation and Loading :	Back Hoes
Loading :	Wheel Loaders
Transportation :	Dump Trucks
Spreading and Compaction :	Bulldozers

Calculation of Daily Job

1) Dredging work

A dredger, wheel loaders and dump trucks are to be used

$$\frac{79{,}200 \text{ m}^3}{182 \text{ days}} = 440 \text{ m}^3/\text{day}$$

2) Underwater excavation work

Back hoes and dump trucks are to be used

$$\frac{79{,}500 \text{ m}^3}{182 \text{ days}} = 440 \text{ m}^3/\text{day}$$

3) Land excavation work

Bulldozers, wheel loaders and dump trucks are to be used

$$\frac{79{,}500 \text{ m}^3}{182 \text{ days}} = 440 \text{ m}^3/\text{day}$$

4) Embankment work

Bulldozers, wheel loaders and dump trucks are to be used

$$\frac{26{,}500 \text{ m}^3}{182 \text{ days}} = 146 \text{ m}^3/\text{day}$$

Number of Construction Equipment

1) Dredging work

Dredging work is to be performed by a pump dredger, 520PS type, since the riverbed is sandy. The dredged sand will be conveyed from the dredger to the river work through a pipe (ϕ 300 mm) which will be floated on the river water. The sand will be loaded by a wheel loader, 2.0 m^3 type, and transported by dump trucks, 8-ton type.

- Pump-Dredger, 520PS type

Capacity of a dredger is fixed at 90 m^3/hr. Assuming that of the dredger is operated for 6 hours per day, the required number of units will be;

$$\frac{440}{6 \times 90} = 0.8$$

A practical approximation is 1 unit.

- Wheel Loader, $2.0m^3$ type

Work volume per hour will be;

$$Q = \frac{3,600 \times q \times E}{Cm}$$

where,

Q	:	capacity (m^3/hr)
q	:	loading volume per cycle (m^3)
E	:	job efficiency
Cm	:	time per cycle (sec)

therefore,

$$Q = \frac{3,600 \times 1.6 \times 0.65}{42}$$

$$= 89.1\ m^3/hr$$

Assuming that the time of daily operation is 6 hours, the number of required units will be;

$$\frac{440}{6 \times 89.1} = 0.8$$

A practical approximation is 1 unit.

- Dump truck, 8-ton type

Work volume per hour will be;

$$Q = \frac{60 \times q \times E}{Cm}$$

$$Cm = \frac{5}{1,000} L + 10$$

where,

Q	:	capacity (m^3/hr)
q	:	loading capacity (m^3)
E	:	job efficiency
Cm	:	time per cycle (min)
L	:	transporting distance (m)

applying L = 1,000 m, q = 4.4 m^3 and E = 0.9,

$$Cm = \frac{5}{1,000} \times 1,000 + 10 = 15 \text{ min}$$

$$Q = \frac{60 \times 4.4 \times 0.9}{15} = 15.8\ m^3/hr$$

Assuming that the daily operation is of 6 hours, the number of required units will be;

$$\frac{440}{6 \times 15.8} = 4.6$$

A practical approximation is 5 units.

2) Underwater excavation work

Excavation and loading will be performed by back hoes, $1.2m^3$ type, and transportation by dump trucks, 8-ton type.

- Back hoe, $1.2m^3$ type

Work volume per hour will be;

$$Q = \frac{3,600 \times q \times E}{Cm}$$

where,

Q : capacity (m^3/hr)
q : excavation and loading volume per cycle (m^3)
E : job efficiency
Cm : time per cycle (sec)

applying $q = 1.06\ m^3$ and $E = 0.4$,

work volume per hour will be;

$$Q = \frac{3,600 \times 1.06 \times 0.4}{36}$$

$$= 42.4\ m^3/hr$$

Assuming that the time of daily operation is 6 hours, the required number of units will be

$$\frac{440}{6 \times 42.4} = 1.7$$

A practical approximation is 2 units.

- Dump truck, 8-ton type

Work volume per hour will be;

$$Q = \frac{60 \times q \times E}{Cm}$$

$$Cm = \frac{25}{6,000} (L - 1,000) + 15$$

where,

Q : capacity (m^3/hr)
q : loading volume per cycle (m^3)
E : job efficiency
Cm : time per cycle (min)
L : transporting distance (m)

applying L = 6,000 m, q = 4.4 m^3 and E = 0.9,

$$Cm = \frac{25}{6,000} (6,000 - 1,000) + 15$$

$$= 35.8 \text{ min}$$

$$Q = \frac{60 \times 4.4 \times 0.9}{35.8} = 6.6 \text{ m}^3/\text{hr}$$

Assuming that the time of daily operation is 6 hours, the required number of units will be;

$$\frac{440}{6 \times 6.6} = 11.1$$

A practical approximation is 11 units.

3) Land excavation work

Soil excavation and hauling will be performed by bulldozers, 21-ton type, loading by wheel loaders, 2.0m^3 type and transportation by dump trucks, 8-ton type.

- Bulldozer, 21-ton type

Work volume per hour will be;

$$Q = \frac{60 \times q \times E}{Cm}$$

$$Cm = \frac{0.37}{10} L + 0.25$$

where,

Q : capacity (m^3/hr)
q : loading volume per cycle (m^3)
E : job efficiency
Cm : time per cycle (min)
L : hauling distance (m)

applying $L = 50$ m, $q = 2.8$ m^3 and $E = 0.5$,

$$Cm = \frac{0.37}{10} \times 50 + 0.25$$

$$= 2.1 \text{ min}$$

$$Q = \frac{60 \times 2.8 \times 0.5}{2.1}$$

$$= 40.0 \text{ m}^3/\text{hr}$$

Assuming that the time of daily operation is 6 hours, the required number of units will be

$$\frac{440}{6 \times 40} = 1.8$$

A practical approximation is 2 units.

- Wheel loader 2.0m^3 type

Work volume per hour will be;

$$Q = \frac{3{,}600 \times q \times E}{Cm}$$

where,

Q	:	capacity (m^3/hr)
q	:	loading volume per cycle (m^3)
E	:	job efficiency
Cm	:	time per cycle (sec)

applying $q = 1.6$ m^3 and $E = 0.65$,

$$Q = \frac{3{,}600 \times 1.6 \times 0.65}{42} = 89.1 \text{ m}^3/\text{hr}$$

Assuming that time of daily operation is 6 hours, the required number of unit will be;

$$\frac{440}{6 \times 89.1} = 0.8$$

A practical approximation is 1 unit.

- Dump truck, 8-ton type

Work volume per hour will be;

$$Q = \frac{60 \times q \times E}{Cm}$$

$$Cm = \frac{25}{6{,}000} (L - 1{,}000) + 15$$

where,

Q	:	capacity (m^3/hr)
q	:	loading volume per cycle (m^3)
E	:	job efficiency
Cm	:	time per cycle (min)
L	:	transporting distance (m)

applying L = 6,000m, q = 4.4 m^3 and E = 0.9,

$$Cm = \frac{25}{6,000}(6,000 - 1,000) + 15 = 35.8 \text{ min}$$

$$Q = \frac{60 \times 4.4 \times 0.9}{35.8} = 6.6 \text{ m}^3\text{/hr}$$

Assuming that the daily operation time is 6 hours, the required number of units will be;

$$\frac{440}{6 \times 6.6} = 11.1$$

A practical approximation is 11 units.

4) Embankment work

Since the sand and soil obtained from dredging and excavation of the Jeneberang river are not suitable for banking material, the materials specially excavated at the borrow pit will be used.
The equipment consist of bulldozers, 21-ton type (for excavation), loaders, 1.0m^3 type (for loading) and dump trucks, 8-ton type (for transportation to the construction site).

Spreading and compaction of embankment materials will be performed by bulldozers, 11-ton type.

- Bulldozer, 21-ton type (excavation)

Work volume per hour will be;

$$Q = \frac{60 \times q \times E}{Cm}$$

$$Cm = \frac{0.37}{10} L + 0.25$$

where,

Q	:	capacity (m^3/hr)
q	:	loading volume per cycle (m^3)
E	:	job efficiency
Cm	:	time per cycle (min)
L	:	hauling distance (m)

applying L = 30 m, $q = 2.8\ m^3$ and E = 0.55,

$$Cm = \frac{0.37}{10} \times 30 + 0.25$$

$$= 1.36 \text{ min}$$

$$Q = \frac{60 \times 2.8 \times 0.55}{1.36}$$

$$= 67.9\ m^3/hr$$

Assuming that the time of daily operation is 6 hours, the required number of units will be;

$$\frac{146}{6 \times 67.9} = 0.4$$

A practical approximation is 1 unit.

- Wheel loader, $1.2m^3$ type

Work volume per hour will be;

$$Q = \frac{3600 \times q \times E}{Cm}$$

where,

Q : capacity (m^3/hr)
q : loading volume per cycle (m^3)
E : job efficiency
Cm : time per cycle (sec)

applying $q = 0.96\ m^3$, E = 0.65 and Cm = 42 sec,

$$Q = \frac{3600 \times 0.96 \times 0.65}{42} = 53.5\ m^3/hr$$

Assuming that the daily operation time is 6 hours, the required number of unit will be;

$$\frac{146}{6 \times 53.5} = 0.5$$

A practical approximation is 1 unit.

- Dump truck, 8-ton type

Work volume per hour will be;

$$Q = \frac{60 \times q \times E}{Cm}$$

$$Cm = \frac{25}{6,000} (L - 1,000) + 15$$

where,

- Q : capacity (m^3/hr)
- q : loading volume per cycle (m^3)
- E : job efficiency
- Cm : time per cycle (min)
- L : transporting distance (m)

applying $L = 6,000$ m, $q = 4.4$ m^3 and $E = 0.9$,

$$Cm = \frac{25}{6,000} (6,000 - 1,000) + 15 = 35.8 \text{ min}$$

$$Q = \frac{60 \times 4.4 \times 0.9}{35.8} = 6.6 \text{ m}^3/\text{hr}$$

Assuming that the daily operation time is 6 hours, the required number of units will be;

$$\frac{146}{6 \times 6.6} = 3.7$$

A practical approximation is 4 units.

- Bulldozer 11-ton type (for spreading and compaction)

Spreading : $Q_1 = 10E \times (11D + 8)$

Compaction : $Q_2 = \frac{V \times W \times D \times E}{N}$

Spreading and Compaction :

$$Q = \frac{Q_1 \times Q_2}{Q_1 + Q_2}$$

where,

- Q_1 : spreading (m^3/hr)
- Q_2 : compaction (m^3/hr)
- Q : spreading and compaction (m^3/hr)
- D : thickness of finishing layer (m)
- V : speed of compaction (m/hr)
- W : width of compaction (m)
- N : number of compaction
- E_1 : job efficiency
- E_2 : job efficiency

applying E_1 = 0.65 and D = 0.3 m,

$$Q_1 = 10 \times 0.65 \times (11 \times 0.3 + 8) = 73.5 \ m^3/hr$$

also applying V = 4,000 m/hr, W = 0.6 m, D = 0.8 m, E_2 = 0.7 and N = 5,

$$Q_2 = \frac{4{,}000 \times 0.6 \times 0.3 \times 0.7}{5}$$

$$= 100.8 \ m^3/hr$$

$$Q = \frac{73.5 \times 100.8}{73.5 + 100.8}$$

$$= 42.5 \ m^3/hr$$

Assuming that the daily operation time is 6 hours, the required number of unit will be;

$$\frac{146}{6 \times 42.5} = 0.6$$

A practical approximation is 1 unit.

3. DRAINAGE SYSTEM IMPROVEMENT WORK

3.1 Work Order

The drainage system improvement work includes excavation and dredging as well as construction of revetment, bridges and a sluice.

Excavation and revetment works are major items in the subject work. The construction schedule of the earthwork and revetment work should be fixed first, because their progress will be the critical path of the construction work. All other related work items will have to be scheduled in accordance with the earthwork and revetment work schedule.

As shown in Fig. 3-1 and Table 3-1, the drainage system construction work will be divided into 4 sections.

Construction of the drainage system works will be conducted in a dry season from April to October, except for dredging work of the old Jeneberang river which is a part of the Jongaya route. A dredger to be used for dredging of the Jeneberang river in a dry season will be operated in a rainy season to dredge the old Jeneberang river because there is no fear of flood even in the rainy season.

Since the damage reduction can be expected after improvement of the Panampu and Jongaya channels, the works will be carried out in the order of Panampu, Jongaya and Sinrijala routes. Consequently, the improvement work of Sinrijala channel will be scheduled in the last year, and its work volume is smaller than those of other years. The work periods are 2 years, 2 years and 1 year respectively. The proposed constructiuon schedule is shown in Fig. 3-2.

The breakdown of the total work volume of urgent drainage system improvement is shown in Table 3-2.

3.2 Selection and Combination of Major Construction Equipment

<u>Conditions</u>

1) Annual working days

Annual working days, excepting average 4 holidays per month, are as follows:

7 months x 26 days = 182 days

2) Annual work volume

A construction work schedule in terms of selection of construction equipment is based on an annual maximum work volume of the respective work items.

Dredging :	83,000 m^3
Spoil of Dredging Work :	83,000 m^3
Excavation :	134,000 m^3
Filling :	5,000 m^3
Spoil of Excavation Work :	129,100 m^3

<u>Major Construction Equipment</u>

In the construction work, the construction equipment as listed below are scheduled to be used.

Dredging :	Dredger
Excavation and loading :	Back Hoe
Transportation :	Dump Truck
Spreading and compaction :	Bulldozer

<u>Calculation of Daily Job</u>

1) Dredger (Dredging)

$$\frac{83,000\ m^3}{3\ \text{months} \times 26\ \text{days}} = 1,060\ m^3/\text{day}$$

2) Back Hoe (Excavation and loading)

$$\frac{134,000\ m^3}{7\ \text{months} \times 26\ \text{days}} = 740\ m^3/\text{day}$$

3) Dump Truck (Transportation)

$$\frac{129,100\ m^3}{7\ \text{months} \times 26\ \text{days}} = 710\ m^3/\text{day}$$

4) Bulldozer (Spreading and compaction)

$$\frac{129,100 \text{ m}^3}{7 \text{ months x } 26 \text{ days}} = 710 \text{ m}^3/\text{day}$$

Number of Construction Equipment

The old Janeberang river section in the Jongaya route is to be dredged by a dredger.

All other sections are excavated by back hoes, capacity 0.6m^3 each, and soil is transported to the spoil-bank by dump trucks, 8-ton type, except for soil to be used for filling.

Soil dumped at the spoil-bank is to be spread and compacted by bulldozers, 11-ton type.

1) Dredger

The dredger to be used are of 140 m^3/hr in rated capacity. Assuming 8-hour operation per day, the required number of dredgers will be;

$$\frac{1,060 \text{ m}^3/ \text{ day}}{8 \text{ hours x } 140 \text{ m}^3/\text{hr}} = 0.9$$

A practical approximation is 1 unit.

2) Back Hoe, 0.6m^3 type

Work volume per hour will be;

$$Q = \frac{3,600 \times q \times E}{Cm}$$

where,

Q	:	Capacity per hour (m^3/hr)
q	:	excavation per cycle (m^3)
E	:	job efficiency
Cm	:	time per cycle (sec)

applying $q = 0.53 \text{ m}^3$, $E = 0.6$ and $Cm = 36$ sec,

$$Q = \frac{3,600 \times 0.53 \times 0.6}{36} = 31.8 \text{ m}^3/\text{hr}$$

Assuming that the time of daily operation is 6 hours, the required number of units is;

$$\frac{740}{6 \times 31.8} = 3.9$$

A practical approximation is 4 units.

3) Dump Truck, 8-ton type

Work volume per hour will be;

$$Q = \frac{60 \times q \times E}{Cm}$$

$$Cm = \frac{25}{6,000}(L - 1,000) + 15$$

where,

- q : loading capacity (m^3)
- E : job efficiency
- Cm : time per cycle (sec)
- L : transporting distance (m)

applying L = 5,000 m, q = 4.4 m^3 and E = 0.9,

$$Cm = \frac{25}{6,000}(5,000 - 1,000) + 15 = 31.7 \text{ (sec)}$$

$$Q = \frac{60 \times 4.4 \times 0.9}{31.7} = 7.5 \ m^3/hr$$

Assuming that the daily operation time is 6 hours, the required number of units is;

$$\frac{710}{6 \times 7.5} = 15.8$$

A practical approximation is 16 units.

4) Bulldozer, 11-ton type

Work volume per hour will be;

Spreading : $Q_1 = 10E \times (11D + q)$

Compaction : $Q_2 = \frac{V \times W \times D \times E}{N}$

Spreading and Compaction : $Q = \frac{Q_1 \times Q_2}{Q_1 + Q_2}$

where,

- Q1 : spreading (m^3/hr)
- Q_2 : compaction (m^3/hr)
- Q : spreading and compaction (m^3/hr)
- D : thickness of finishing layer (m)
- V : speed of compaction (m/hr)
- W : width of compaction
- N : number of compaction
- E_1 : job efficiency
- E_2 : job efficiency

applying $D = 0.3$ m, $E_1 = 0.65$, $V = 4{,}000$ m/hr, $W = 0.6$ m and $E_2 = 0.7$,

$$Q_1 = 10 \times 0.65 \; (11 \times 0.3 + 8) = 73.5 \text{ m}^3/\text{hr}$$

$$Q_2 = \frac{4{,}000 \times 0.6 \times 0.3 \times 0.7}{5} = 100.8 \text{ m}^3/\text{hr}$$

$$Q = \frac{73.5 \times 100.8}{73.5 + 100.8} = 42.5 \text{ m}^3/\text{hr}$$

Assuming that the time of daily operation is 6 hours, the required number of units is;

$$\frac{710}{6 \times 42.5} = 2.8$$

A practical approximation is 3 units.

4. CONSTRUCTION MATERIALS AND EQUIPMENT

Cement

Tonasa cement and Fortune cement, both of which are produced in Indonesia, are procurable in Ujung Pandang city and available in both quantity and quality.

Aggregates

Aggregates necessary for the construction consists of sand, gravels and cobbles. Sand, gravels and cobbles may be obtained in the middle and upper reaches of the river.

Log Piles, Sod and Steel Gate

Log piles and sod obtainable near the project site are also adequte for the project. Steel sluice gates to be employed in the project can be produced in Indonesia.

Banking Materials

Banking materials are obtained from the borrow pit. The embankment volume is estimated at 111,500 m^3. River bed materials from dredging are in general for embankment. However, the riverbed materials of the Jeneberang river con sists of fine sand of a uniform grain size, which is not suitable for embankment due to excessive permeability. Compaction can hardly be achieved to a satisfactory degree because of fine sand. Accordingly, a borrow-pit will be required to provide banking materials. The hilly land located at 2 km north-east to the Sungguminasa bridge is recommended as a borrow pit (refer to Fig. 5-5). The available volume is estimated at 120,000 m^3.

The quantity of construction materials is shown in Tables 4-1 and 4-2.

Equipment

The construction equipment to be purchased from abroad is shown in Table 4-3.

Table 2-1 URGENT RIVER IMPROVEMENT WORK SECTION

	Item	Work Section
2nd Yr.	Dredging	0.0K JL -- 0.3K JL
	Excavation	2.4K JL -- 3.1K JL
	Embankment and Sodding	
	(Right Bank)	2.6K -- 6.0K
	Revetment	
	High Water Channel	
	(Right Bank)	5.3K -- 6.0K
3rd Yr.	Dredging	0.3K JL -- 1.0K JL
	Excavation	3.1K JL -- 5.6K
	Embankment and Sodding	
	(Right Bank)	6.0K -- JB
	Revetment	
	High Water Channel	
	(Right Bank)	6.0K -- 7.6K
	(Right Bank)	near to the Jeneberang bridge
	Low Water Channel	
	(Left Bank)	4.4K -- 5.6K
	Sluice (Right Bank)	near to the point 7.6K
4th Yr.	Dredging	1.0K JL -- 1.9K JL
	Excavation	5.6K -- 7.2K
	Embankment and Sodding	
	(Left Bank)	2.K JL -- 5.0K
	Revetment	
	High Water Channel	
	(Left Bank)	4.4K -- 5.0K
	Low Water Channel	
	(Right Bank)	5.6K -- 7.2K
	(Left Bank)	5.6K -- 6.4K
	Sluice (Left Bank)	near to the point 5.4K and 6.6K
	Groyne	Right 6.0K -- 7.0K
5th Yr.	Dredging	1.9K JL -- 2.4K JL
	Excavation	7.2K -- JB
	Embankment and Sodding	
	(Left Bank)	5.0K -- JB
	Revetment	
	High Water Channel	
	(Left Bank)	near to the Sungguminasa Bridge
	Low Water Channel	
	(Left and Right Banks)	near to the Sungguminasa Bridge

Table 2-2 BREAKDOWN OF THE TOTAL WORK VOLUME
(URGENT RIVER IMPROVEMENT)

ITEM	UNIT	2nd Yr.	3rd Yr.	4th Yr.	5th Yr	TOTAL
Dredging	m^3	39,600	79,200	72,600	72,600	264,000
Excavation						
Overland	m^3	39,800	79,500	73,000	72,700	265,000
Under Water	m^3	39,800	79,500	73,000	72,700	265,000
Embankment	m^3	16,600	26,500	24,600	27,800	95,500
Sodding	m^2	11,700	21,500	25,400	24,400	83,000
Revetment						
High-water	m	700	2,300	600	2,100	5,700
Low-water	m	-	1,300	2,600	1,800	5,700
Groyne	PC	-	-	23	-	23
Sluice(1.5x1.5m)	PC	-	-	-	2	2
Sluice(1.1x1.1m)	PC	-	1	-	-	1
Drainage Ditch	m^2	-	600	-	2,200	2,800
Land Aquisition	m^2	9,000	14,300	4,300	27,700	55,300
House Evacuation	PC	20	20	10	10	60
Road Raising	m	-	-	-	2,950	2,950

Table 3-1 URGENT DRAINAGE SYSTEM IMPROVEMENT WORK SECTION

	Item	Work Section
2nd Yr.	Dredging: Jongaya Route	No. 0.0 - No. 1.0
	Excavation, Revetment: Panampu Route	No. 0.0 - No. 3.0
	Bridge: Panampu Route	No. 1 - No. 5
3rd Yr.	Dredging: Jongaya Route	No. 1.0 - No. 2.4
	Excavation, Revetment: Panampu Route Jongaya Route	 No. 3.0 - No. H3 No. 2.4 - No. H18
	Bridge: Panampu Route Jongaya Route	 No. 6 - No. 10 No. 16 - No. 20
4th Yr.	Excavation, Revetment Jongaya Route	No. H18 - No. H3
	Bridge: Jongaya Route	No. 11 - No. 15
5th Yr.	Excavation, Revetment Sinrijala Route	No. 0.0 - No. P1
	Bridge, Sluice: Sinrijala Route	No. 21 - No. 23

Table 3-2 BREAKDOWN OF THE TOTAL WORK VOLUME (URGENT DRAINAGE SYSTEM IMPROVEMENT)

ITEM	UNIT	2nd Yr.	3rd Yr.	4th Yr.	5th Yr.	TOTAL
Excavation	m^3	119,200	134,000	116,000	30,000	399,200
Dredging	m^3	55,400	83,000	-	-	138,400
Filling	m^3	4,000	2,100	2,800	10,600	19,500
Backfill	m^3	2,600	2,800	2,800	2,300	10,500
Revetment	m^3	23,000	25,000	24,700	21,300	94,000
Bridge	PC	5	10	5	2	22
Sluice	PC	-	-	-	1	1
Land Aquisition	m^2	35,800	52,000	50,800	12,800	151,400
House Evacuation	PC	140	150	80	-	370

Table 4-1 CONSTRUCTION MATERIALS FOR RIVER IMPROVEMENT WORKS

1.	Concrete (for revetment and sluice)	7,000 m^3
2.	Cement	2,200 tons
3.	Reinforcement bar (for sluices)	8 tons
4.	Steel sluice gate	4 tons
5.	Bridge (for sluices)	3 tons
6.	Wire (for revetment and groyne)	170 tons
7.	Sand (for concrete)	3,000 m^3
8.	Gravel (for revetment)	8,000 m^3
9.	Rubble (for revetment)	2,000 m^3
10.	Boulder (for groyne)	9,700 m^3
11.	Log pile, ϕ 0.15 m L=5.0 m (for groyne)	1,500 pcs
12.	Log pile, ϕ 0.12 m L=2.0 m (for revetment)	2,900 pcs
13.	Log ϕ 0.10 m L=6.0 m	1,000 pcs

Table 4-2 CONSTRUCTION MATERIALS FOR DRAINAGE SYSTEM IMPROVEMENT WORKS

1.	Concrete	10,500 m^3
2.	Cement	3,200 tons
3.	Reinforcement bar	460 tons
4.	Gravel	34,000 m^3
5.	Rubble	28,200 m^3
6.	Sand	4,400 m^3
7.	Log pile, ϕ 0.15 m L=2.0 m	11,800 pcs
8.	Concrete pile	760 pcs
9.	Steel sluice gate	8 tons

Table 4-3 CONSTRUCTION EQUIPMENT TO BE PURCHASED

Equipment	Capacity	Unit
Drainage System Improvement		
Bulldozer	11 ton	3
Back Hoe	0.6 m^3	4
Dump Truck	8 ton	16
Pile Driver	2.5 ton Ram	1
Tamper	80 kg	1
River Improvement		
Dredger	520 PS	1
Anchor Barge	30 PS	1
Wheel Loader	1.2 m^3	1
Wheel Loader	2.0 m3	2
Bulldozer	11 ton	1
Bulldozer	21 ton	3
Back Hoe	1.2 m^3	2
Dump Truck	8 ton	31
Vibrating Roller	2.5 ton	1
Soil Compactor	90 kg	1
Tamper	80 kg	1
Road Roller	11-12 ton	1
Tire Roller	8-20 ton	1
Asphalt Engine Sprayer	200 ℓ	1
Asphalt Finisher	2.5 m class	1

Fig. 2-1 RIVER IMPROVEMENT WORK SECTION

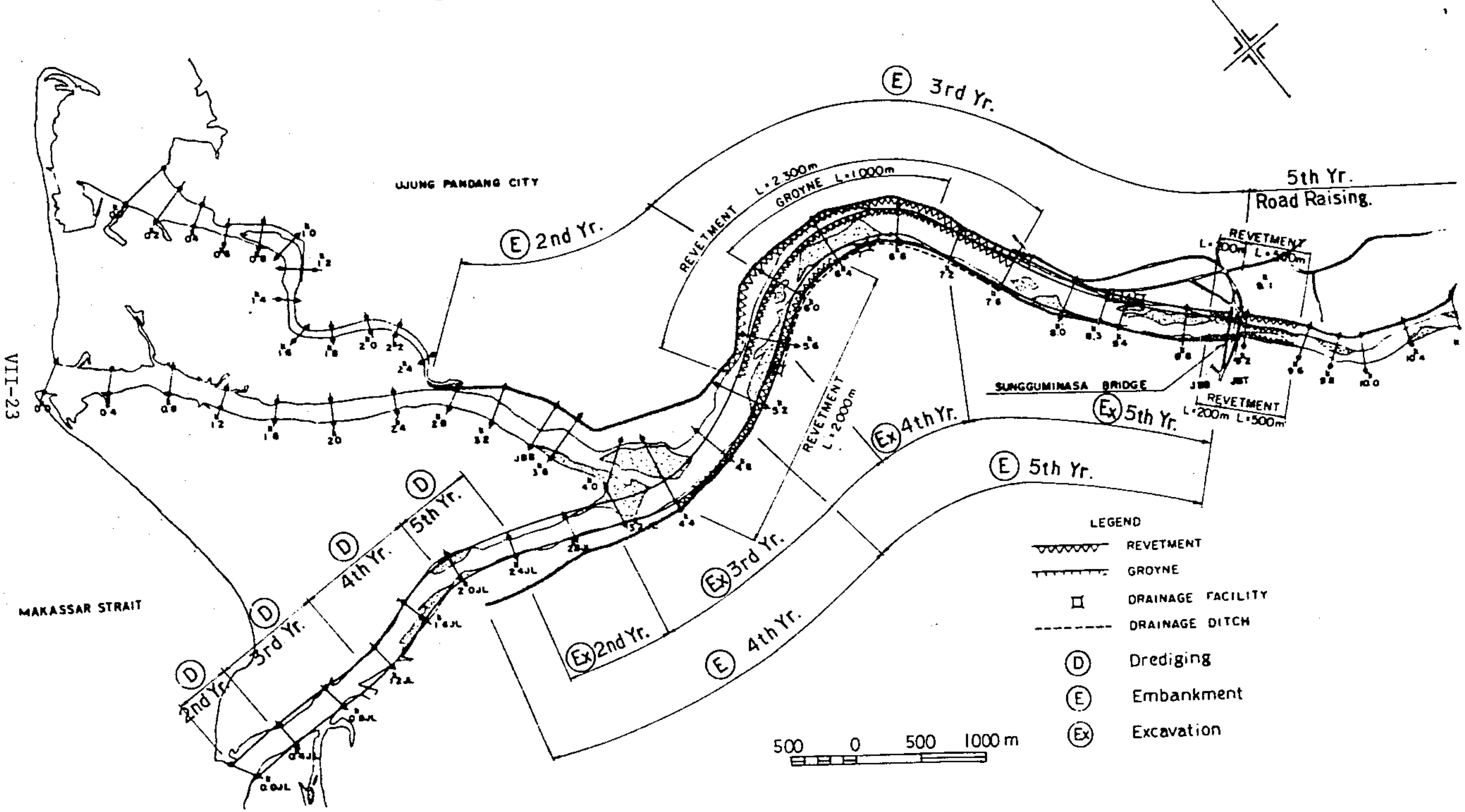


Fig. 2-2 CONSTRUCTION SCHEDULE (RIVER IMPROVEMENT)

WORK			VOLUME	1st Yr.	2nd Yr.	3rd Yr.	4th Yr.	5th Yr.
PREPARATION			(LUMP SUM)					
DETAILED ENGINEERING								
EARTH WORK	DREDGING		264,000 m^3		39,600	79,200	72,600	72,600
EARTH WORK	OVERLAND EXCAVATION		265,000 m^3		39,800	79,500	73,000	72,700
EARTH WORK	UNDERWATER EXCAVATION		265,000 m^3		39,800	79,500	73,000	72,700
EARTH WORK	EMBANKMENT		95,500 m^3		16,600	26,500	24,600	27,800
SODDING			83,000 m^2		11,700	21,500	25,400	24,400
REVETMENT	HIGH WATER		5,700 m		700	2,300	600	2,100
REVETMENT	LOW WATER					1,300	2,600	1,800
GROYNE			23 PCS				23	
SLUICE			3 PCS			1		2
ROAD RAISING			2,950 m					2,950
EARTH WORK PROGRESS			0 – 250,000 m^3		19,400 m^3/mon	37,800 m^3/mon	34,700 m^3/mon	35,100 m^3/mon
MACHINERY	DREDGER				1	1	1	1
MACHINERY	WHEEL LOADER	2.0 m^3			1	2	2	2
MACHINERY	WHEEL LOADER	1.2 m^3			1	1	1	1
MACHINERY	BACK HOE	1.2 m^3			1	2	2	2
MACHINERY	BULL DOZER	21 ton			2	3	3	3
MACHINERY	BULL DOZER	11 ton			1	1	1	1
MACHINERY	DUMP TRUCK	8 ton			18	31	31	31
MACHINERY	VIBRATING ROLLER	2.5 ton			1	1	1	1
LABOR	EARTH WORK	OPERATION			26	42	42	42
LABOR	EARTH WORK	MAINTENANCE			3	5	5	5
LABOR	EARTH WORK	MISC			20	25	20	20
LABOR	REVETMENT GROYNE	MASON			15	130	160	150
LABOR	REVETMENT GROYNE	MISC			30	250	300	300
LABOR	TOTAL				94	452	527	517

Fig.3-1 DRAINAGE CHANNEL WORK SECTION

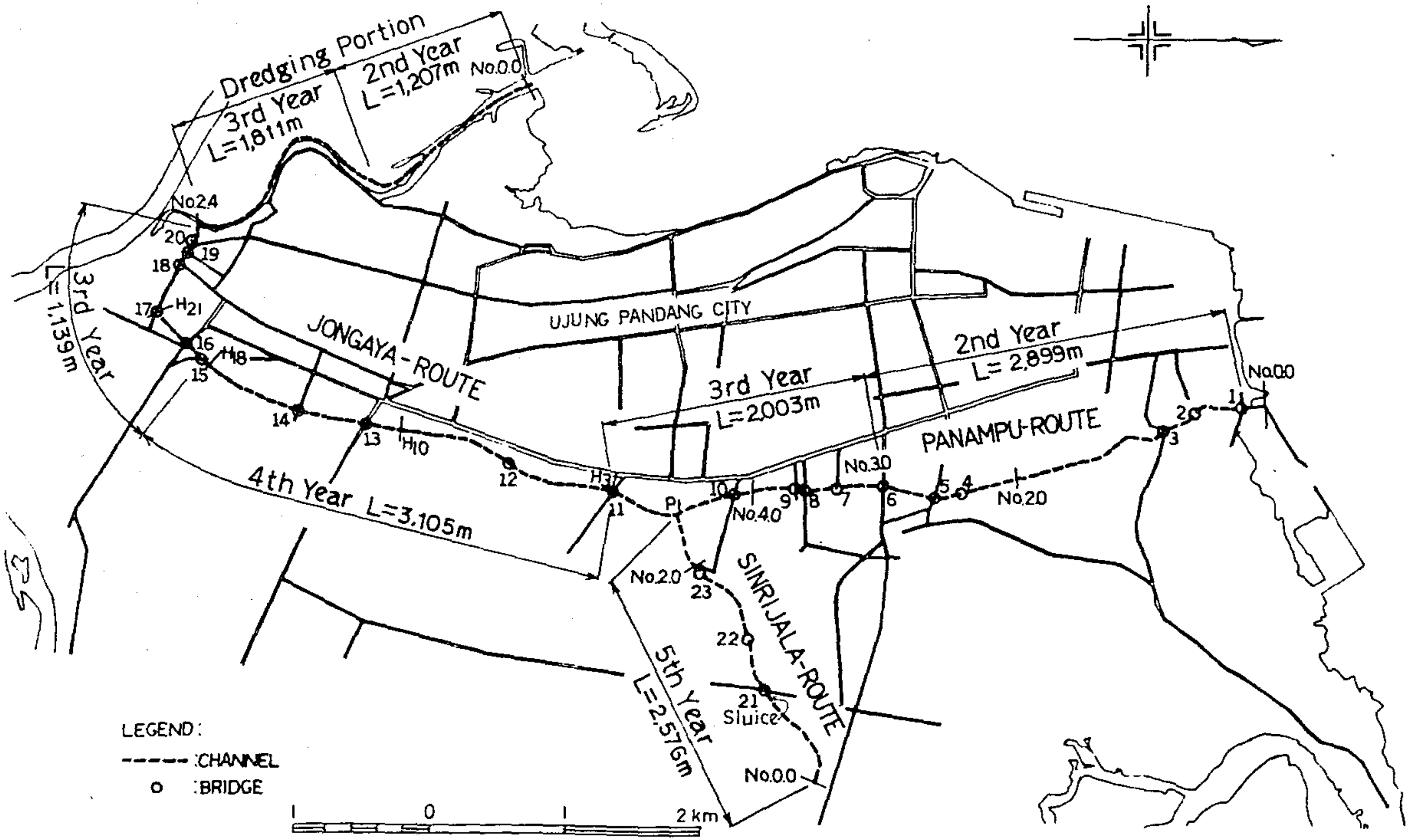
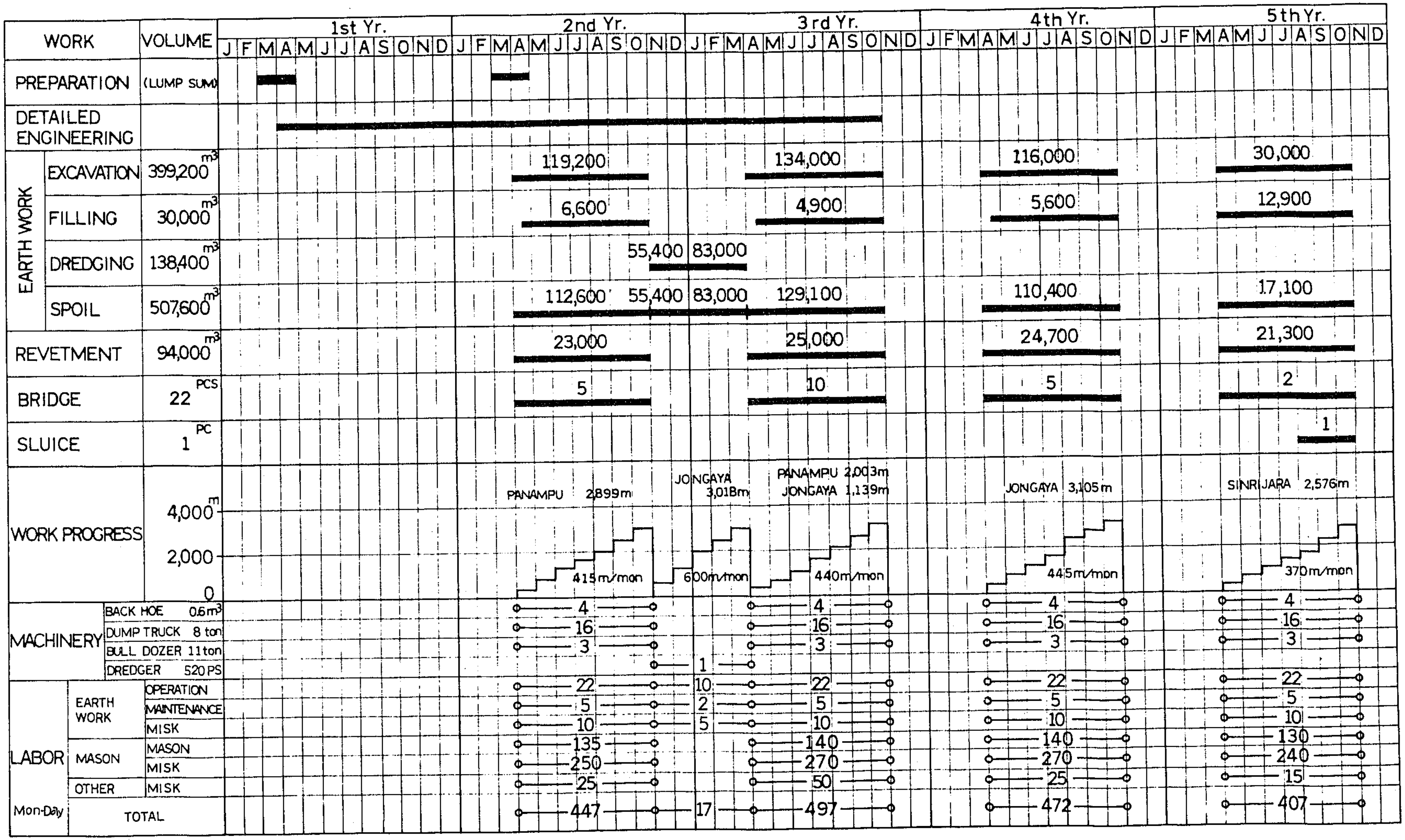


Fig. 3-2 CONSTRUCTION SCHEDULE (DRAINAGE IMPROVEMENT)



VIII PROJECT ECONOMY

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1. GENERAL

The project evaluation is made to ascertain the feasibility of the project in view of economic and socio-economic aspects. The economic feasibility of the project is evaluated by calculating the internal rate of return, assuming that the estimated damages is inflicted on the buildings, household effects and farm crops existing after completion of the first development stage. Sensitivity analysis is also made with respect to change in the economic contruction cost and the completion time of the on-going regional development plan. The fund requirement for the project is estimated on both force account basis and contract basis.

The project area is divided into two areas; namely, city-side area and mountain-side area which are bordered by the Jl. Panakkukang, because the ponding water depths in these two areas are different from each other.

The assets in the inundation area are defined as buildings, household effects and farm crops in this study. The value of the present assets is estimated at 59,000 million Rupiah.

Socio-economic impacts of the project are briefly assessed in due consideration of the effect of the project on the regional development.

Rupiah and Yen are converted to US Dollar at the exchange rates of Rp. 625 = US$1 and ¥ 250 = US$1. The project life for the economic evaluation is 50 years from 1981 to 2030.

2. ASSETS IN THE INUNDATION AREA

2.1 Buildings and Household Effects

The distribution by ground height of the present and future buildings and household effects in each development stage are presented in Table 2-1 in quantitative terms and in Table 2-2 and Fig. 2-1 in monetary terms. The average values of the buildings and household effects are estimated as presented in Table 2-3, based on the research conducted in the project area. The assets after completion of the second and third development stages are presented in these tables for reference, though they are not related to this economic evaluation.

The future assets in the first development stage are estimated in accordance with the plan prepared by P.T. Timurama/1. The estimation of those in the second and third development stages is based on the plan prepared by BIEC International, Inc. (Refer to Fig. 2-2.)

/1 : P.T. Timurama has prepared a plan only for the first development area.

2.2 Farm Crops

Farm crops in the agricultural land of the inundation area consist mainly of paddy. The distribution of present and future agricultural area is researched from the plans prepared by BIEC International, Inc. and P.T. Timurama, and is given in Table 2-4.

The annual paddy yield in Ujung Pandang city is known from the statistics (refer to Table 2-5).

3. PROJECT BENEFIT

Project benefit is the reduction in flood damage brought about by the implementation of this project. The annual benefit is given as the difference of the annual damage which are obtained by multiplying the total damage potential by the probable flood rate.

Flood damage may be defined as the physical deterioration or destruction caused by floodwaters. The term flood loss refers to the net effect of historic flood damage on the regional economy and well-being with the tangible components of the loss being expressed in monetary units. Flood risk is the probable damage, expressed either on per flood event basis or on an average annual basis, that will be incurred as a result of future flooding with the tangible portion of the risk expressed in monetary terms. All losses resulting from historic flooding or the risk attendant to future flooding can be classified into one of three types of damage categories - direct, indirect and trangible - or they can be classified according to whether the private or the public sector incurs the losses or risks. This two-way classsification of flood losses and risks is set forth in Table 3-1.

Direct flood losses or risks were defined as monetary expenditures required, or which would be required, to restore flood-damaged property to its pre-flood condition. This includes the cost of cleaning, repairing, and replacing residential, commercial, industrial, and agricultural buildings and contents and other objects and materials located outside of the buildings. Direct losses and risks also encompass the cost of cleaning, repairing, and replacing roads and bridges, storm water systems, sanitary sewer systems, and other utilities, as well as the cost of restoring damaged park and recreational lands.

Indirect flood losses and risks were defined as the net monetary cost of evacuation, relocation, lost wages, lost production, and lost sales; the increased cost of highway and railroad transportation because of flood-caused detours; the costs of flood fighting and emergency services provided by governmental units, as well as the cost of post-flood floodproofing of individual structures. The costs of post-flood engineering and planning studies and of implementing the structural and non-structural measures recommended by those studies also are categorized as indirect losses and risks.

Although often difficult to determine with accuracy, indirect losses and risks nevertheless constitute a real monetary burden on the economy of the region.

Intangible flood losses and risks were defined as flood effects which cannot be measured in monetary terms. Such losses and risks include loss of life, health hazards, property value depreciation as a result of flooding, and the general disruption of normal community activities. Intangible losses and risks also include severe psychological stress experienced by owners or occupants of riverine area structures.

The direct damages inflicted on buildings and household effects and to farm crops are considered in this study. The indirect damages are also assessed for the evaluation.

Average annual flood damage is determined by summing up the potential direct damage from flood of different frequencies, and the potential indirect damage is estimated by applying an indirect damage factor.

By implementing the urgent flood control project, the project benefit will be expected in city-side and mountain-side areas. Estimation of the total flood damage potential with and without a project is based on the following assumptions.

Buildings and Household Effects

1) Inundation water with a depth of 20 cm or less above the ground level inflicts no damage on houses.

2) Temporary houses and their household effects are free from flood damage, because their floors are elevated at 1.5 m to 2.0 m above the ground level.

3) The damage rates as shown in Table 3-2 are applied for the damage estimation. Those rates are quoted from the report, "Feasibility Study on Surabaya River Improvement" prepared by JICA in 1973.

Farm Crops

1) Flood damage to the farm crops is estimated by only the paddy damages, because the paddy field covers most of the agricultural land in the project area.

2) Crop unit price of Rp.75.40/kg in 1978 is employed for the estimation (refer to Table 3-3).

3) Average yeild per ha is 2.3 t in Ujung Pandang city and 3.2 t in Kabupaten Gowa. Since the flood affected area lies in Ujung Pandang city, the average paddy yield is fixed at 2.3 t per ha.

4) Damage rate is fixed at 20% in consideration of 1) high frequency of flood occurrence in January, 2) cropping pattern and 3) ponding period. (Refer to Table 3-4 and Fig. 3-1.)

5) The cultivated area in January covers 55% of the agricultural land of the project area (refer to Fig. 3-1).

Indirect Damage

1) The indirect damage to buildings and household effects is fixed at 15%.

2) The indirect damage to farm crops is fixed at 20%.

The estimated annual damages to buildings and household effects and to farm crops in the inundation area are summarized in Table 3-6.

Table 3-6 ANNUAL MEAN DAMAGE
(Firest Stage Development)

Unit: $x10^6$ US$

		Without Project	With Project
House and	Direct	1.999	0.250
Household Effects	Indirect	0.300	0.038
Sub-total		2.298	0.288
Farm Crops	Direct	0.034	0.028
	Indirect	0.007	0.006
Sub-total		0.042	0.034
Total		2.340	0.322

The relation between the inundation water stage and the direct damage is given in Table 3-7 and Fig. 3-2.

The annual expected damage reduction is estimated at US$2.02 million, the breakdown of which is set forth in Table 3-8.

4. PROJECT COST

4.1 Economic Construction Cost

Economic construction cost for the project is estimated for the economic evaluation based on the following conditions.

1) The cost estimate is made on the basis of the estimated quantities and volume of the works required for the project in such a manner that the cost shall reasonably reflect social opportunity costs excluding the effects of import duties and subsidies. The 1979 year price level is applied to the estimate.

2) Major construction machinery will be procured form abroad. In the estimate of the machinery cost, only the depreciation cost is included instead of the purchase cost.

3) Cost of the imported equipment and services to be procured by international competitive bidding is estimated based on international price level. The local cost such as materials and labor is estimated taking into account the experience of on-going projects in South Sulawesi and east Jawa.

4) The cost of the cultivated land, yards and houses to be acquired for the implementation of the project is included in the economic construction cost.

5) Physical contingency of the cost estimate is about 10% of the direct cost. Price contingency, or price escalation is excluded in the estimate of the economic construction cost.

Economic construction cost for the drainage system and river improvement is estimated at US$ 11,902,000 equivalent, which consists of US$ 7,539,000 equivalent of local currency and US$ 4,363,000 of foreign currency portion as given below.

Unit: x10^3 US$

Item	Foreign Currency	Local Currency	Total
River Improvement	2,757	3,472	6,229
Drainage Improvement	1,606	4,067	5,673
Total	4,363	7,539	11,902

The breakdown and disbursement of the economic cost are presented in Tables 4-1 and 4-2.

4.2 Operation and Maintenance Cost

Operation and maintenance cost will be required after the completion of the Panampu improvement. The cost will gradually increase in accordance with the progress of the construction of the project. After the completion of the project construction, a unifrom amount for operation and maintenance will be required annually till the end of project life.

The annual cost necessary for the maintenance and operation after completion of the project is estimated at US$ 0.055 million. The breakdown is set forth in Table 4-3.

5. FUND REQUIREMENT FOR CONSTRUCTION

Fund requirement for the project construction is estimated for two different systems of the construction, that is, contract basis and force account basis. The estimate was made at escalation rates of 7% for foreign currency and 10% for local currency.

For the estimate of the fund requirement, actual cost of the land to be acquired for the implementation of the project and the expected cost escalation are included in both construction systems. With respect to the cost of construction machinery, only the depreciation cost is included in the estimate for the contract basis, while all the purchasing cost is included for the force account basis.

Fund requirements for the construction amount to US$ 5,272,300 in foreign currency portion and US$ 12,385,000 in domestic currency portion, provided that the construction works are carried out on contract basis. If the works are implemented on force account basis, the fund requirements amounts to US$ 7,139,000 in foreign currency portion and US$ 12,385,000 in domestic currency portion, which is shown below.

1) Contract basis

Unit: $\times 10^3$ US$

Item	Foreign Currency	Local Currency	Total
River Improvement	3,679	5,905	9,584
Drainage Improvement	2,044	6,480	8,524
Total	5,723	12,385	18,108

2) Force account basis

Unit: $\times 10^3$ US$

Item	Foreign Curreny	Local Currency	Total
River Improvement	4,849	5,906	10,755
Drainage Improvement	2,290	6,479	8,769
Total	7,139	12,385	19,524

The more detailed breakdown and annual disbursement are given in Tables 5-1 and 5-2.

In the case that all the constrution cost is estimated in local currency (excluding foreign engineering service), the fund requirement amounts to US$ 18,502,000 on contract basis and to US$ 19,967,000 on force account basis. The engineering service cost of US$ 1,884,000 is included in the above two cases. The summary of the fund requirement is shown below.

1) Contract basis

Unit: $x10^3$ US$

Item	Foreign Currency	Local Currency	Total
River Improvement	1,211	8,646	9,857
Drainage Improvement	673	7,972	8,645
Total	1,884	16,618	18,502

2) Force account basis

Unit: $x10^3$ US$

Item	Foreign Curreny	Local Currency	Total
River Improvement	1,211	9,841	11,052
Drainage Improvement	673	8,242	8,915
Total	1,884	18,083	19,967

The more detailed breakdown and annual disbursement are set forth in Tables 5-3 and 5-4.

6. EVALUATION

6.1 Internal Rate of Return

Internal rate of return is calculated at 13.2% on the basis of the benefits and costs estimated as above. The internal rate of return indicates the economic soundness of the project.

Although benefit is expected to accrue after the completion of the Panampu channel improvement, this can be disregarded in this study because of its slightness.

The annual disbursement of the costs and benefits are presented in Table 6-1.

6.2 Sensitivity Analysis

Project sensitivity is analyzed with respect to change in the economic construction cost and the completion time of the regional development.

The sensitivity analysis indicates that the project maintains a relatively high internal rate of return of about 12.1% for the case of 10% increase in the construction cost and about 12.2% for the case in which the first stage of regional development completes in 1990.

7 SOCIO-ECONOMIC IMPACTS

In addition to the benefits stipulated in the economic evaluation, favourable socio-economic impacts are created by the implementation of the project.

Increase of employment opportunity by the project implementation will give a favourable impact on the regional economy. About 1,000 persons will be newly employed during the construction period of the river and drainage improvement works, and 22 persons will be required permanently for the operation and maintenance works.

Technical knowledge will be also transferred to the Indonesian staff through the construction work in various fields, which will facilitate realization of other flood control projects in the future.

The living environment will be surely improved by the project, and the enhanced economic activity through the improved living environment will also exert a good influence on the socio-economic stability in the region.

Tabel 2-1(1) DISTRIBUTION OF BUILDINGS BY GROUND HEIGHT

(EXISTING)

Unit:nos.

	Ground Height (M.S.L.m)	House			Shop	Factory	School	Office
		permanent	semi-permanent	temporary				
Mountain-Side Area	0.5 - 1.0	27	67	147	12	2	3	3
	1.0 - 2.0	74	190	413	33	-	4	-
	2.0 - 3.0	1,430	1,409	3,143	127	11	23	14
	Sub-total	1,531	1,666	3,703	172	13	30	17
City-Side Area	0.5 - 1.0	-	-	-	-	-	-	-
	1.0 - 2.0	206	524	1,141	24	-	11	8
	2.0 - 3.0	5,689	5,193	11,014	779	84	105	88
	Sub-total	5,895	5,717	12,155	803	84	116	96
Total	0.5 - 1.0	27	67	147	12	2	3	3
	1.0 - 2.0	280	714	1,554	57	-	15	8
	2.0 - 3.0	7,119	6,602	14,157	906	95	128	102
	Ground Total	7,426	7,383	15,858	975	97	146	113

Tabel 2-1(2) DISTRIBUTION OF BUILDINGS BY GROUND HEIGHT

(PRE-DEVELOPED AREA AT FIRST STAGE)

Unit:nos.

	Ground Height (M.S.L.m)	House			Shop	Factory	School	Office
		permanent	semi-permanent	temporary				
Mountain-Side Area	0.5 - 1.0	27	67	147	12	2	3	3
	1.0 - 2.0	74	190	413	33	0	4	-
	2.0 - 3.0	1,405	1,317	3,032	127	11	23	14
	Sub-total	1,506	1,574	3,592	172	13	30	17
City-Side Area	0.5 - 1.0	-	-	-	-	-	-	-
	1.0 - 2.0	124	317	690	6	-	11	8
	2.0 - 3.0	5,328	4,788	10,740	761	84	105	88
	Sub-total	5,452	5,105	11,430	767	84	116	96
Total	0.5 - 1.0	27	67	147	12	2	3	3
	1.0 - 2.0	198	507	1,103	39	-	15	8
	2.0 - 3.0	6,733	6,105	13,772	888	95	128	102
	Ground Total	6,958	6,679	15,022	939	97	146	113

Tabel 2-1(3) DISTRIBUTION OF BUILDINGS BY GROUND HEIGHT

(DEVELOPED AREA AT FIRST STAGE)

	Ground Height (M.S.L.m)	House (nos.)			Shop (m^2)	Factory (m^2)	School (m^2)	Office (m^2)
		small	medium	large				
Mountain-Side Area	0.5 - 1.0	-	-	-	-	-	-	-
	1.0 - 2.0	5	33	108	-	32,786	3,580	-
	2.0 - 3.0	35	128	661	-	53,494	16,420	51,487
	Sub-total	40	161	769	-	86,280	20,000	51,487
City-Side Area	0.5 - 1.0	-	-	-	-	-	-	-
	1.0 - 2.0	134	138	109	16,929	-	-	-
	2.0 - 3.0	469	554	205	9,336	-	28,200	48,488
	Sub-total	603	692	314	26,265	-	28,200	48,488
Total	0.5 - 1.0	-	-	-	-	-	-	-
	1.0 - 2.0	139	171	217	16,929	32,786	3,580	-
	2.0 - 3.0	504	682	866	9,336	53,494	44,620	99,975
	Ground Total	643	853	1,083	26,265	86,280	48,200	99,975

Tabel 2-1(4) DISTRIBUTION OF BUILDINGS BY GROUND HEIGHT

(PRE-DEVELOPED AREA AT SECOND STAGE)

Unit:nos.

	Ground Height (M.S.L.m)	House			Shop	Factory	School	Office
		permanent	semi-permanent	temporary				
Mountain-Side Area	0.5 - 1.0	4	33	113	5	2	-	1
	1.0 - 2.0	11	15	-	-	-	-	-
	2.0 - 3.0	519	390	1,238	54	6	11	5
	Sub-total	534	438	1,351	59	8	11	6
City-Side Area	0.5 - 1.0	-	-	-	-	-	-	-
	1.0 - 2.0	118	215	430	-	-	9	8
	2.0 - 3.0	4,939	4,623	10,712	732	83	103	87
	Sub-total	5,057	4,838	11,142	732	83	112	95
Total	0.5 - 1.0	4	33	113	5	2	-	1
	1.0 - 2.0	129	230	430	-	-	9	8
	2.0 - 3.0	5,458	5,013	11,950	786	89	114	92
	Ground Total	5,591	5,276	12,493	791	91	123	101

Table 2-1(5) DISTRIBUTION OF HOUSES BY GROUND HEIGHT (DEVELOPED AREA AT SECOND STAGE)

	Ground Height (M.S.L.m)	House (nos)	Shop (m^2)	Factory (m^2)	School (m^2)	Office (m^2)
Mountain-Side Area	0.5 - 1.0	2,148	3,000	48,000	6,000	20,000
	1.0 - 2.0	19,330	-	367,000	-	-
	2.0 - 3.0	-	-	-	-	-
	Sub-total	21,478	3,000	415,000	6,000	20,000
City-Side Area	0.5 - 1.0	-	-	-	-	-
	1.0 - 2.0	-	-	-	-	-
	2.0 - 3.0	905	-	-	-	-
	Sub-total	905	-	-	-	-
Total	0.5 - 1.0	2,148	3,000	48,000	6,000	20,000
	1.0 - 2.0	19,330	-	369,000	-	-
	2.0 - 3.0	905	-	-	-	-
	Groun Total	22,383	3,000	415,000	6,000	20,000

Tabel 2-1(6) DISTRIBUTION OF BUILDINGS BY GROUND HEIGHT (PRE-DEVELOPED AREA AT THIRD STAGE)

Unit:nos.

	Ground Height (M.S.L.m)	House			Shop	Factory	School	Office
		permanent	semi-permanent	temporary				
Mountain-Side Area	0.5 - 1.0	1	2	20	2	1	-	-
	1.0 - 2.0	-	-	-	-	-	-	-
	2.0 - 3.0	469	228	942	39	5	8	4
	Sub-total	470	230	962	41	6	8	4
City-Side Area	0.5 - 1.0	-	-	-	-	-	-	-
	1.0 - 2.0	118	215	430	-	-	9	8
	2.0 - 3.0	4,939	4,623	10,712	732	83	103	87
	Sub-total	5,057	4,838	11,142	732	83	112	95
Total	0.5 - 1.0	1	2	20	2	1	-	-
	1.0 - 2.0	118	215	430	-	-	9	8
	2.0 - 3.0	5,408	4,851	11,654	771	88	111	91
	Ground Total	5,527	5,068	12,104	773	89	120	99

Table 2-1(7) DISTRIBUTION OF BUILDINGS BY GROUND HEIGHT (DEVELOPED AREA AT THIRD STAGE)

	Ground Height (M.S.L.m)	House (nos)	Shop (m^2)	Factory (m^2)	School (m^2)	Office (m^2)
Mountain-Side Area	0.5 - 1.0	8,769	29,000	-	58,000	191,000
	1.0 - 2.0	4,839	25,000	46,000	5,000	167,000
	2.0 - 3.0	-	-	82,000	-	2,000
	Sub-total	13,608	54,000	128,000	63,000	360,000
City-Side Area	0.5 - 1.0	-	-	-	-	-
	1.0 - 2.0	-	-	-	-	-
	2.0 - 3.0	905	-	-	-	-
	Sub-total	905	-	-	-	-
Total	0.5 - 1.0	8,769	29,000	-	58,000	191,000
	1.0 - 2.0	4,839	25,000	46,000	5,000	167,000
	2.0 - 3.0	905	-	82,000	-	2,000
	Groun Total	14,513	54,000	128,000	63,000	360,000

Table 2-2 DISTRIBUTION OF ASSETS BY GROUND HEIGHT

Unit: $x10^9$Rp

	Ground Height (M.S.L.m)	Existing	First Stage	Second Stage	Third Stage
Mountain side Area	0.5 - 1.0	0.753	0.753	40.452	205.498
	1.0 - 2.0	2.006	4.877	61.468	176.920
	2.0 - 3.0	28.749	44.284	83.966	90.802
City side Area	0.5 - 1.0	0	0	0	0
	1.0 - 2.0	5.623	13.367	13.832	13.832
	2.0 - 3.0	113.403	140.411	126.545	126.545
Total	0.5 - 1.0	0.753	0.753	40.452	205.498
	1.0 - 2.0	7.629	19.044	75.300	190.752
	2.0 - 3.0	142.153	184.835	210.511	217.345

Table 2-3 AVERAGE VALUE OF BUILDINGS AND HOUSEHOLD EFFECTS

Unit: Rp

	Classification	Building	Household Effects	Total
Existing	Residence (Unit)			
	Permanent	11,000,000	4,820,000	15,820,000
	Semi-permanent	2,800,000	670,000	3,470,000
	Temporary	200,000	60,000	260,000
	Store (Unit)	580,000	960,000	1,540,000
	Factory (Unit)	4,100,000	1,900,000	6,000,000
	School (Unit)	4,000,000	470,000	4,470,000
	Office (Unit)	3,000,000	950,000	3,950,000
Newly Constructed	Residence (Unit)			
	big	25,000,000	11,250,000	36,250,000
	medium	10,000,000	4,500,000	14,500,000
	small	3,200,000	800,000	4,000,000
	Store (per m2)	60,000	102,000	162,000
	Factory (per m^2)	50,000	25,000	75,000
	School (per m^2)	50,000	6,000	56,000
	Office (per m2)	80,000	28,000	108,000

Table 2-4 DISTRIBUTION OF AGRICULTURAL AREA BY GROUND HEIGHT

Unit: ha

	Stage / Ground Height (M.S.L.m)	Existing		First Stage		Second Stage		Third Stage	
		Area	Acumulative	Area	Acumulative	Area	Acumulative	Area	Acumulative
Mountain side Area	0 - 0.5	132.3		132.3		132.3		51.9	
	0.5 - 1.0	583.4	715.7	583.4	715.7	478.7	611.0	91.7	143.6
	1.0 - 2.0	682.8	1,398.5	682.3	1,398.5	600.1	1,211.1	327.3	470.9
	2.0 - 3.0	358.7	1,757.2	338.3	1,736.8	85.4	1,296.5	51.5	522.4
City side Area	0 - 0.5	-	-	-	-	-	-	-	-
	0.5 - 1.0	-	-	-	-	-	-	-	-
	1.0 - 2.0	125.6		75.2		75.2		75.2	
	2.0 - 3.0	267.0	392.6	164.0	239.2	125.5	200.7	125.5	200.7
Total Area	0 - 0.5	132.3		132.3		132.3		51.9	
	0.5 - 1.0	583.4	715.7	583.4	715.7	478.7	611.0	91.7	143.6
	1.0 - 2.0	808.4	1,524.1	757.5	1,473.2	675.3	1,286.3	402.5	546.1
	2.0 - 3.0	625.7	2,149.8	502.3	1,975.5	210.9	1,497.2	177.0	723.1

Table 2-5 ANNUAL PADDY YIELD IN UJUNG PANDANG CITY

Year	Planted Area (ha)	Annual Yield (ton)	Yield per Area (ton/ha)
1974	2,814	5,664	2.01
1975	3,660	7,893	2.19
1976	3,405	7,973	2.34
1977	3,549	8,560	2.41
1978	3,579	8,999	2.51
Average	3,389	7,818	2.29

Table 3-1 CATEGORIES OF FLOOD LOSSES AND RISKS

Ownership Type	Private Sector	Public Sector
Direct	Cost of cleaning, repairing, or replacing residential, commercial, and industruial buildings;contents and land. Cost of cleaning, repairing, or replacing agricultural buildings and contents and cost of lost crops and livestock.	Cost of repairing or replacing roads, segments, bridges, culverts, and dams. Cost of repairing damage to storm water systems, sanitary sewerage systems, and other utilities. Cost of restoring parks and other public recreational lands.
Indirect	Cost of temporary evacuation and relocation. Lost wages. Lost production and sales. Incremental cost of transportation. Cost of post-flood floodproofing.	Incremental costs to governmental units as a result of flood fighting measures. Cost of post-flood engineering and planning studies and of implementing structural and nonstructural floodland management recommendations.
Intangible	Loss of life. Health hazards. Psychological stress. Reluctance by individuals to inhabit flood-prone areas thereby depreciating riverine area property values.	Disruption of normal community activities.' Reluctance by business interests to continue development of flood-prone commercial-industrial areas therreby advesely affecting the community tax base.

Table 3-2 FLOOD DAMAGE RATE OF BUILDINGS AND HOUSEHOLD EFFECTS

Ponding Depth about Floor Level (m)	House	Household Effects			
		House	Shop	Office	School and Factory
0 - 0.5	0.05	0.11	0.08	0.08	0.08
0.5 - 1.0	0.07	0.29	0.22	0.28	0.24
1.0 - 1.5	0.11	0.41	0.35	0.42	0.35
1.5 - 2.0	0.11	0.47	0.44	0.47	0.39
2.0 - 2.5	0.15	0.49	0.51	0.49	0.40
2.5 - 3.0	0.15	0.51	0.57	0.49	0.41

Table 3-3 UNIT REICE OF PADDY

(Rp./Kg)

	1974	1975	1976	1977	1978
Paddy	30.0	42.2	51.0	54.0	75.4
Gabah	38.5	64.0	66.5	70.0	95.0
Beras	68.5	97.0	110.0	119.0	166.0

Table 3-4 FLOOD FREQUENCY

Month	1	2	3	4	5	6	7	8	9	10	11	12
Frequency (%)	44	16	3	9	3	0	0	0	0	0	3	22

Table 3-5 FLOOD DAMAGE RATE OF PADDY

Growing Stage		Tillering Stage	Booting Stage	Heading Stage	Ripening Stage
Relative Growth (%)		0 - 59	60 - 76	77 - 79	80 - 100
Relative Growth (cm)		0 - 74	75 - 95	96 - 99	100 - 125
Over Head Flooding	1-2 day	10 %	70 %	30 %	5 %
	3-4	20	80	80	20
	5-6	30	85	90	30
	Over 7	35	95	100	30
Flooding up to 75% plant Height	1-2 day	6	40	10	4
	3-4	9	46	23	15
	5-6	14	49	26	23
	Over 7	16	55	30	23
Flooding up to 50% plant Height	1-2 day	4	37	8	2
	3-4	9	42	22	4
	5-6	13	45	25	6
	Over 7	15	50	28	6

Table 3-7 DISTRIBUTION OF DAMAGE BY GROUND HEIGHT

BUILDING AND HOUSEHOLD EFFECTS

Unit: x10^6Rp.

	Ground Height (M.S.L.m)	Existing	First Stage	Second Stage	Third Stage
Mountain Side Area	1.2	47	47	2,699	14,388
	1.7	154	268	7,421	32,103
	2.2	321	665	14,046	53,653
	2.7	1,387	2,619	21,345	68,359
	3.2	3,340	5,474	29,620	84,959
City Side Area	1.2	-	-	-	-
	1.7	-	-	-	-
	2.2	348	829	863	863
	2.7	4,381	6,450	6,645	6,645
	3.2	12,084	16,546	17,205	17,205

CROPS

Unit: x10^6Rp.

	Ground Height (M.S.L.m)	Existing	First Stage	Second Stage	Third Stage
Mountain Side Area	0.5	2.56	2.56	2.56	1.00
	1.0	13.84	13.84	11.82	2.78
	2.0	27.04	27.04	23.42	9.11
	3.0	33.98	33.59	25.07	10.10
City Side Area	0.5	-	-	-	-
	1,0	-	-	-	-
	2.0	2.43	1.45	1.45	1.45
	3.0	7.59	4.63	3.88	3.88

Table 3-8(1)   ANNUAL EXPECTED DIRECT DAMAGE REDUCTION

BUILDING AND HOUSEHOLD EFFECTS IN THE FIRST DEVELOPMENT STAGE

1	2				3		4	5	6	7	8
Return Period (1/T)	Inundation Water Stage(M.S.L.m)				Flood Damage(10^6Rp)		Flood Damage Reduction (10^6Rp)	Average (10^6Rp)	Expected Value	5 x 6 (10^6Rp)	Total (10^6Rp)
	Without Project		With Project		Without Project	With Project					
	H_1	H_2	H_1	H_2							
(1/1)	(1.30)	(1.89)	(1.12)	(1.27)	520	40	480				
								535	0.50	268	
1/2	1.45	2.03	1.30	1.54	690	100	590				
								655	0.084	55	
1/2.4	1.50	2.14	1.34	1.60	840	120	720				
								1,500	0.216	324	
1/5	1.68	2.53	1.50	1.87	2,700	420	2,280				
								4,460	0.100	446	
1/10	2.11	2.79	1.64	2.01	7,300	660	6,640				
											1,093

NOTE H_1 : Inundation water stage in the mountain-side area

H_2 : Inundation water stage in the city-side area

Table 3-8(2) ANNUAL EXPECTED DIRECT DAMAGE REDUCTION

FARM CROPS IN THE FIRST DEVELOPMENT STAGE

1	2				3		4	5	6	7	8
Return Period (1/T)	Inundation Water Stage(M.S.L.m)				Flood Damage(10^6Rp)		Flood Damage Reduction (10^6Rp)	Average (10^6Rp)	Expected Value	5 x 6 (10^6Rp)	Total (10^6Rp)
	Without Project		With Project		Without Project	With Project					
	H1	H2	H1	H2							
(1/1)	(1.30)	(1.89)	(1.12)	(1.27)	20.1	17.3	3.8				
								3.60	0.50	1.800	
1/2	1.45	2.03	1.30	1.54	23.0	19.6	3.4				
								3.45	0.084	0.290	
1/2.4	1.50	2.14	1.34	1.60	23.7	20.2	3.5				
								4.50	0.216	0.972	
1/5	1.68	2.53	1.50	1.87	27.2	21.7	5.5				
								6.20	0.100	0.620	
1/10	2.11	2.79	1.64	2.01	31.4	24.5	6.9				
											3.682

NOTE H_1 : Inundation water stage in the mountain-side area

H_2 : Inundation water stage in the city-side area

Table 4-1 BREAKDOWN OF ECONOMIC COST

WORKS	UNIT	QUANTITY	COST ($x10^3$US$)	
			F.C.	L.C.
1. RIVER IMPROVEMENT				
Main Works				
Dredgeing	m^3	264,000	225	233
Excavation	m^3	530,000	1,051	946
Embankment	m^3	95,500	202	199
Sodding	m^2	83,000	-	41
Revetment	m	5,700	-	1,155
Groyne	pc	23	-	53
Sluice	pc	3	1	31
Drainage Ditch	m	2,800	9	9
Road Raising	m	2,950	27	162
Sub-Total			1,515	2,829
Preparatory Work	LS		227	424
Land Acquisition and House Evacuation	LS		-	55
Total for 1			1,742	3,308
2. DRAINAGE CHANNEL				
Main Works				
Excavation	m^3	399,200	339	184
Foundation	m	23,290	-	207
Revetment	m^2	93,700	-	937
Backfill	m^3	10,500	-	2
Filling	m^3	19,500	6	4
Spoil	m^3	369,200	334	406
Dredging	m^3	138,400	195	227
Bridge	nos.	22	8	621
Sluice	nos.	1	1	64
Sub-Total			883	2,652
Preparatory Work	LS		132	398
Land Acquisition and House Evacuation	LS		-	824
Total for 2			1,015	3,874
Total for 1 and 2			2,757	7,182
3. Engineering	LS		1,606	357
Grand Total			4,363	7,539

Table 4-2 ANNUAL DISBURSEMENT OF ECONOMIC COST

UNIT: x10^3US$

	ITEM	TOTAL		1		2		3		4		5	
		F.C.	L.C.	F.C.	L.C.	F.C.	L.C.	F.C.	L.C.	F.C.	L.C.	F.C.	L.C.
Jeneberang River Improvement	MAIN WORKS	1,515.0	2,829.0	-	-	226.8	253.9	441.1	744.4	403.3	864.6	443.8	966.1
	PREPARATORY WORK	75.8	141.4	-	-	11.3	12.7	22.1	37.2	20.2	43.2	22.2	48.3
	SURVEY & SOIL TEST	151.5	282.9	-	-	22.7	25.4	44.1	74.4	40.3	86.5	44.4	96.6
	LAND ACQUISITION	-	35.4	-	-	-	5.8	-	9.1	-	2.8	-	17.7
	HOUSE EVACUATION	-	19.2	-	-	-	6.4	-	6.4	-	3.2	-	3.2
	SUB-TOTAL	1,742.3	3,307.9	-	-	260.8	304.2	507.3	871.5	463.8	1,000.3	510.4	1,131.9
Drainage System Improvement	PANAMPU ROUTE	293.5	1,054.3	-	-	206.1	623.6	87.4	430.7	-	-	-	-
	JONGAYA ROUTE	543.4	1,233.1	-	-	77.9	90.7	264.4	522.3	201.0	620.1	-	-
	SINRIJALA ROUTE	45.4	365.3	-	-	-	-	-	-	-	-	45.4	365.3
	PREPARATORY WORK	44.1	132.7	-	-	14.2	35.7	17.6	47.7	10.1	31.0	2.3	18.3
	SURVEY & SOIL TEST	88.2	265.2	-	-	28.4	71.4	35.2	95.3	20.1	62.0	4.5	36.5
	LAND ACQUISITION	-	705.3	-	-	-	229.1	-	252.1	-	162.7	-	61.4
	HOUSE EVACUATION	-	118.4	-	-	-	44.8	-	48.0	-	25.6	-	-
	SUB-TOTAL	1,014.6	3,874.3	-	-	326.6	1,095.3	404.6	1,396.1	231.2	901.4	52.2	481.5
	ENGINEERING WORK	1,606.0	357.4	750.0	120.0	216.0	59.7	240.0	66.3	200.0	55.5	200.0	55.5
	TOTAL	4,362.9	7,539.2	750.0	120.0	803.4	1,459.2	1,151.9	2,333.9	895.0	1,957.2	762.6	1,668.9
	GRAND TOTAL	11,902.1		920.0		2,262.6		3,485.8		2,852.2		2,431.5	

NOTE: The maiantenance cost is not included in this table
Conversion rate; 250 YEN to 1 US$

Table 4-3 BREAKDOWN OF OPERATION AND MAINTENANCE COST

Unit: x10^3Rp.

Item	1983		1984		1985		1986 - 2030	
	Number	Amount	Number	Amount	Number	Amount	Number	Amount
Remuneration								
- Supervisor	1 person	1,080	1 person	1,080	1 person	1,080	1 person	1,080
- Staff	1 person	720	2 persons	1,440	4 persons	2,160	6 persons	4,320
- Driver	1 person	480	1 person	480	2 persons	960	2 persons	960
- Operator	1 person	1,080	1 person	1,080	1 person	1,080	1 perosn	1,080
- Labor	6 persons	1,620	8 persons	2,160	10 persons	2,700	12 persons	3,240
Machinery								
- Jeep	1 nos.	1,000	1 nos.	1,000	2 nos.	2,000	2 nos.	2,000
- Clamshell	1 nos.	9,000	1 nos.	9,000	1 nos.	9,000	1 nos.	9,000
- Mechinary Maintenance	L.S.	5,000	L.S.	5,000	L.S.	5,500	L.S.	5,500
Office running Cost	L.S.	4,000	L.S.	4,000	L.S.	4,000	L.S.	4,000
Miscellaneous	L.S.	2,370	L.S.	2,520	L.S.	2,850	L.S.	3,120
T o t a l		26,080		27,760		31,330		34,300

Table 5-1(1) BREAKDOWN OF CONSTRUCTION COST (CONTRACT BASE)

WORKS	UNIT	QUANTITY	COST ($x10^3$US$) F.C.	L.C.
1. RIVER IMPROVEMENT				
Main Works				
Dredging	m^3	264,000	289	338
Excavation	m^3	530,000	1,352	1,352
Embankment	m^3	95,500	260	315
Sodding	m^2	83,000	-	94
Revetment	m	5,700	-	1,908
Groyne	pc	23	-	86
Sluice	pc	3	1	51
Drainage Ditch	m	2,800	12	15
Road Raising	m	2,950	37	267
Sub-Total			1,951	4,426
Preparatory Work	LS		293	664
Land Acquisition and House Evacuation	LS		-	78
Total for 1			2,244	5,168
2. DRAINAGE CHANNEL				
Main Works				
Excavation	m^3	399,200	420	251
Foundation	m	23,290	-	334
Revetment	m^2	93,700	-	1,537
Backfill	m^3	10,500	-	11
Filling	m^3	19,500	7	5
Spoil	m^3	369,200	412	547
Dredging	m^3	138,400	232	291
Bridge	nos.	22	10	887
Sluice	nos.	1	2	106
Sub-Total			1,083	3,969
Preparatory Work	LS		163	596
Land Acquisition and House Evacuation	LS		-	1,105
Total for 2			1,246	5,671
Total for 1 and 2			3,490	10,838
3. Contingencies	%	10	349	1,084
4. Engineering	LS		1,884	463
Grand Total			5,723	12,385

NOTE: Escalation rate - 7% for F.C. and 10% for L.C.
Conversion rate - 250 Yen to 1 US$

Table 5-1(2) BREAKDOWN OF CONSTRUCTION COST
(FORCE ACCOUNT BASE)

WORKS	UNIT	QUANTITY	COST ($x10^3$US$)	
			F.C.	L.C.
1. RIVER IMPROVEMENT				
Main Works				
Dredging	m^3	264,000	136	338
Excavation	m^3	530,000	492	1,352
Embankment	m^3	95,500	97	315
Sodding	m^2	83,000	-	94
Revetment	m	5,700	-	1,908
Groyne	pc	23	-	86
Sluice	pc	3	1	51
Drainage Ditch	m	2,800	4	15
Road Raising	m	2,950	14	267
Sub-Total			744	4,426
Preparatory Work	LS		111	664
Land Acquisition and House Evacuation	LS		-	78
Equipment	LS		2,452	-
Total for 1			3,307	5,168
2. DRAINAGE CHANNEL				
Main Works				
Excavation	m^3	399,200	140	251
Foundation	m	23,290	-	334
Revetment	m^2	93,700	-	1,537
Backfill	m^3	10,500	-	11
Filling	m^3	19,500	3	5
Spoil	m^3	369,200	148	547
Dredging	m^3	138,400	82	291
Bridge	nos.	22	4	887
Sluice	nos.	1	1	106
Sub-Total			378	3,969
Preparatory Work	LS		56	596
Land Acquisition and House Evacuation	LS		-	1,105
Equipment	LS		1,036	-
Total for 2			1,470	5,670
Total for 1 and 2			4,777	10,838
3. Contingencies	%	10	478	1,084
4. Engineering	LS		1,884	463
Grand Total			7,139	12,385

NOTE: Escalation rate - 7% for F.C. and 10% for L.C.
Conversion rate - 250 Yen to 1 US$

Table 5-2(1) ANNUAL DISBURSEMENT OF CONSTRUCTION COST
(CONTRACT BASE)

UNIT: x10^3US$

	ITEM	TOTAL		1		2		3		4		5	
		F.C.	L.C.	F.C.	L.C.	F.C.	L.C.	F.C.	L.C.	F.C.	L.C.	F.C.	L.C.
Jeneberang River Improvement	MAIN WORKS	1951.4	4425.6	-	-	259.7	321.7	540.3	1061.7	528.7	1377.2	622.7	1665.0
	PREPARATORY WORK	97.5	221.3	-	-	12.9	16.1	27.0	53.1	26.5	68.8	31.1	83.3
	SURVEY & SOIL TEST	195.1	442.8	-	-	26.0	32.2	54.0	106.2	52.8	137.8	62.3	166.6
	LAND ACQUISITION	-	51.7	-	-	-	7.0	-	12.1	-	4.1	-	28.5
	HOUSE EVACUATION	-	26.1	-	-	-	7.7	-	8.5	-	4.7	-	5.2
	SUB-TOTAL	2244.0	5167.5	-	-	298.6	384.7	621.3	1241.6	608.0	1592.6	716.1	1948.6
Drainage System Improvement	PANAMPU ROUTE	343.1	1461.1	-	-	236.0	827.9	107.1	633.2	-	-	-	-
	JONGAYA ROUTE	676.6	1884.1	-	-	89.2	109.9	323.9	735.9	263.5	998.3	-	-
	SINRIJALA ROUTE	63.7	664.7	-	-	-	-	-	-	-	-	63.7	664.7
	PREPARATORY WORK	54.3	198.4	-	-	16.3	46.9	21.6	68.4	13.2	49.9	3.2	33.2
	SURVEY & SOIL TEST	108.3	396.8	-	-	32.5	93.8	43.1	136.7	26.4	99.8	6.3	66.5
	LAND ACQUISITION	-	949.8	-	-	-	277.2	-	335.5	-	238.2	-	98.9
	HOUSE EVACUATION	-	155.6	-	-	-	54.2	-	63.9	-	37.5	-	-
	SUB-TOTAL	1246.0	5670.5	-	-	374.0	1409.9	495.7	1973.6	303.1	1423.7	73.2	863.3
	CONTINGENCIES	349.0	1083.8	-	-	67.3	179.5	111.7	321.5	91.5	301.6	78.9	287.2
	ENGINEERING WORK	1884.0	463.1	800.0	132.0	247.3	72.2	294.0	88.2	262.2	81.3	280.5	89.4
	TOTAL	5723.0	12384.9	800.0	132.0	987.2	2046.3	1522.7	3624.9	1264.4	3399.2	1148.7	3182.5
	GRAND TOTAL	18107.9		932.0		3033.5		5147.6		4663.6		4331.2	

NOTE: Escalation rate - 7% for F.C. and 10% for L.C., Conversion rate - 250 Yen = 1 US$

Table 5-2(2) ANNUAL DISBURSEMENT OF CONSTRUCTION COST
(FORCE ACCOUNT BASE)

UNIT: x10^3US$

	ITEM	TOTAL		1		2		3		4		5	
		F.C.	L.C.	F.C.	L.C.	F.C.	L.C.	F.C.	L.C.	F.C.	L.C.	F.C.	L.C.
Jeneberang River Improvement	MAIN WORKS	743.7	4425.6	-	-	98.9	321.7	206.0	1061.7	201.8	1377.2	237.0	1665.0
	PREPARATORY WORK	37.1	221.3	-	-	4.9	16.1	10.3	53.1	10.1	68.8	11.8	83.3
	SURVEY & SOIL TEST	74.3	442.8	-	-	9.8	32.2	20.6	106.2	20.2	137.8	23.7	166.6
	LAND ACQUISITION	-	51.7	-	-	-	7.0	-	12.1	-	4.1	-	28.5
	HOUSE EVACUATION	-	26.1	-	-	-	7.7	-	8.5	-	4.7	-	5.2
	CONSTRUCTION EQUIPMENT	2452.4	-	-	-	1551.2	-	757.8	-	-	-	143.4	-
	SUB-TOTAL	3307.5	5167.5	-	-	1664.8	384.7	994.7	1241.6	232.1	1592.6	415.9	1948.6
Drainage System Improvement	PANAMPU ROUTE	119.5	1458.9	-	-	82.1	827.9	37.4	633.2	-	-	-	-
	JONGAYA ROUTE	236.1	1844.1	-	-	31.4	109.9	113.2	735.9	91.5	998.3	-	-
	SINRIJALA ROUTE	22.2	664.7	-	-	-	-	-	-	-	-	22.2	664.7
	PREPARATORY WORK	18.9	198.4	-	-	5.7	46.9	7.5	68.4	4.6	49.9	1.1	33.2
	SURVEY & SOIL TEST	37.8	396.8	-	-	11.3	93.8	15.1	136.7	9.2	99.8	2.1	66.5
	LAND ACQUISITION	-	949.8	-	-	-	277.2	-	335.5	-	238.2	-	98.9
	HOUSE EVACUATION	-	217.0	-	-	-	54.2	-	63.9	-	37.5	-	-
	CONSTRUCTION EQUIPMENT	1035.5	-	-	-	1035.5	-	-	-	-	-	-	-
	SUB-TOTAL	1470.0	5670.5	-	-	1166.0	1409.9	173.2	1973.6	105.3	1423.7	25.5	863.3
	CONTINGENCIES	477.8	1083.8	-	-	283.1	179.5	116.8	321.5	33.7	301.6	44.1	281.2
	ENGINEERING WORK	1884.0	463.1	800.0	132.0	247.3	72.2	294.0	88.2	262.2	81.3	280.5	89.4
	TOTAL	7139.3	12384.9	800.0	132.0	3361.2	2046.3	1578.7	3624.9	633.3	3399.2	766.0	3182.5
	GRAND TOTAL	19524.1		932.0		5407.5		5203.6		4032.5		3948.5	

NOTE: Escalation rate - 7% for F.C. and 10% for L.C., Conversion rate - 250 Yen = 1 US$

Table 5-3(1) BREAKDOWN OF CONSTRUCTION COST (CONTRACT BASE)

WORKS	UNIT	QUANTITY	COST (x10^3US$) F.C.	L.C.
1. RIVER IMPROVEMENT				
Main Works				
Dredging	m^3	264,000	-	659
Excavation	m^3	530,000	-	2,852
Embankment	m^3	95,500	-	603
Sodding	m^2	83,000	-	94
Revetment	m	5,700	-	1,909
Groyne	pc	23	-	86
Sluice	pc	3	-	52
Drainage Ditch	m	2,800	-	28
Road Raising	m	2,950	-	309
Sub-Total			-	6,592
Preparatory Work	LS		-	989
Land Acquisition and House Evacuation	LS		-	78
Total for 1			-	7,659
2. DRAINAGE CHANNEL				
Main Works				
Excavation	m^3	399,200	-	709
Foundation	m	23,290	-	334
Revetment	m^2	93,700	-	1,537
Backfill	m^3	10,500	-	11
Filling	m^3	19,500	-	14
Spoil	m^3	369,200	-	997
Dredging	m^3	138,400	-	541
Bridge	nos.	22	-	898
Sluice	nos.	1	-	108
Sub-Total			-	5,147
Preparatory Work	LS		-	773
Land Acquisition and House Evacuation	LS		-	1,105
Total for 2			-	7,027
Total for 1 and 2			-	14,686
3. Contingencies	%	10	-	1,469
4. Engineering	LS		1,884	463
Grand Total			1,884	16,618

NOTE: Escalation rate - 7% for F.C. and 10% for L.C.
Conversion rate - 250 Yen to 1 US$

Table 5-3(2) BREAKDOWN OF CONSTRUCTION COST (FORCE ACCOUNT BASE)

WORKS	UNIT	QUANTITY	COST ($x10^3$US$)	
			F.C.	L.C.
1. RIVER IMPROVEMENT				
Main Works				
Dredging	m^3	264,000	-	488
Excavation	m^3	530,000	-	1,899
Embankment	m^3	95,500	-	422
Sodding	m^2	83,000	-	95
Revetment	m	5,700	-	1,909
Groyne	pc	23	-	86
Sluice	pc	3	-	51
Drainage Ditch	m	2,800	-	19
Road Raising	m	2,950	-	283
Sub-Total			-	5,252
Preparatory Work	LS		-	788
Land Acquisition and House Evacuation	LS		-	78
Equipment	LS		-	2,627
Total for 1			-	8,745
2. DRAINAGE CHANNEL				
Main Works				
Excavation	m^3	399,200	-	404
Foundation	m	23,290	-	334
Revetment	m^2	93,700	-	1,536
Backfill	m^3	10,500	-	12
Filling	m^3	19,500	-	9
Spoil	m^3	369,200	-	708
Dredging	m^3	138,400	-	379
Bridge	nos.	22	-	891
Sluice	nos.	1	-	106
Sub-Total			-	4,379
Preparatory Work	LS		-	657
Land Acquisition and House Evacuation	LS		-	1,143
Equipment	LS		-	1,094
Total for 2			-	7,273
Total for 1 and 2			-	16,018
3. Contingencies	%	10	-	1,602
4. Engineering	LS		1,884	463
Grand Total			1,884	18,083

Table 5-4(1) ANNUAL DISBURSEMENT OF CONSTRUCTION COST
(CONTRACT BASE)

UNIT: x10^3US$

	ITEM	TOTAL		1		2		3		4		5	
		F.C.	L.C.	F.C.	L.C.	F.C.	L.C.	F.C.	L.C.	F.C.	L.C.	F.C.	L.C.
Jeneberang River Improvement	MAIN WORKS	-	6,592.5	-	-	-	596.2	-	1,648.8	-	1,967.6	-	2,379.9
	PREPARATORY WORK	-	329.7	-	-	-	29.8	-	82.4	-	98.4	-	119.1
	SURVEY & SOIL TEST	-	659.3	-	-	-	59.7	-	164.9	-	196.8	-	237.9
	LAND ACQUISITION	-	51.7	-	-	-	7.0	-	12.1	-	4.1	-	28.5
	HOUSE EVACUATION	-	26.1	-	-	-	7.7	-	8.5	-	4.7	-	5.2
	SUB-TOTAL	-	7,659.2	-	-	-	700.3	-	1,916.8	-	2,271.5	-	2,770.6
Drainage System Improvement	PANAMPU ROUTE	-	1,826.8	-	-	-	1,077.3	-	749.5	-	-	-	-
	JONGAYA ROUTE	-	2,584.5	-	-	-	204.1	-	1,087.8	-	1,292.6	-	-
	SINRIJALA ROUTE	-	737.8	-	-	-	-	-	-	-	-	-	737.8
	PREPARATORY WORK	-	257.4	-	-	-	64.1	-	91.8	-	64.6	-	36.9
	SURVEY & SOIL TEST	-	514.9	-	-	-	128.1	-	183.7	-	129.3	-	73.8
	LAND ACQUISITION	-	949.8	-	-	-	277.2	-	335.5	-	238.2	-	98.9
	HOUSE EVACUATION	-	155.6	-	-	-	54.2	-	63.9	-	37.5	-	-
	SUB-TOTAL	-	7,027.2	-	-	-	1,805.4	-	2,512.3	-	1,762.1	-	947.4
	CONTINGENCIES		1,468.7	-	-	-	250.6	-	443.0	-	403.3	-	371.8
	ENGINEERING WORK	1,884.0	463.1	800.0	132.0	247.3	72.2	294.0	88.2	262.2	81.3	280.5	89.4
	TOTAL	1,884.0	16,618.2	800.0	132.0	247.3	2,828.6	294.0	4,960.3	262.2	4,518.2	280.5	4,179.3
	GRAND TOTAL	18,502.2		932.0		3,075.9		5,254.3		4,780.4		4,459.8	

NOTE: ESCALATION RATE - 7% FOR F.C. AND 10% FOR L.C.
CONVERSION RATE - 250 YEN to 1US$

Table 5-4(2) ANNUAL DISBURSEMENT OF CONSTRUCTION COST
(FORCE ACCOUNT BASIS IN LOCAL PORTION)

UNIT: x10^{3}US$

ITEM		TOTAL		1		2		3		4		5	
		F.C.	L.C.	F.C.	L.C.	F.C.	L.C.	F.C.	L.C.	F.C.	L.C.	F.C.	L.C.
Jeneberang River Improvement	MAIN WORKS	-	5,251.5	-	-	-	426.3	-	1,285.6	-	1,602.5	-	1,937.1
	PREPARATORY WORK	-	262.5	-	-	-	21.3	-	64.3	-	80.1	-	96.8
	SURVEY & SOIL TEST	-	525.3	-	-	-	42.6	-	128.6	-	160.3	-	193.8
	LAND ACQUISITION	-	51.7	-	-	-	7.0	-	12.1	-	4.1	-	28.5
	HOUSE EVACUATION	-	26.1	-	-	-	7.7	-	8.5	-	4.7	-	5.2
	EQUIPMENT	-	2,627.3	-	-	-	1,639.3	-	823.4	-	-	-	164.6
	SUB-TOTAL	-	8,744.4	-	-	-	2,144.2	-	2,322.5	-	1,851.7	-	2426.0
Drainage System Improvement	PANAMPU ROUTE	-	1,586.2	-	-	-	914.6	-	671.6	-	-	-	-
	JONGAYA ROUTE	-	2,102.4	-	-	-	143.0	-	858.9	-	1,100.5	-	-
	SINRIJALA ROUTE	-	690.2	-	-	-	-	-	-	-	-	-	690.2
	PREPARATORY WORK	-	219.0	-	-	-	53.0	-	76.5	-	55.0	-	34.5
	SURVEY & SOIL TEST	-	438.1	-	-	-	105.8	-	153.1	-	110.1	-	69.1
	LAND ACQUISITION	-	991.8	-	-	-	288.8	-	357.8	-	260.9	-	84.3
	HOUSE EVACUATION	-	151.2	-	-	-	85.2	-	42.6	-	23.4	-	-
	EQUIPMENT	-	1,094.2	-	-	-	1,094.2	-	-	-	-	-	-
	SUB-TOTAL	-	7,273.1	-	-	-	2,684.6	-	2160.5	-	1549.9	-	878.1
	CONTINGENCIES	-	1,601.8	-	-	-	482.9	-	448.3	-	340.2	-	330.4
	ENGINEERING WORK	1884.0	463.1	800.0	132.0	247.3	72.2	294.0	88.2	262.2	81.3	280.5	89.4
	TOTAL	1884.0	18,082.4	800.0	132.0	247.3	5,383.9	294.0	5,019.5	262.2	3,823.1	280.5	3,723.9
	GRAND TOTAL	19,966.4		932.0		5,631.2		5,313.5		4,085.3		4,004.4	

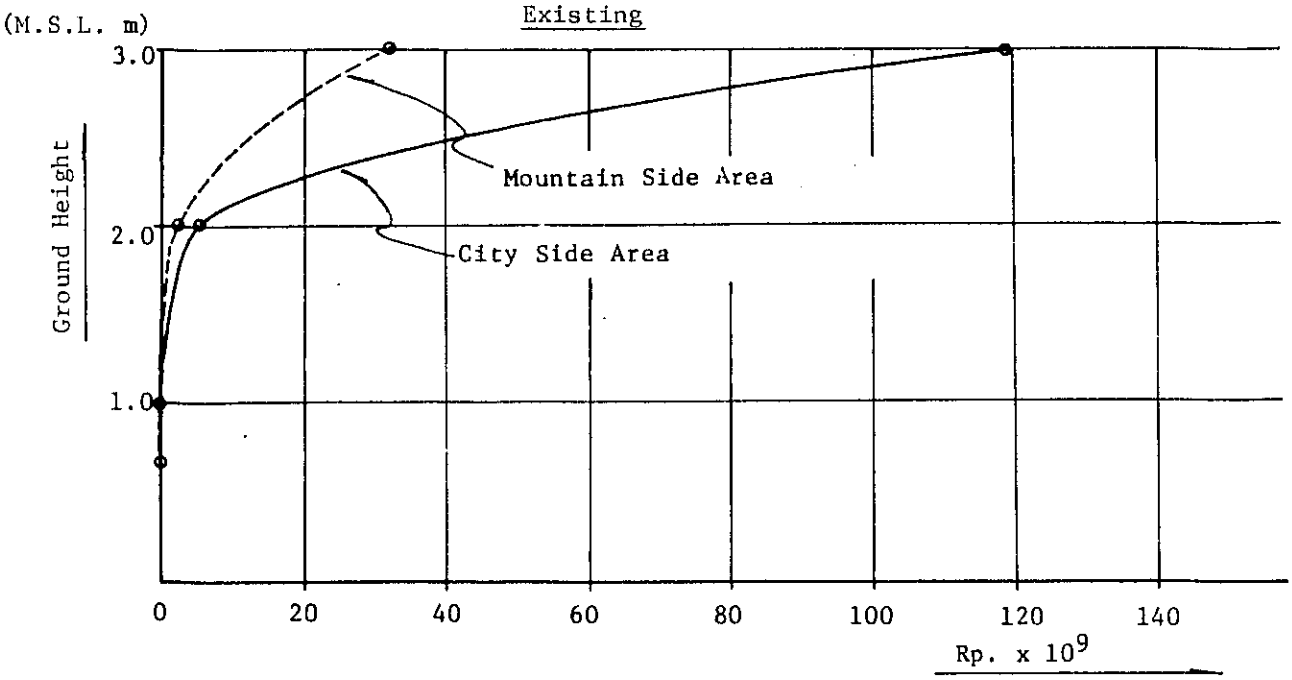
NOTE: Estimation all in local currency except for the foreign engineering services,
Escalation rate - 7% for F.C. and 10% for L.C., Conversion rate - 250 Yen = 1 US$

Table 6-1 ANNUAL DISBURSEMENT OF ECONOMIC COST AND BENEFIT

Unit: x10^3 US$

YEAR	COST				BENEFIT
	Engineering	Construction	O & M	Total	
1981	870.0	0.0	0.0	870.0	0.0
1982	275.5	1,986.9	0.0	2,262.6	0.0
1983	306.3	3,179.5	41.7	3,527.5	0.0
1984	255.5	2,596.7	44.4	2,896.6	0.0
1985	255.5	2,176.0	50.1	2,481.6	0.0
1986	0.0	0.0	55.0	55.0	2,018.0
1987	0.0	0.0	55.0	55.0	2,018.0
1988	0.0	0.0	55.0	55.0	2,018.0
1989	0.0	0.0	55.0	55.0	2,018.0
1990	0.0	0.0	55.0	55.0	2,018.0
1991	0.0	0.0	55.0	55.0	2,018.0
1992	0.0	0.0	55.0	55.0	2,018.0
1993	0.0	0.0	55.0	55.0	2,018.0
1994	0.0	0.0	55.0	55.0	2,018.0
1995	0.0	0.0	55.0	55.0	2,018.0
1996	0.0	0.0	55.0	55.0	2,018.0
1997	0.0	0.0	55.0	55.0	2,018.0
1998	0.0	0.0	55.0	55.0	2,018.0
1999	0.0	0.0	55.0	55.0	2,018.0
2000	0.0	0.0	55.0	55.0	2,018.0
2001	0.0	0.0	55.0	55.0	2,018.0
2002	0.0	0.0	55.0	55.0	2,018.0
2003	0.0	0.0	55.0	55.0	2,018.0
2004	0.0	0.0	55.0	55.0	2,018.0
2005	0.0	0.0	55.0	55.0	2,018.0
2006	0.0	0.0	55.0	55.0	2,018.0
2007	0.0	0.0	55.0	55.0	2,018.0
2008	0.0	0.0	55.0	55.0	2,018.0
2009	0.0	0.0	55.0	55.0	2,018.0
2010	0.0	0.0	55.0	55.0	2,018.0
2011	0.0	0.0	55.0	55.0	2,018.0
2012	0.0	0.0	55.0	55.0	2,018.0
2013	0.0	0.0	55.0	55.0	2,018.0
2014	0.0	0.0	55.0	55.0	2,018.0
2015	0.0	0.0	55.0	55.0	2,018.0
2016	0.0	0.0	55.0	55.0	2,018.0
2017	0.0	0.0	55.0	55.0	2,018.0
2018	0.0	0.0	55.0	55.0	2,018.0
2019	0.0	0.0	55.0	55.0	2,018.0
2020	0.0	0.0	55.0	55.0	2,018.0
2021	0.0	0.0	55.0	55.0	2,018.0
2022	0.0	0.0	55.0	55.0	2,018.0
2023	0.0	0.0	55.0	55.0	2,018.0
2024	0.0	0.0	55.0	55.0	2,018.0
2025	0.0	0.0	55.0	55.0	2,018.0
2026	0.0	0.0	55.0	55.0	2,018.0
2027	0.0	0.0	55.0	55.0	2,018.0
2028	0.0	0.0	55.0	55.0	2,018.0
2029	0.0	0.0	55.0	55.0	2,018.0
2030	0.0	0.0	55.0	55.0	2,018.0

Fig. 2-1(1) GROUND HEIGHT - ASSETS CURVE



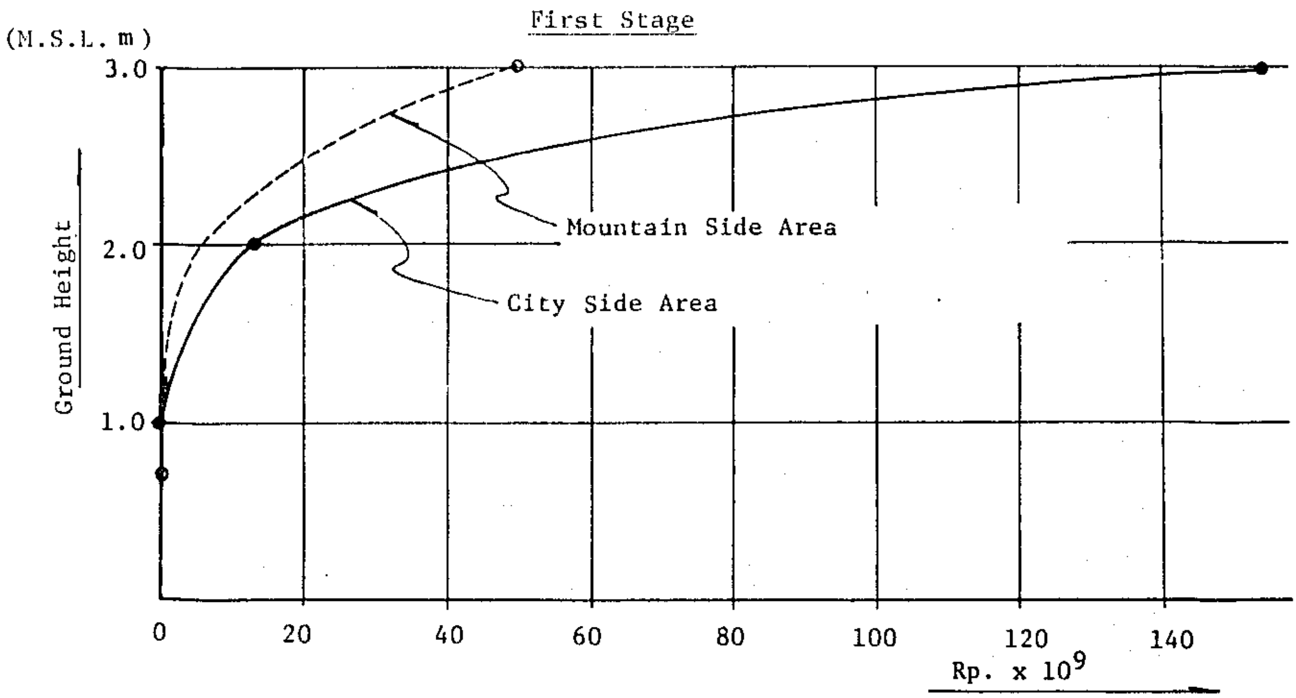
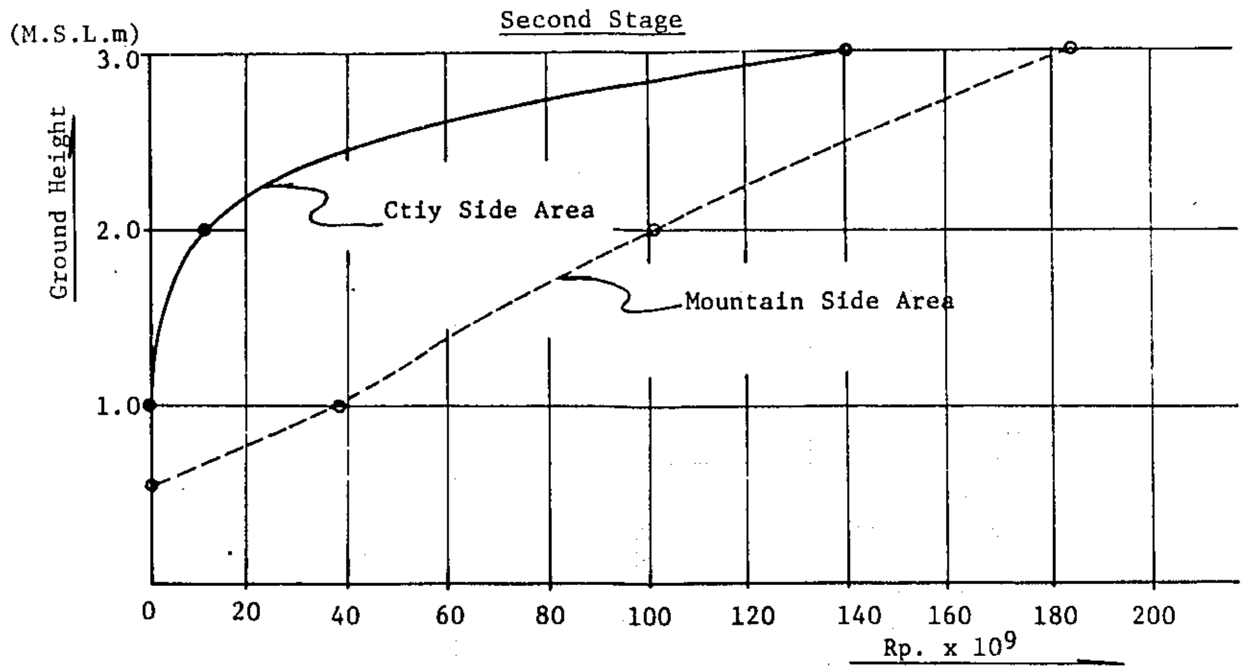


Fig. 2-1(2) GROUND HEIGHT - ASSETS CURVE



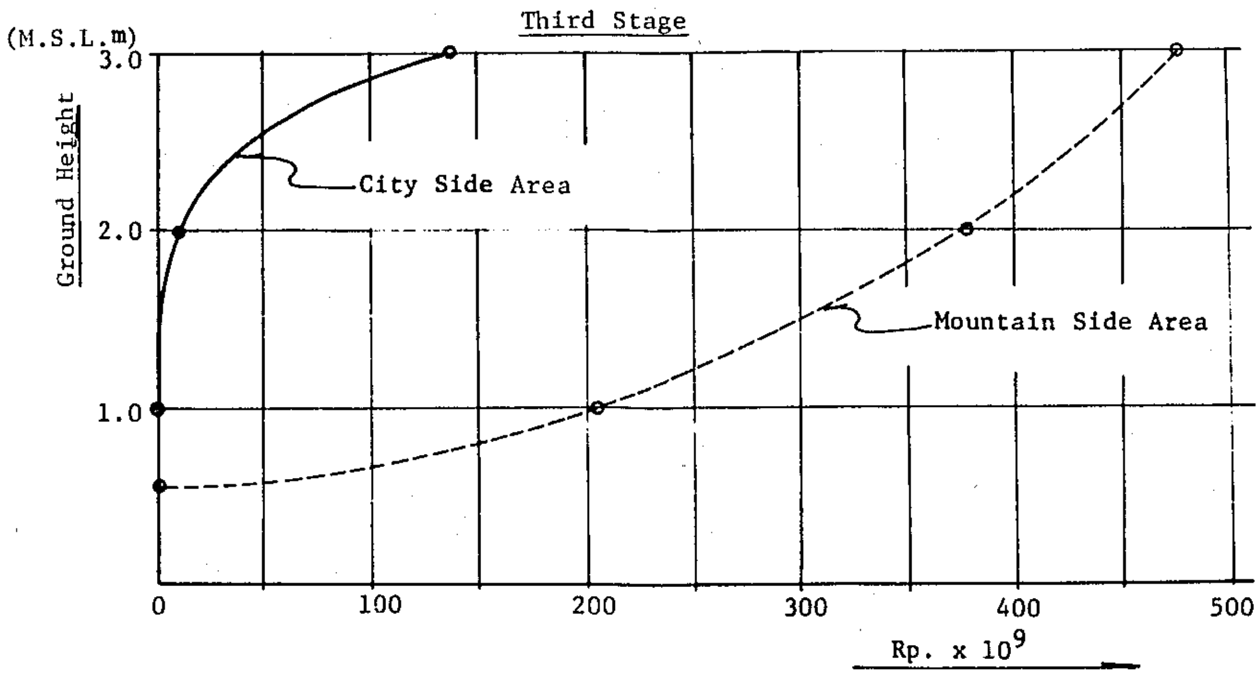


Fig. 2-2 PROJECT AREA AND DEVELOPMENT AREA

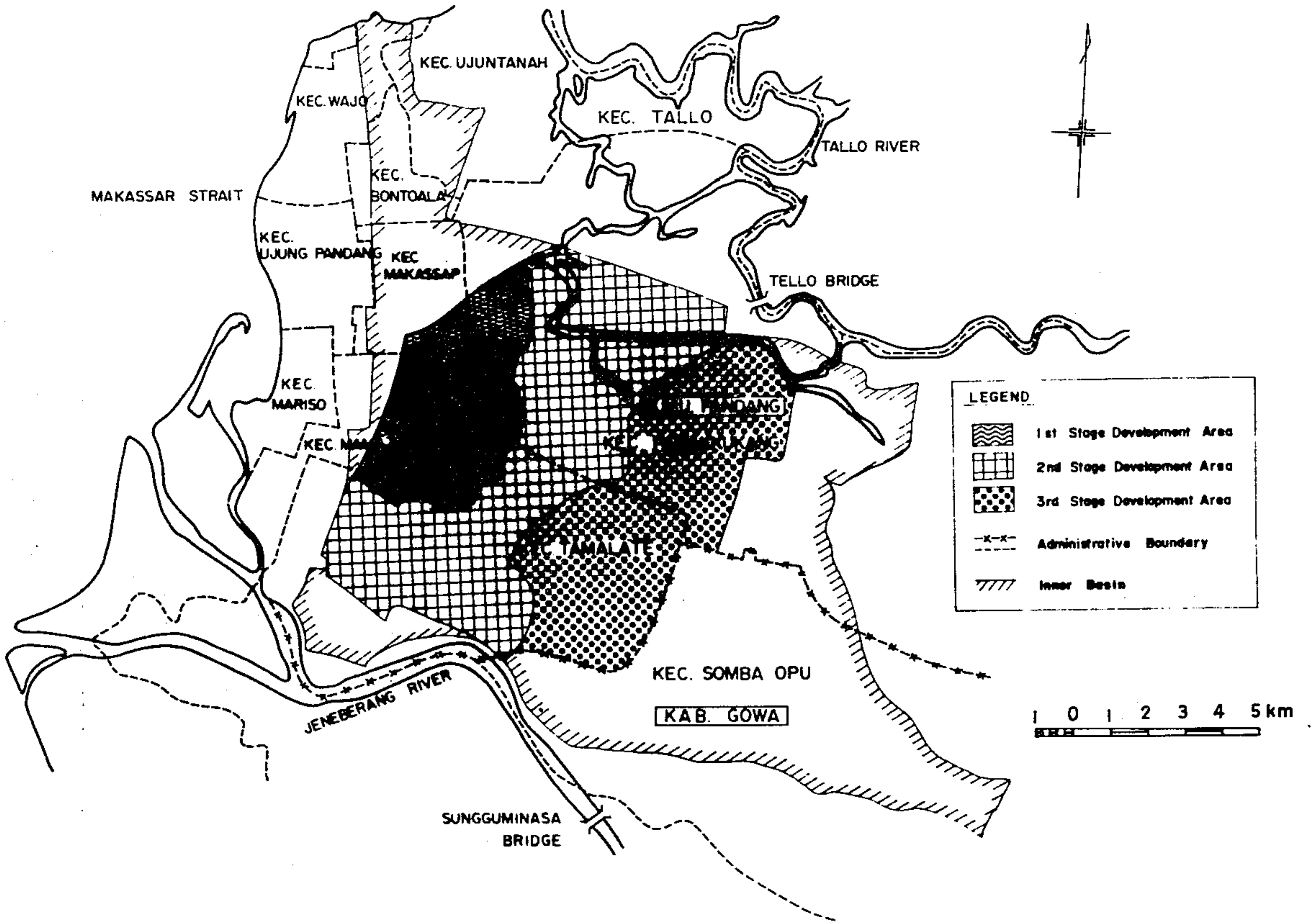


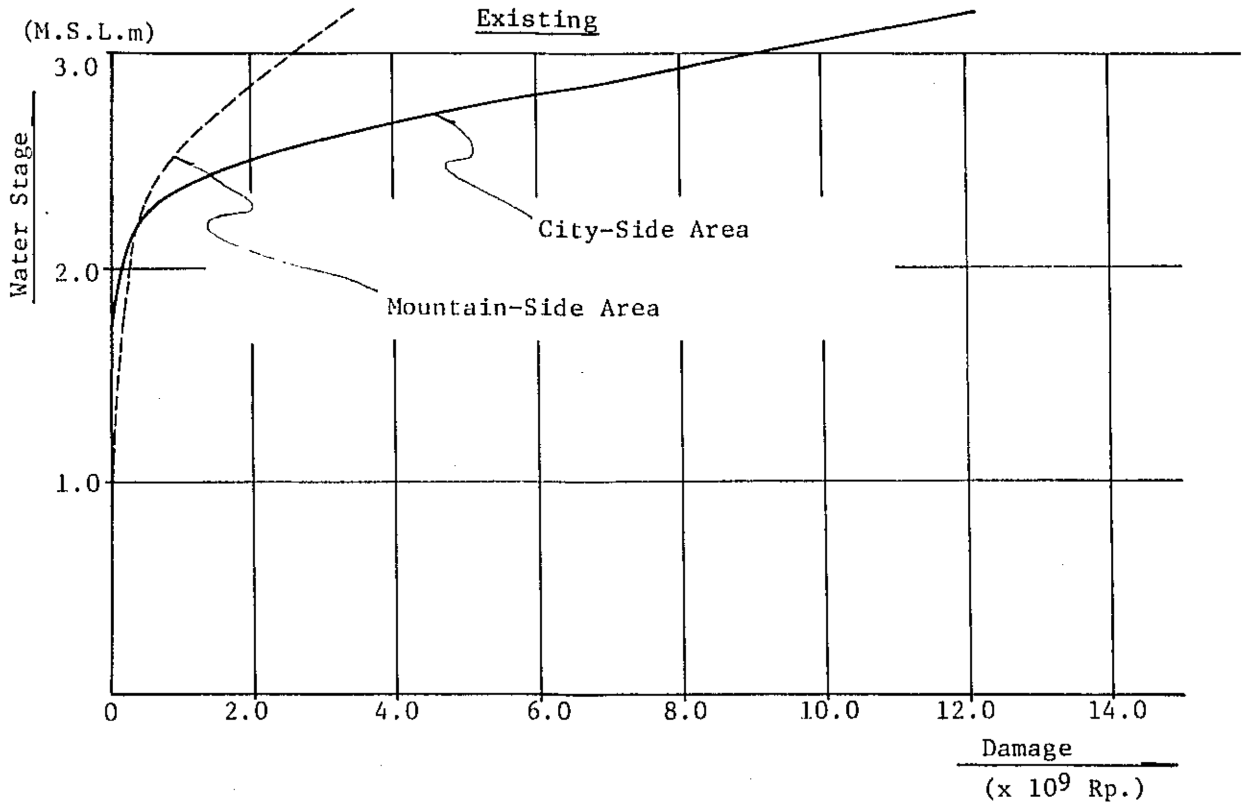
Fig. 3-1 PADDY PLANTING PATTERN

Planted Area (ha)	Month						
	D	J	F	M	A	M	J
70							
1,934							
1,546							
99							
Total Area Planted (ha)	70	2,004	3,550	3,649	3,579	1,645	99

Cultivated area in January :

2,004 / 3,649 = 0.55

Fig. 3-2(1) WATER STAGE - DIRECT DAMAGE CURVE
(Buildings and household effects)



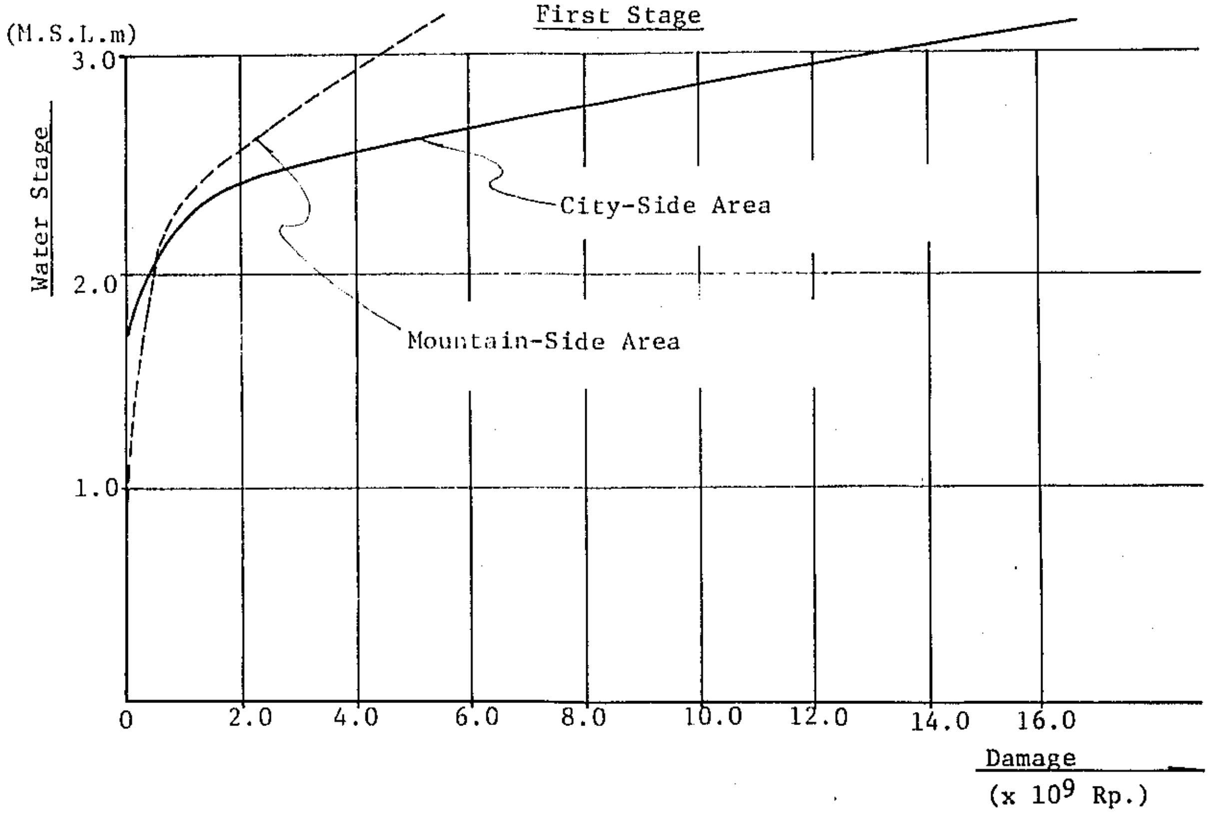
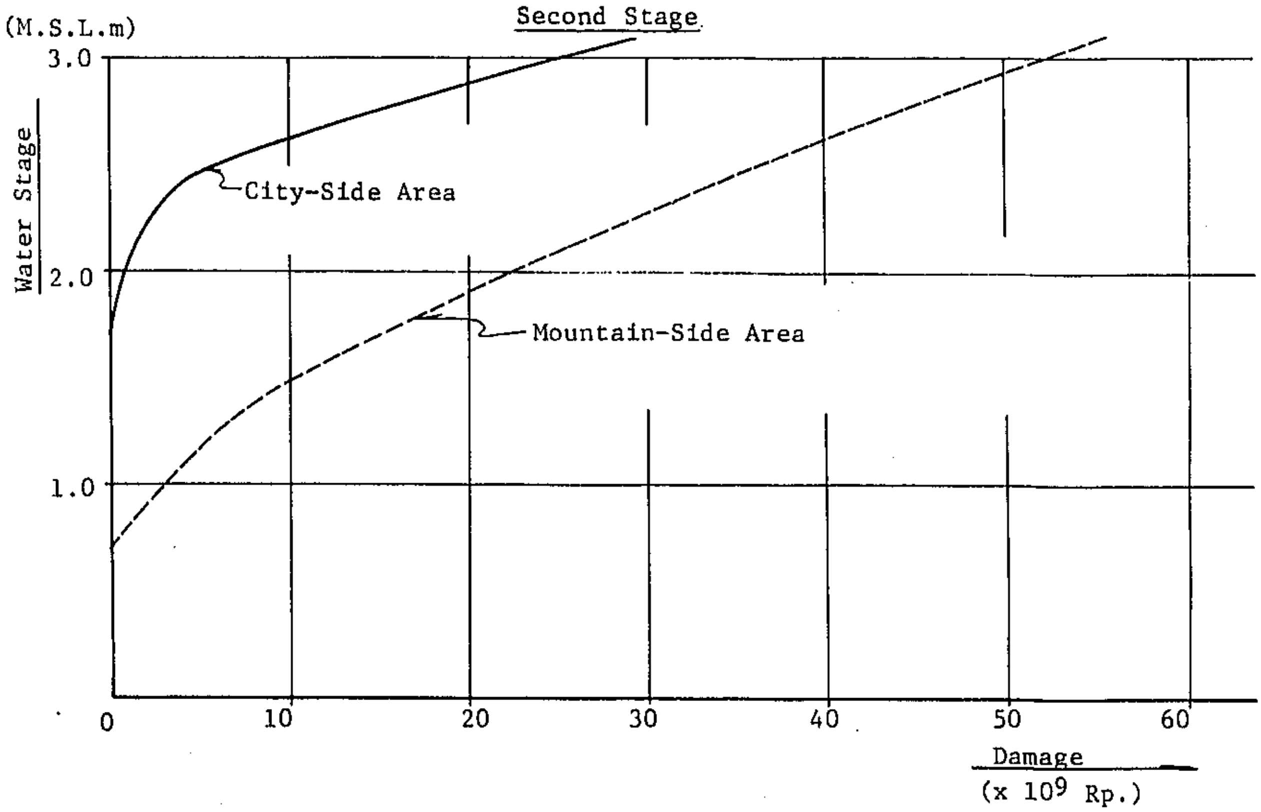


Fig. 3-2(2) WATER STAGE - DIRECT DAMAGE CURVE
(Buildings and household effects)



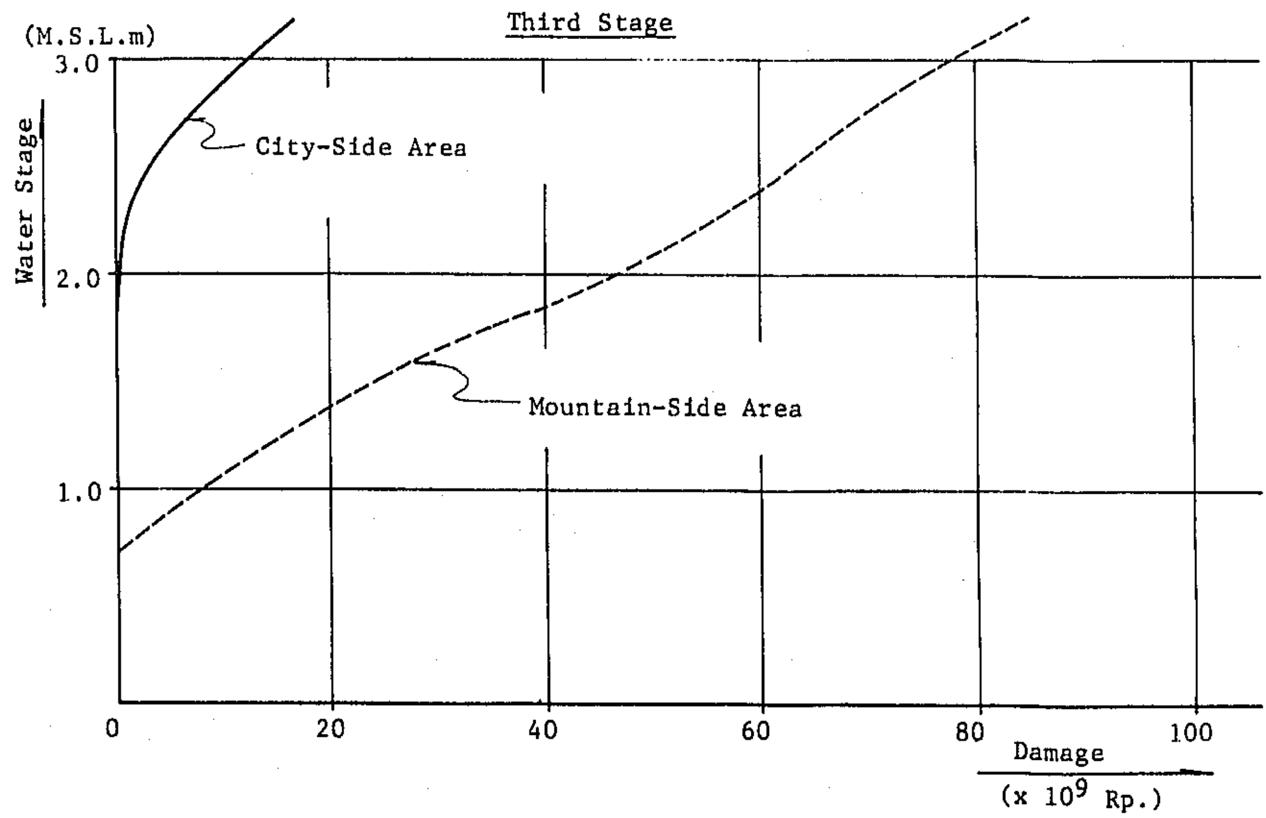


Fig. 3-2(3) WATER STAGE - DIRECT DAMAGE CURVE

(Form crops)