Location	Section Nos.	Excavation (x 10^3 m^3)		
		Plan-A	Plan-B	Plan-C
Around K. Samin	No. 15 - 24	1,079*	1,260	1,392
Around K. Brambang	No. 34 - 47	1,477*	1,576	1,456
Around K. Jlantah	No. 64 - 73	333*	458	360

Note: * Proposed short-cuts route

3.2.5 Longitudinal Profile

The most suitable longitudinal profile may be decided by taking into consideration the existing river profile; this is desirable for the maintenance of the stability of the channel. However, in the neighborhood of the Sala's confluences with the Jlantah and the Dengkeng, there occurs habitual inundation as the discharge capacity at these points is about 200 m^3/s and the river depth is only about 2 m. The extreme shallowness seldom seen in other river reaches is deemed to be the main reason why the discharge capacity is limited at these points and causes frequent and extensive inundation.

Furthermore, other details of the river as it exists today, such as the bridge piers, the longitudinal profiles of upper and lower reaches, the ground heights, the profiles and discharge capacities of both the main stream and tributaries have been taken into consideration, and the longitudinal profiles as shown in Fig. 3-2-7 are proposed for the entire distance of the river improvement section.

3.2.6 Cross Section

Single cross section usually requires the least flow area. However, in the case of the Sala river where the seasonal discharge fluctuation is extremely large, a composite cross section is recommendable in view of stabilizing the low water channel in medium and small floods.

In determination of the low water channel discharge, which governs the stability of the river channel, or the river-bed evolution, it is generally considered better to confine the said discharge. As the design low-water discharge, it is an accepted procedure to consider a probable discharge of once or twice a year.

The design width of the river is fairly wide because of the intention of effective utilization of the high water channel as farmland. Therefore, the low water channel was designed fairly large to reduce inundation of the high-water channel as much as possible.

Therefore, in determination of the design low water discharge for this Project, the use of a discharge as high as practically possible is recommended, taking stability of the river channel into consideration. In this proposed plan, 900 m^3/s of 2-year flood is used as adequate for the purpose.

The design criteria for the high water channel and the low water channel are described below:

a. The cross section should be of the composite type.

b. The low water channel discharges for 2,000 m^3/s-case and 1,600 m^3/s-case are 900 m^3/s and 600 m^3/s respectively.

c. Rivers in general tend to have natural meandering, especially at the times of flood, usually causing an extensive bank erosion. Therefore, the river width should be designed as wide as possible within the economically justifiable limits taking the future development program into consideration.

d. The height of the levee should be lowered against the damage potential in the basin and the danger to the bank itself. The design high water level has, therefore, been decided to be limited within 2 m in height above the ground height as a rule; (refer to Fig.);

e. The slope of the levee for both high water channel and low water channel is fixed at 1:3 for their maintenance; (refer to Item);

f. The width of the levee crest should have enough width for its own security as well as to make the maintenance road on it. Thus, the width of levee crown has been decided at 7 m and the width of the outer banquette at 5 m on which the maintenance road will be constructed.

The cross section of the high-water-channel and the low water channel is decided by the criteria mentioned above and according to Figs. 3-2-8 and 3-2-9.

The results are shown in Figs. 3-2-10 and 3-2-11 as the standard cross section. The river width is required to be 300 m for 2,000 m^3/s discharge.

As the channel designing has important bearings on the cost of construction and the channel stability for sediment, some studies have been made on these points.

1) Construction cost

The construction cost has been estimated from such items as the amount of excavation, banking and spoil bank since they must occupy the main part of the cost.

The earthwork cost is examined for the three cases; the 2,000 m^3/s-case and 1,600 m^3/s-case and the 0.5 m-lowered case of 2,000 m^3/s-case.

The 0.5 m-lowered case means that the whole section of 2,000 m^3/s is lowered to 0.5 m except the reach of Jurug and of Nguter to reduce the H.W.L. of the middle part of reach to be improved. The results are shown in Table below.

Item	2,000 m^3/s-case*		0.5 m-lowered case		1,600 m^3/s-case	
	Quantity	Cost	Quantity	Cost	Quantity	Cost
	m^3	US$	m^3	US$	m^3	US$
Excavation	6,814,400	10,194,000	9,845,100	14,728,000	5,213,500	7,799,000
Banking	3,497,200	4,270,000	2,647,600	3,233,000	3,497,200	4,270,000
Spoil Bank	407,900	424,000	359,300	374,000	407,900	424,000
Total Cost	-	US$ 14,888,000	-	US$ 18,335,000	-	US$ 12,493,000

Note: * = proposed plan

Item	Equation	2,000m^3/s-case*	0.5m-lowered case
Excavation	(1)	6,814,400 m^3	9,845,100
Main levee banking	(2)	3,494,200	2,647,600
Tributary levee banking	(3)	2,488,200	1,891,000
Remained soil	(4)=(1)-(2)-(3)	832,000	5,306,500
Ratio	(5)=(1)/(2)+(3)	114%	217%

Note: * = proposed plan

2) Stability

For stability of river bed, a study has been made at around the confluence with K. Dengkeng and at around Jurug reach to find out whether the channel is in scouring condition or in depositing condition after being controlled by the Wonogiri dam.

The result of the study is shown in Table below and it shows that Jurug reach may have a depositing tendency, with annual sand deposition estimated at 1,400 m^3/year, since the backwater influence due to the narrow sections at the Jurug Bridge and the section of the river course downstream of the project area makes the transportation ability of sand to downstream reach weaker than in upstream reach.

No.	Reach	Transportable bed-load (m^3/year)	Supplied bed-load (m^3/year)
1.	Jurug	26,800	39,270
2.	Confluent with K. Samin	40,800	36,057
3.	Downstream of Confluent with K. Dengkeng	20,900	24,234
4.	K. Dengkeng	31,000	17,493
5.	Upstream of Confluent with K. Dengkeng	4,000	6,741

Note: supplied bed load = Area x 21 m^3/km^2/year

3.2.7 Flood-Control Facility

Retarding basin and open-levee are the suitable facilities to mitigate the flood hydrograph, to lower the peak discharge and to reduce the flood in the downstream area of Surakarta.

Furthermore, an emergency inundation area may have to be provided against the floods of larger scales. Accordingly, 2 retarding basins and 3 mergency inundation areas have been proposed in the plan to lower the flood peak discharge.

1) Retarding basin

The 2 retarding basins are planned in the downstream of the confluence with the K. Dengkeng by utilizing the existing river channel. Their areas are 1.35 km^2 and 0.90 km^2, with respective flood control volumes of 2.7 x 10^6 m^3, the former for the upstream and the latter for the downstream.

The second levee along the channel is built to a height of 0.5 m to effectively retard the middle class floods and minimize the frequency of ponding of the farmland.

Fig. 3-2-6 shows how to drain rain water from the retarding basin. The rain water is drained through drainage channels and sluiceways. The diameter and number of the sluiceway pipes are 60cm x 2 in the upstream area and 60cm x 1 in the downstream retarding basins.

The effect of the retarding basin is checked for three types of floods:

Very sharp hydrograph .... imaginary 1966 flood

Rather sharp hydrograph .... 1966 flood

Gentle hydrograph .... 1968 and 1975 floods

The results as shown in Fig. 3-2-13 show that the effect of flood control by retarding basins is conspuous when the flood discharge is 1,100 m^3/s or over, and an average peak cut of 150 m^3/s is expected for the peak discharge of 2,000 m^3/s at Jurug site.

Much difficulty is encountered with when estimating the silt deposit in a retarding basin. In most cases of short-cutting in Indonesia, it is quite usual to have the downstream end of the abandoned meandering section left open and allow the old channel to be gradually and naturally filled with silt deposit carried by the river water. It is believed that it takes about 7 to 8 years before the old channel is practically filled. In the case of a retarding basin, no inundation will occur under 3-year flood. In another words, there will be about 17 to 18 cases of inundation in a 50-year period. With a higher discharge the volume of silt transferred is usually considered greater, however, the volume of silt deposit in the retarding basin is assumed to be not very large because of the low frequency of inundation. This is why the expected life time of the retarding basin is believed considerably long.

Construction of 2 retarding basins is also economically justifiable.

2) Open-levee

The two open-levees were taken into consideration, together with the two retarding basins. Open-levees were disregarded as their flood control effect was not so remarkable as shown in Fig. 3-2-13; the peak cut effect was only 50 m^3/s for 2,000 m^3/s or one third of the effect of retarding basin. Therefore, open-levees have not been adopted in the proposed plan.

3) Emergency inundation area

Since there is a succession of valleys on the channel of the K. Walikan, the rain water over the catchment area has a tendency togather into the channel almost directly causing a possible discharge well over the design discharge. Therefore, the emergency inundation areas are planned to secure safety of the river channel and the downstream areas.

Since, the topography of the K. Walikan is considerably steep, the discharge hydrograph is considered sharp and damage expected on the overflow sections of the levee may not be too serious.

The 3 emergency inundation areas are adopted in the proposed plan. Their locations are shown in Fig. 3-2-12. In these areas, the levee height is set up to the high water level for the distance of one km so that the overflowing may take place only gently, causing less scouring of the levee.

4) Effect of peak-cut at Jurug

Because of the peak discharge cut by these flood-control facilities, probable discharge in proposed channel at Jurug site is reduced. The results are as shown in Fig. 3-2-14 and Table below.

Return period (year)	Probable discharge (Jurug site)	
	Existing condition	Proposed channel
	m^3/s	m^3/s
200	2,500	2,200
100	2,300	2,000
50	2,030	1,850
40	2,000	1,800
20	1,750	1,600
10	1,500	1,400
5	1,250	1,200
2	900	900

5) Land utilization

Land utilization in the retarding basin and emergency inundation areas shall remain subject to applicable regulations concerning land use. This is because of possible inundation once in every 3 years and 40 years in the retarding basin and in the emergency inundation area, respectively. The land in the retarding basin is not suitable for constructing residential quarters as the area is struck by frequent floods. The emergency inundation area is ideal for a farmland but not suitable as the sites for the construction of such tangible assets as factories. It is recommended that whenever possible, such public buildings as schools, hospitals, government offices and some residences are to be constructed at locations as high as practically possible.

3.2.8 Tributary

There are 9 tributaries along the prposed reach for improvement. It is proposed that only the reach which is influenced by the backwater from the main river should be protected with the back levee. The study shows that the length of the required back levee is 30.5 km in total, as shown in Table below.

Name of tributary	Measuring No. of confluence	Levee - length (km)		
		Left	Right	Average
K. Jlantah	No. 65	3.8	3.8	3.8
K. Dengkeng	No. 49	3.2	3.2	3.2
K. Gawe	"	3.8	3.8	3.8
K. Pusur	No. 45	3.8	3.6	3.7
K. Buntungan	No. 44	3.9	3.9	3.9
K. Brambang	No. 38	3.1	3.1	3.1
K. Kembangan	No. 33	1.6	1.6	1.6
K. Samin	No. 19	4.7	4.7	4.7
K. Wingko	No. 14	1.7	3.7	2.7
Total				30.5

In the study, special attention has been paid to the treatment of K. Buntungan and K. Wingko. K. Buntungan has a rather small basin area of 20 km^2. But it needs to be protected with the back levee otherwise the landside water of the basin flows directly into the landside water area of K. Brambang, raising its landside water level higher than the H.W.L. of the 1966 flood due to the steep slope of land and small strage capacity of basin.

About the K. Wingko, if it is not protected with a back levee, a big gate having 120 m^3/s discharge capacity corresponding to the design discharge of K. Wingko itself should be set up at its mouth, costing much more than constructing a back levee.

Name	Area (km^2)	Confluent discharge in the design flood (m^3/s)	Proposed design discharge (m^3/s)
K. Jlantah	75.0	130	350
K. Dengkeng	833.0	285	830
K. Pusur	43.0	110	290
K. Buntungan	20.0	90	220
K. Brambang	125.0	165	410
K. Kembangan	38.0	100	280
K. Samin	305.0	220	580
K. Wingko	57.0	120	320

3.2.9 Riparian Structures

1) Bank protection

At present, the banks are eroded at many places, especially at the sharply meandering sections, along the river course. Bank protection works are not provided except wooden groins at a few places.

The proposed river improvement works involve the construction of the long continuous levees along the both banks of the river. Both the levees and banks will be protected by sodding.

The stone pitching is planned at the confluences with the main tributaries and at bridge points to protect the bridge embankment, but the bank protection of the low water channel will be done mainly by groins for the following reasons:

a. As the result of short-cutting in the long reaches of the river channel, the proposed river channel will be put in quite a different condition compared with the present. This makes the assumption of the whereabouts of the points attacked by the flowing water along the proposed channel very difficult. Moreover, when the banks are protected by the revetment along the sections where the flow attacks, the revetment base will be undermined and the flow changes its attacking points. The alternative would be the lengthening of the revetment for the protection of the river banks, which is very costly.

b. The groin primarily regulates the stream direction and secondly protects the banks. Therefore, it is easier to protect the section from the attacks of the flow by groin rather than by revetment.

2) Landside water drainage facilities

a. Main drainage channel

Construction of the main drainage channel is planned for the distance of 74 km in total, along the main stream, its tributaries and in the 2 retarding basins. The cross section of the main drainage channel is determined by using the uniform flow formula, on the basis of the relations between area and specific discharge as shown in Fig. 3-2-25. The proposed route and the design discharge of the main drainage channel are shown in DWG No. RW40.

b. Sluiceway

The landside water gathered by ditches and drainage channels will be drained to the main channel and tributaries through the sluiceways. From the safety point-of-view of the levees at the time of flooding, it is not advisable to install the sluiceway at too many places. Therefore, the sluiceway will be installed at 35 sites as shown in DWG No. RW40, in full consideration of the alignment of the existing ditches and drainage channels. Each sluiceway is provided with gates to shut-off the reverse flow from the river into the landside area.

3) Ground-sill

The purposes of the ground sill are two:one is for protecting the river bed from scouring, and another is for the fixation of flow area in the low-water channel, especially in the dry season.

The former is called the Ground-sill (1) and the latter, the Ground-sill (2) here.

a. Ground-sill (1)

The bed load which has been sent down to the Nguter railway bridge at present will be mostly deposited in the Wonogiri dam and the Colo afterbay. In anticipation of the scouring of the channel bed of the main stream, the ground-sill are planned 20 m downstream of Nguter railway and road bridge to protect to be constructed their foundation. The locations of the Ground-sill (1) are shown in Fig. 3-2-26.

b. Ground-sill (2)

The Ground-sill (2) is proposed at three places along the proposed reach as a test case, combining with the groins. To say anything definite about their setting positions is premature since the forecasting of severe scouring positions is very difficult at this stage. Therefore, their positions should be decided in the construction stage by observing variation of river bed and of flow course.

4) Bridge

a. New bridge at Nguter

New road bridge planned under Bina Marga is to cross the existing channel in parallel with the Nguter road bridge at its immediate downstream. On the other hand, short-cutting is planned at the reach near the Nguter road bridge under the present river improvement project on the ground that the river channel is heavily meandering in that place and the flood flows had often over-topped the river banks running on farmlands, roads, etc. Three alternative plans have been compared with a bridge which will be built after short-cutting (refer Fig. 3-2-26).

They are:

Plan A - In case the short-cutting is made for the river improvement works,

Plan B - In case the river improvement work is carried out along the existing river channel, and

Plan C - In case a bridge is built over the river channel just downstream of the Nguter railway bridge and the road is relocated accordingly.

The following conclusions have been arrived at as the result of comparative studies:

Plan B - This plan requires the lowest construction cost, but not feasible on the ground that the river is meandering too heavily at this point where has to be narrowed to about 90 m. (Proposed bridge span)

Plans A & C - Both plans will not hinder the flow at the time of flood of a discharge below the proposed design discharge. Both plans cost almost the same amount.

Consequently, the plan-A is adopted in the proposed plan considering the channel course which has reasonable stream line in the existing river especially at the short cut starting point.

Table 3.2.2 shows the construction cost of these 3 alternative plans.

b. Bacem Bridge

The discharge capacity of the channel at the Bacem Bridge site is not sufficient for the design flood. Moreover, the bottom of the bridge beam is 1 m lower than the proposed height of the levee-crest, that is to say, as the levees have 1 m free-board, the bottom of the bridge beam will spread at the same level with the proposed high water level. On the other hand, the length of the existing bridge is 115 m while the proposed river channel is 300 m wide. Bacem Bridge should, therefore, be reconstructed before or simultaneously with the implementation of the river improvement work. From the view-point of the river improvement work, a new bridge should be of the following height and length:

a) Height of the bottom of the bridge beam .... S.H.V.P. 92.32 m

b) Length of the bridge .... 297 m

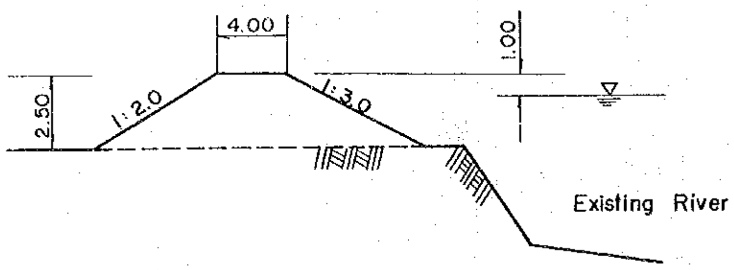
c. New road bridge at Jurug

A new road bridge is being planned to be constructed on the immediate upstream of the existing Jurug road bridge under Bina Marga. While the width of the proposed river channel at the bridge site is 318 m, the existing road bridge is forming a narrow section 168 m long. Consequently, the bridge causes the backwater reaching as far as the confluence with the K. Samin at the time of flooding. The river channel downstream of the bridge is also responsible for the backwater as it has insufficient discharge capacity to safely flow down a large amount of flood water.

Because the cross section at the existing bridge has the capacity to flow down the design flood discharge, the height and width of a new bridge may be planned on the same scale as at present. However, in anticipation of a future situation, it is advisable to plan a new road bridge so that it may be lengthened by jointing.

3.2.10 A Study on the Construction of a Levee at the Down Stream of Jurug Bridge

There is a recreation center on the left bank side downstream from the Jurug bridge. This recreation center consists of amusement facilities, auto race circuit, etc. for the citizens of the Surakarta city. Topographically, the area in which this recreation center is located is low, and it is subject to floods. To protect this area from floods, it is required to construct a levee of the following cross section.



The total length of the levee will be approx. 600 m, and the volume of excavation and banking is approx. 15,600 m^3. Area of the slope will be approx. 8,000 m^2. Construction cost is as follows;

Excavation & Banking	= 15,600 m^3 x 3 \$/m	= \$47,000
Sodding	= 8,000 m^2 x 0.36 \$/m	= \$3,000
	Total	= \$50,000

It is unable to judge whether the above work is suitable since the flood in this area is confined to the recreation center only. Furthermore, it was very difficult to evaluate the recreation center economically as no data had been made available at the timely the study was made. Therefore, this plan is not included in this improvement work plan. However, this matter may be taken up again in the stages of the detailed design.

Fig-3.2.1 Design Discharge Distribution (2,000 m³/s - Case)

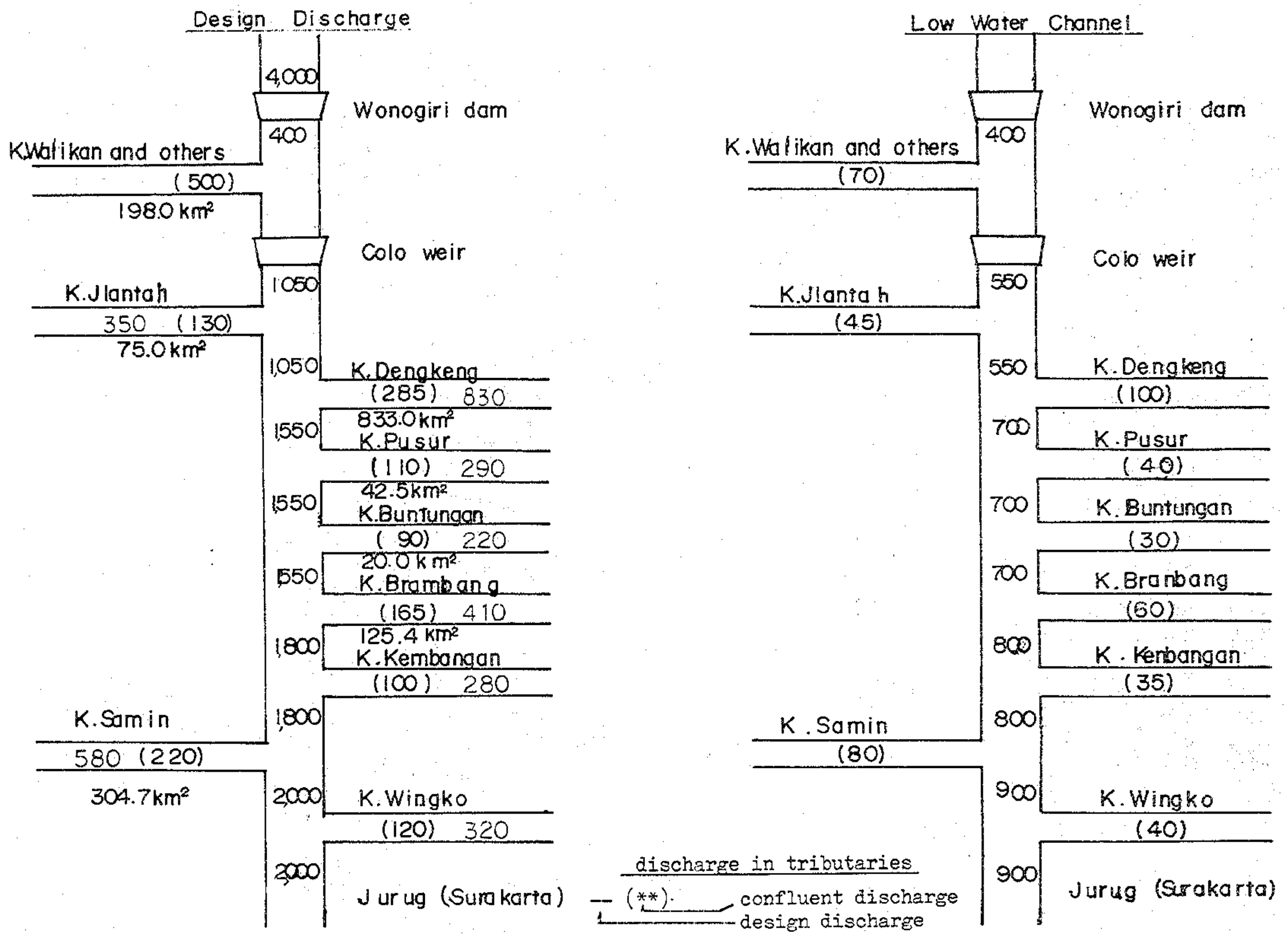
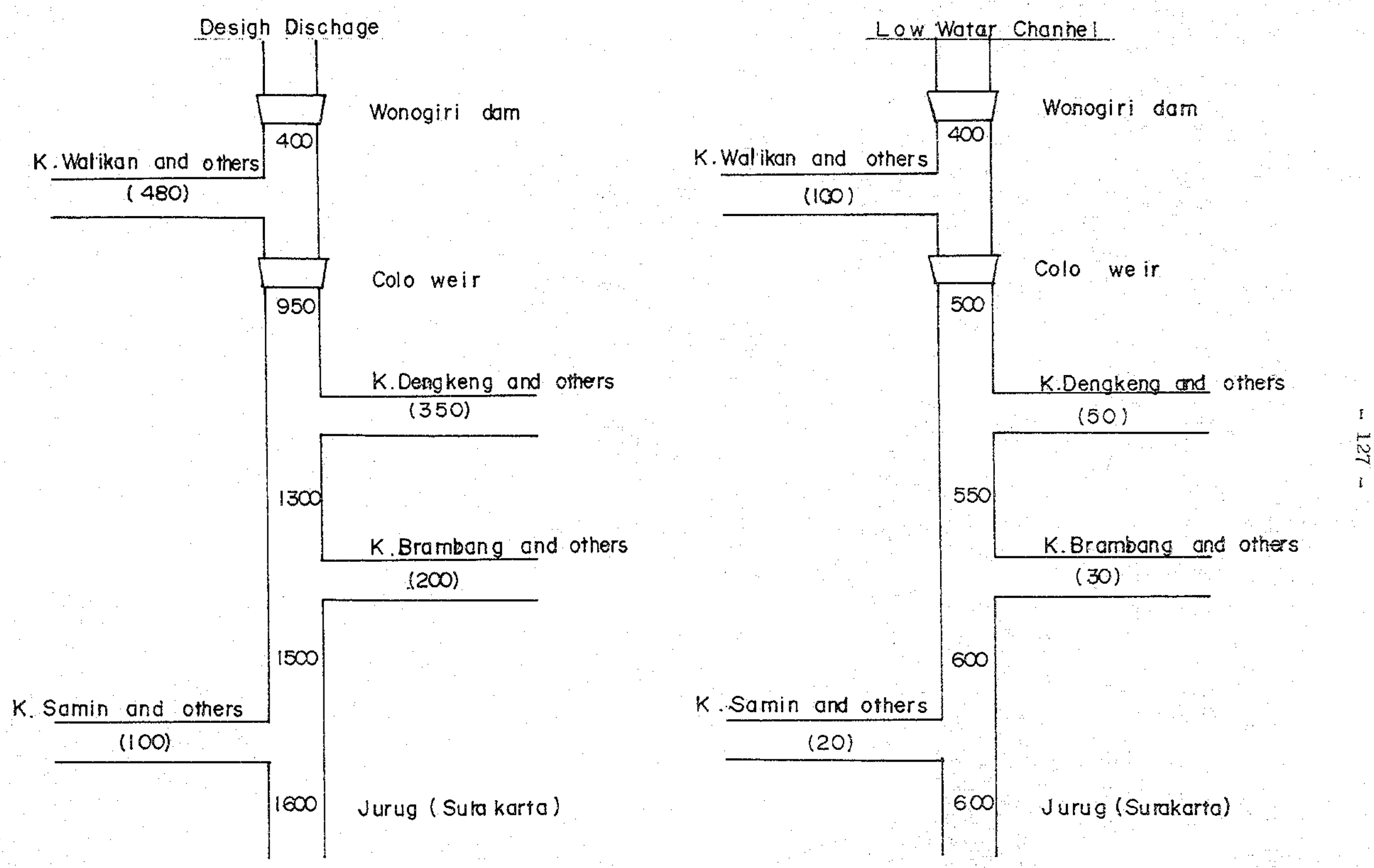


Fig 3.2.2 Desigh Discharge Distribution (1,600 m^3/s - Case)



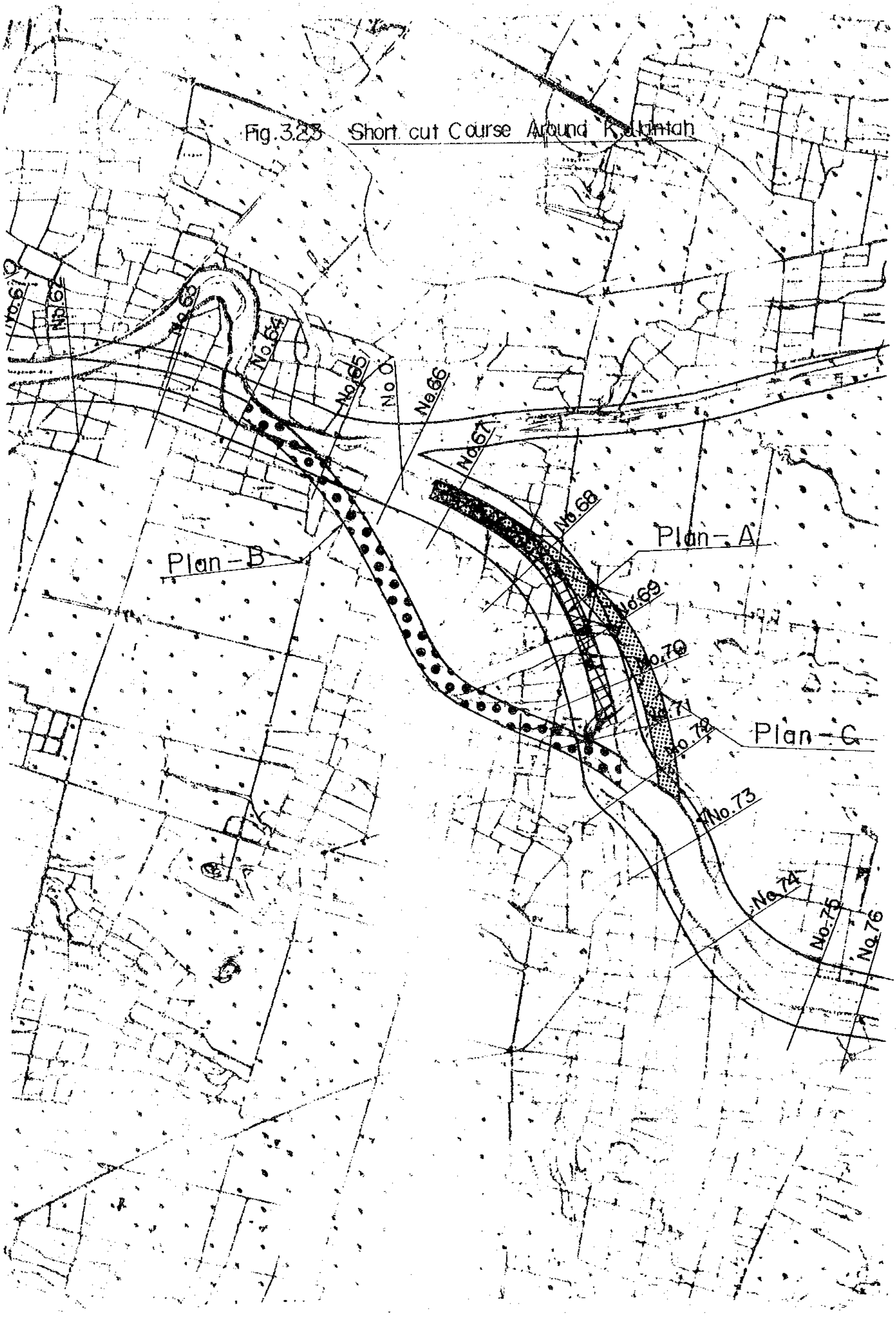


Fig. 3.23 Short cut Course Around Kalimtah

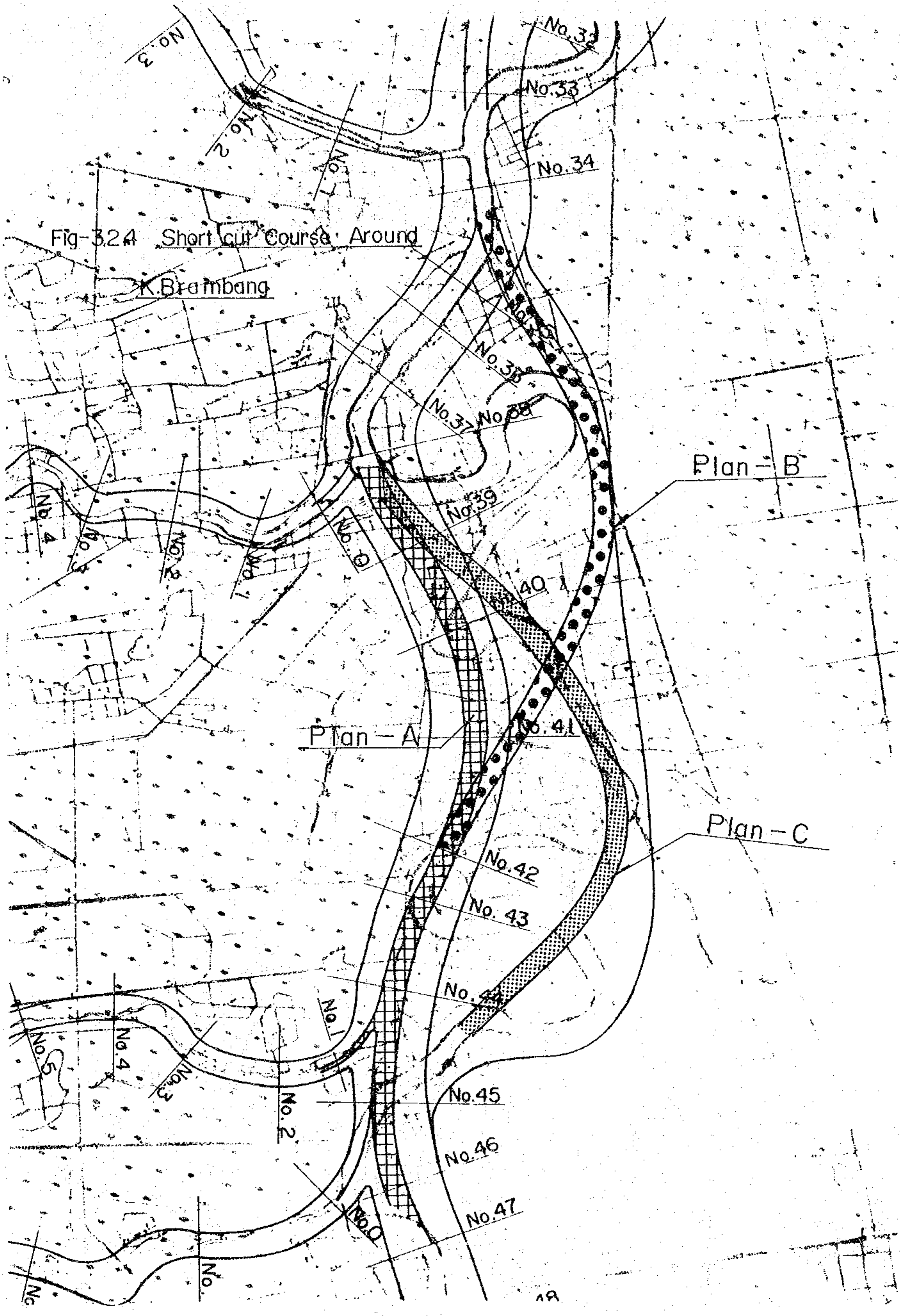


Fig-3.2.4 Short cut Course Around
K.Brambang
Plan - A
Plan - B
Plan - C
No.32
No.33
No.34
No.36
No.37
No.38
No.39
40
No.41
No.42
No.43
No.44
No.45
No.46
No.47
No.0
No.1
No.2
No.3
No.4
No.5

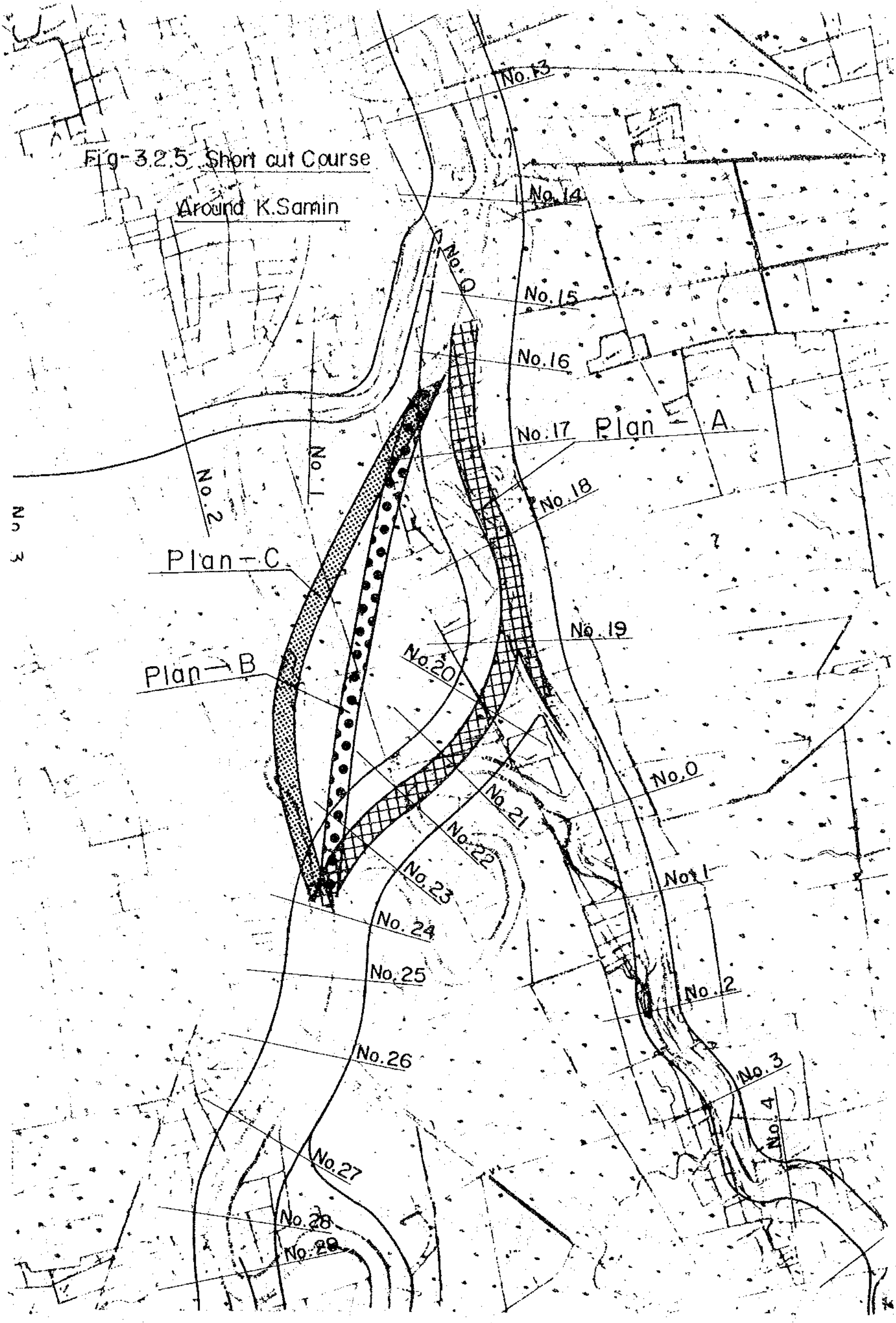
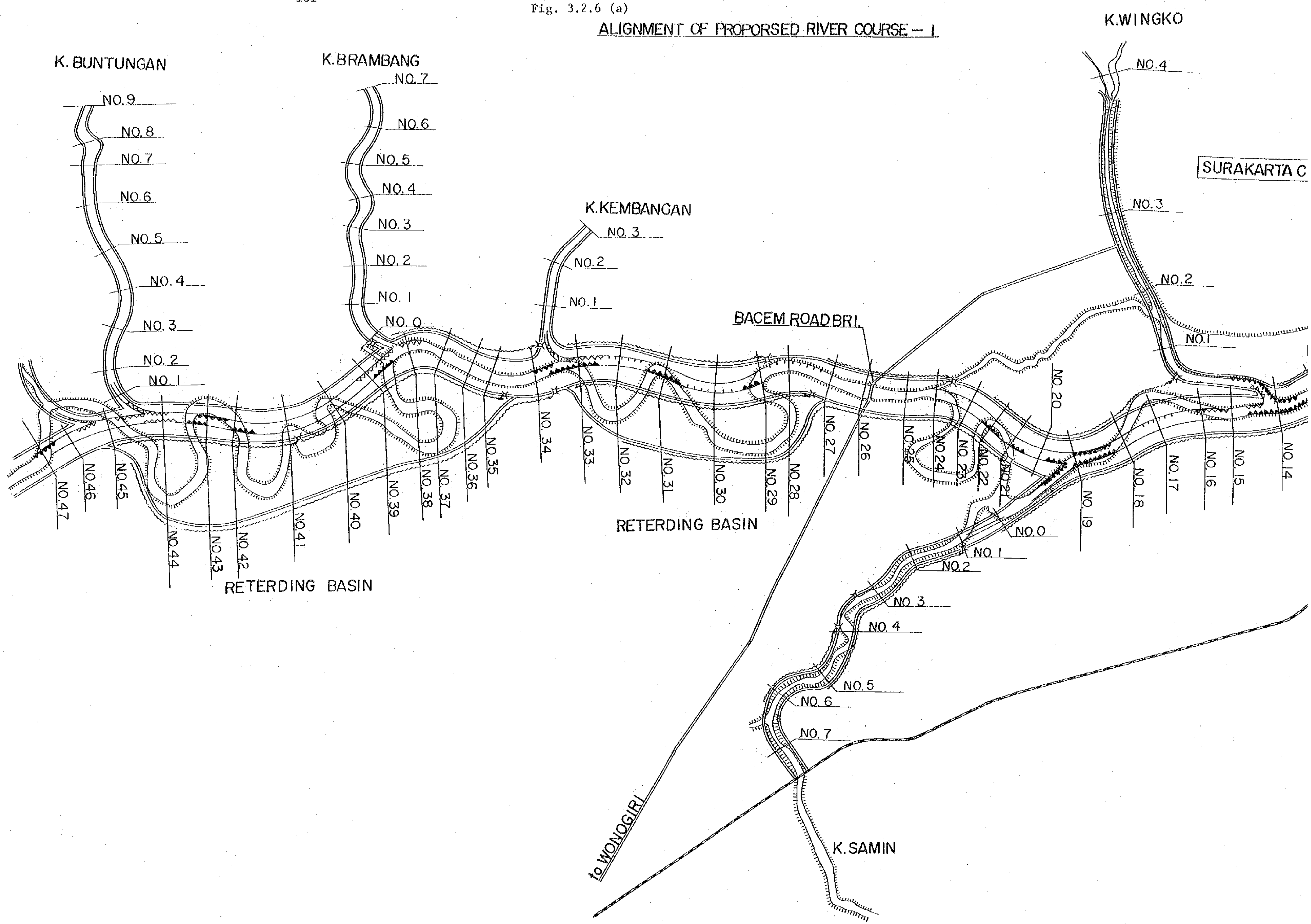


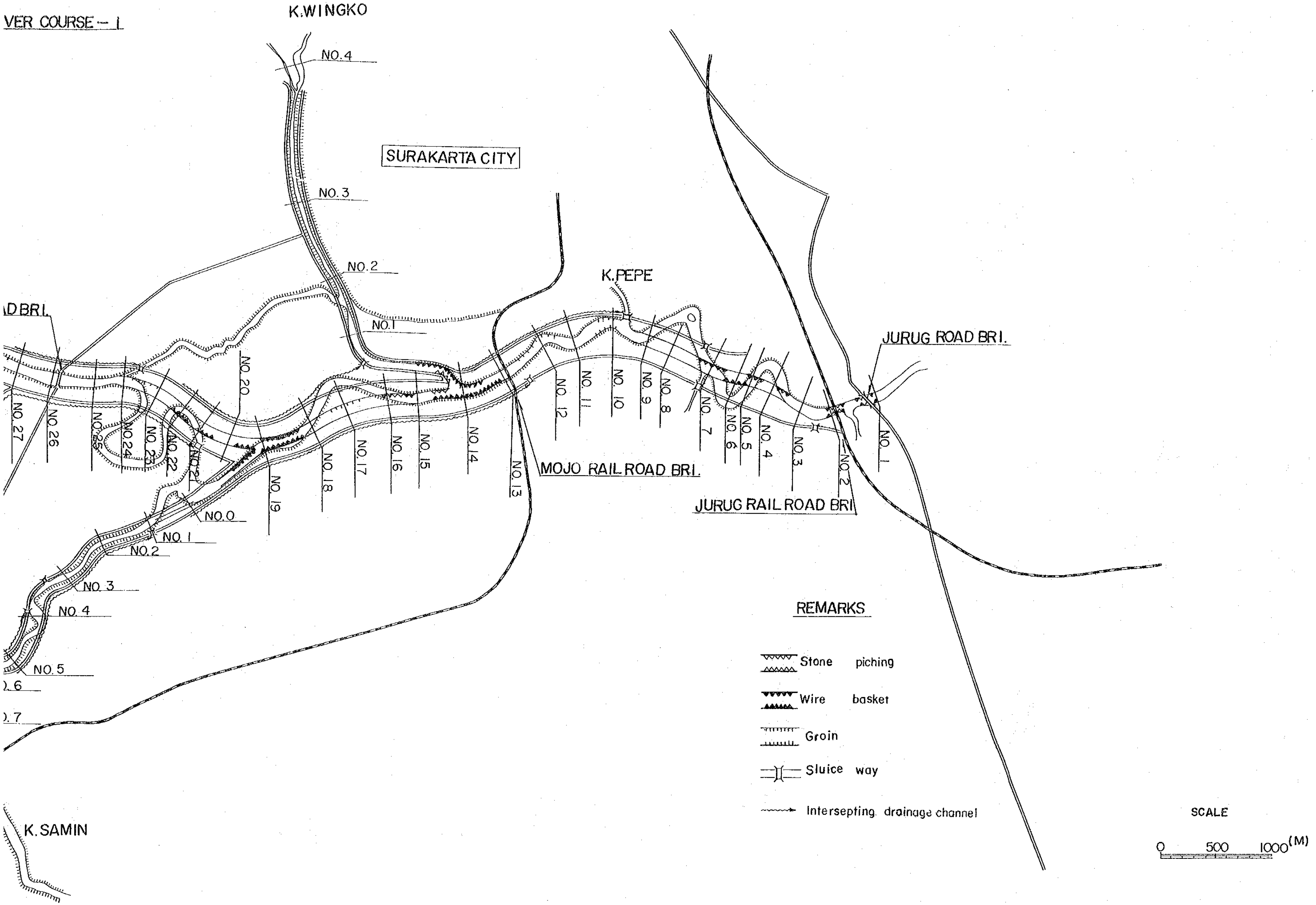
Fig-3.2.5 Short cut Course
Around K.Samin
Plan - A
Plan - B
Plan - C
No.13
No.14
No.15
No.16
No.17
No.18
No.19
No.20
No.21
No.22
No.23
No.24
No.25
No.26
No.27
No.28
No.29
No.0
No.1
No.2
No.3
No.4

Fig. 3.2.6 (a)

ALIGNMENT OF PROPORSED RIVER COURSE – I



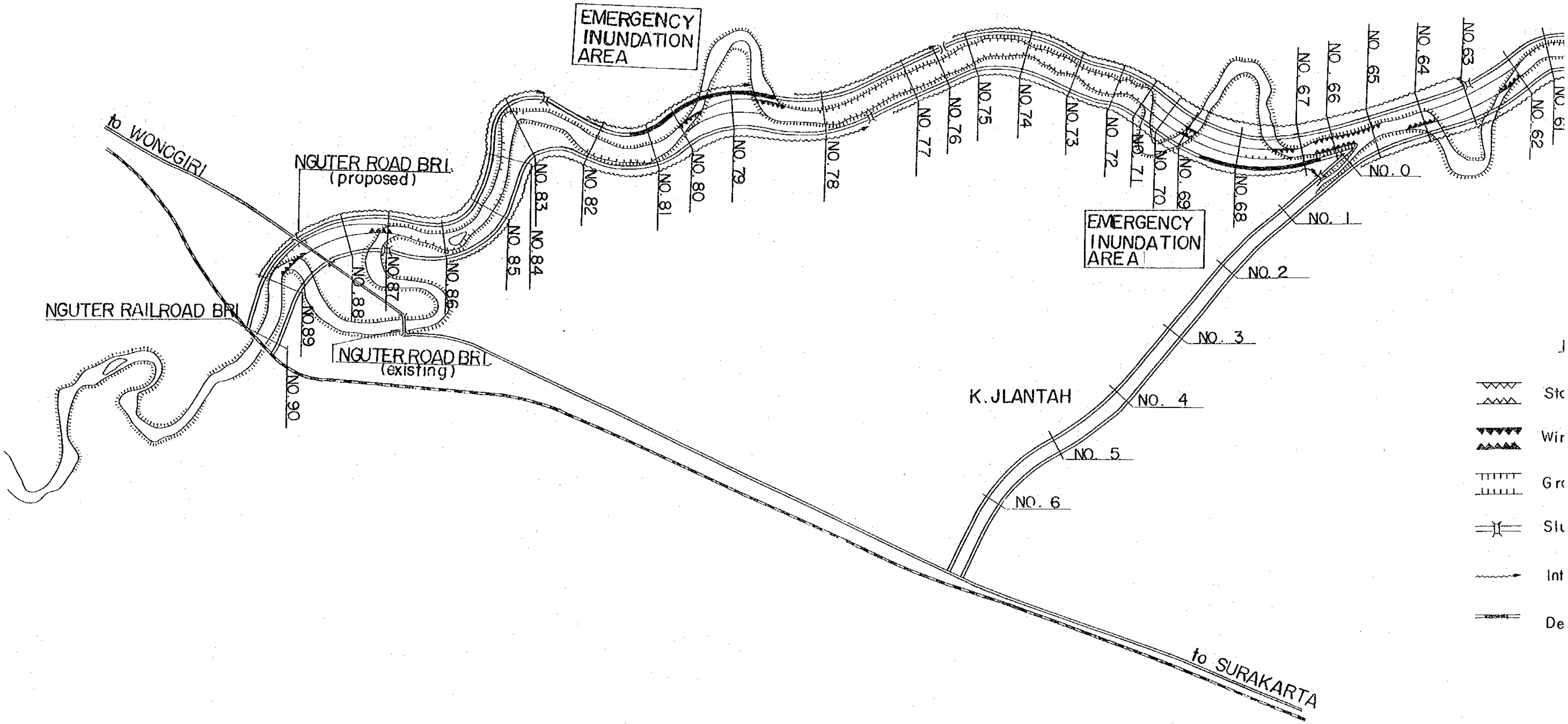
VER COURSE — I



K.WINGKO
SURAKARTA CITY
K.PEPE
JURUG ROAD BRI.
DBRI.
MOJO RAIL ROAD BRI.
JURUG RAIL ROAD BRI
NO.4
NO.3
NO.2
NO.1
NO.0
NO.1
NO.2
NO.3
NO.4
NO.5
).6
).7
NO.27
NO.26
NO.25
NO.24
NO.23
NO.22
NO.21
NO.20
NO.19
NO.18
NO.17
NO.16
NO.15
NO.14
NO.13
NO.12
NO.11
NO.10
NO.9
NO.8
NO.7
NO.6
NO.5
NO.4
NO.3
NO.2
NO.1
K.SAMIN
REMARKS
Stone piching
Wire basket
Groin
Sluice way
Intersepting drainage channel
SCALE
0 500 1000 (M)

Fig. 3.2.6 (b)

ALIGNMENT OF PROPOPSED RIVER COURSE-2



IENT OF PROPOPSED RIVER COURSE-2

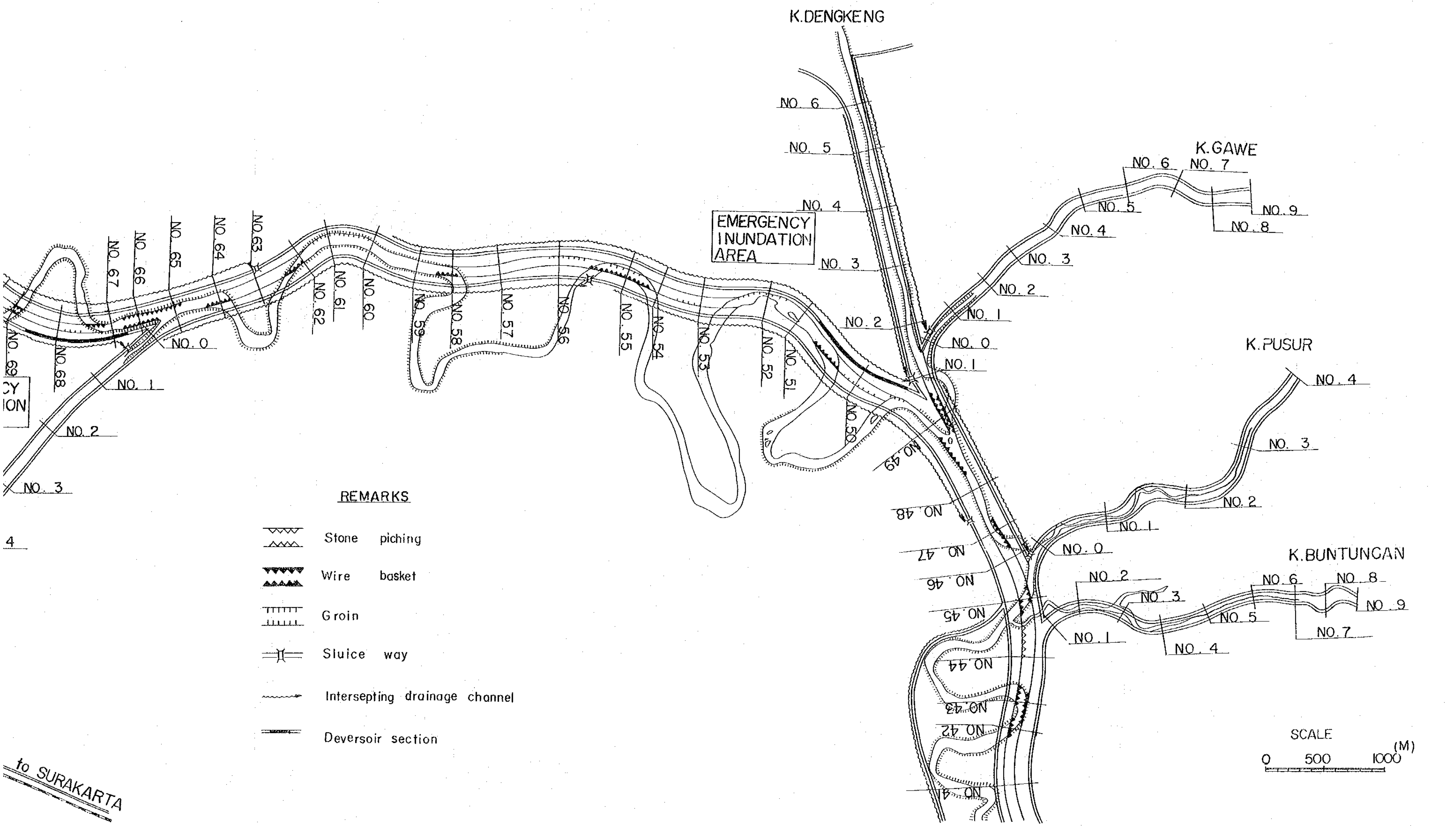


Fig. 3.2.7 PROFILE of PROPOSED RIVER COURSE

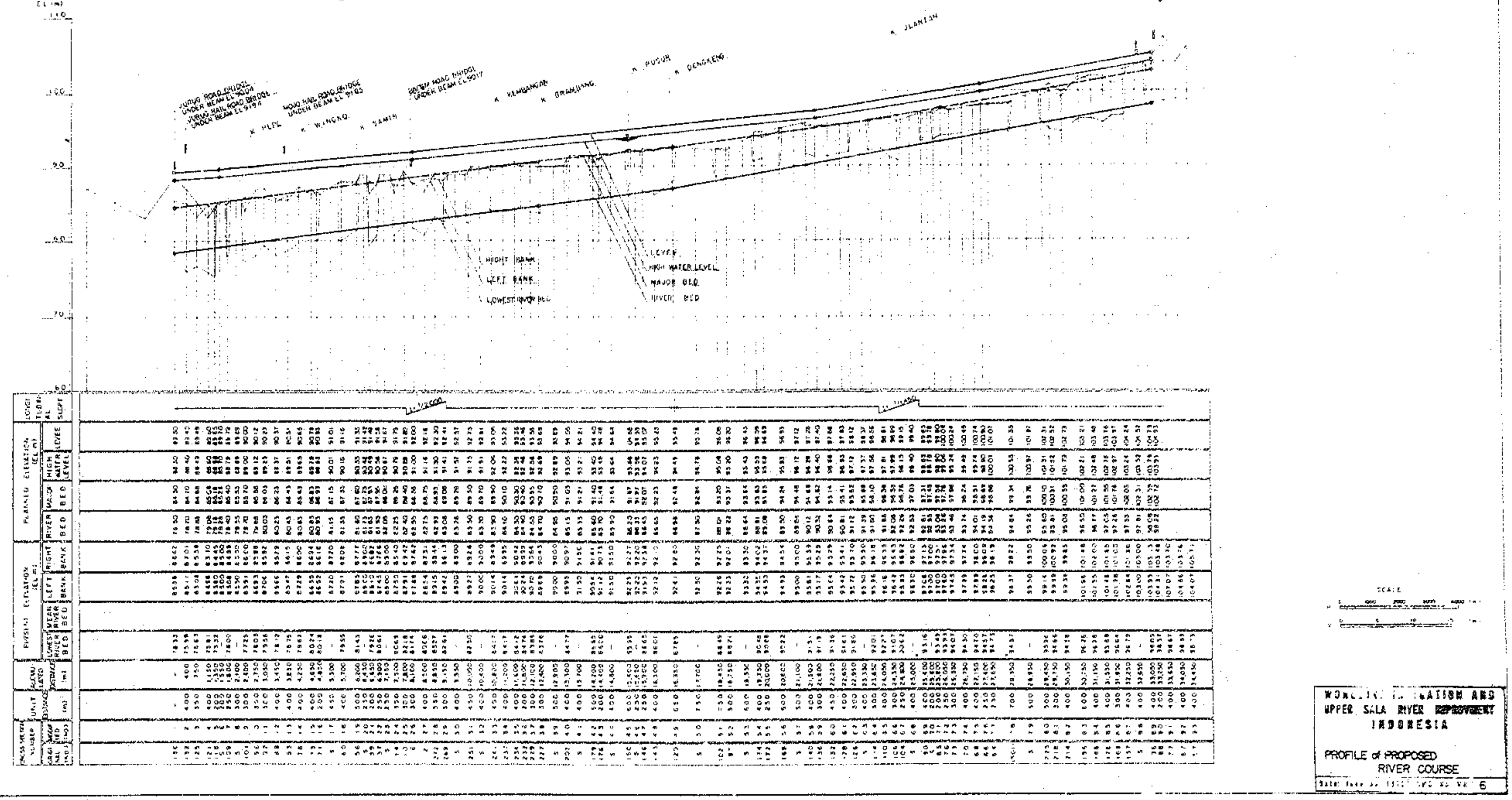


Fig-3.28 Relationship between Low water Channel width and Discharge

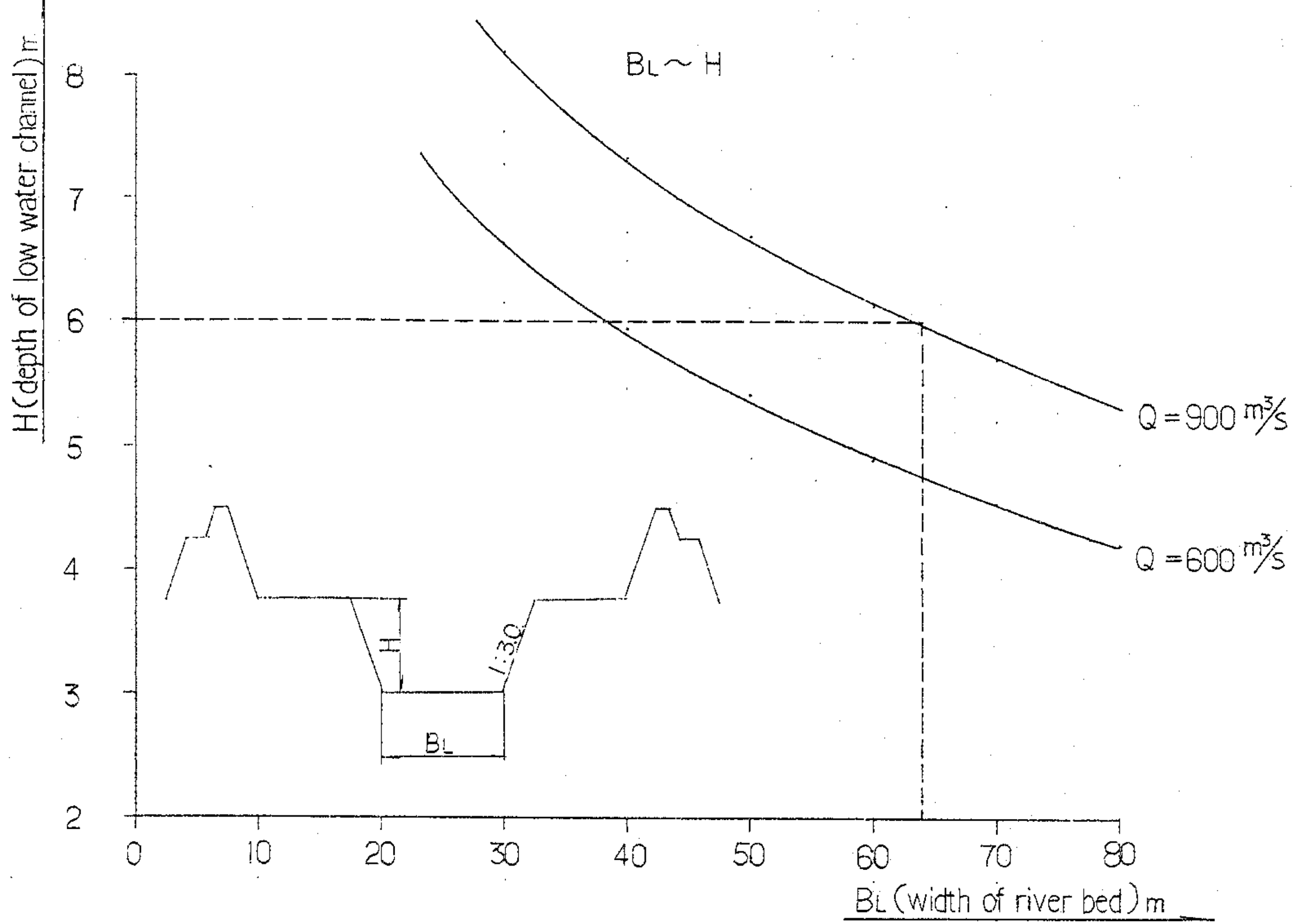


Fig-329 Relationship between High water Channel width and Discharge

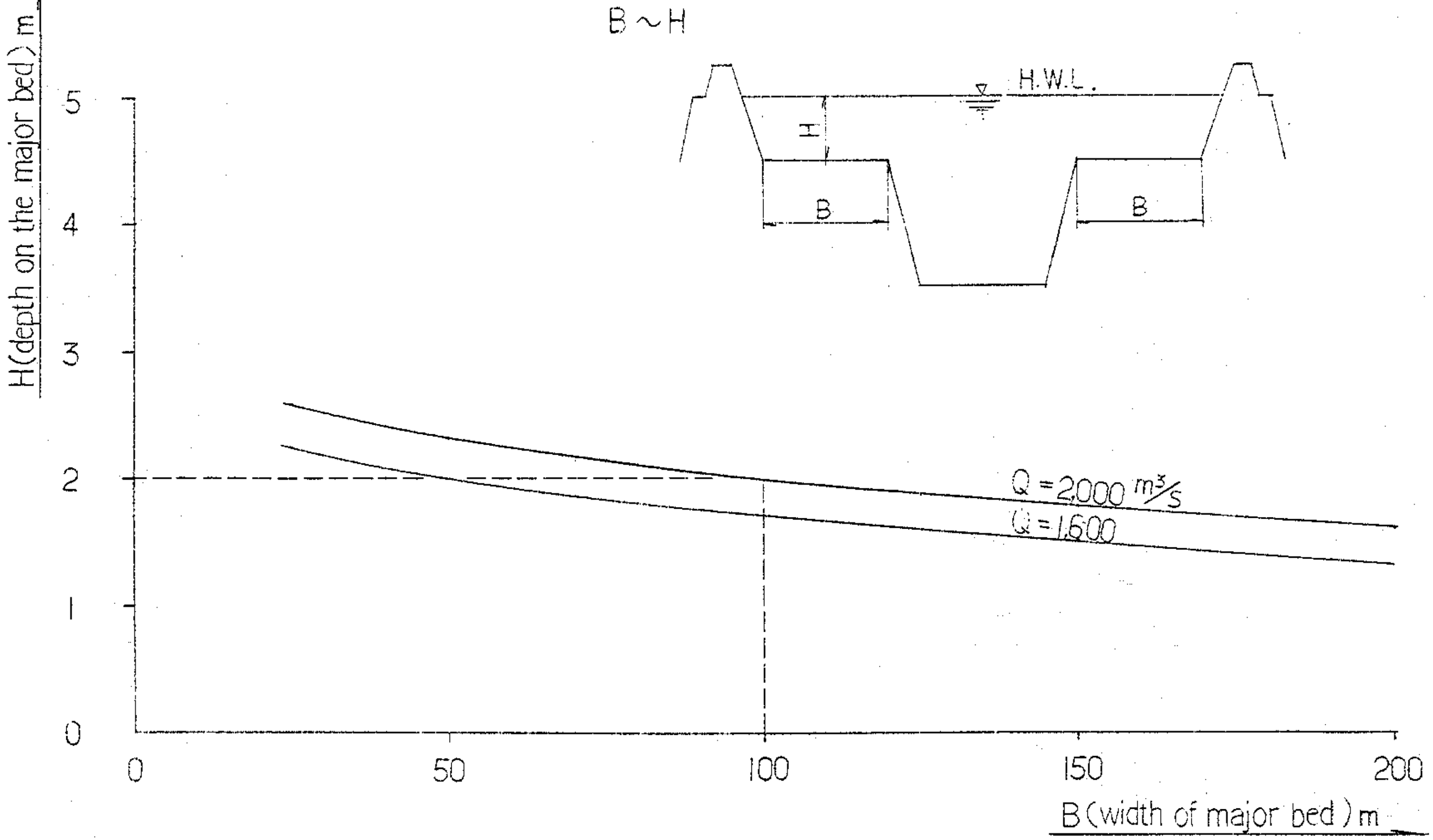
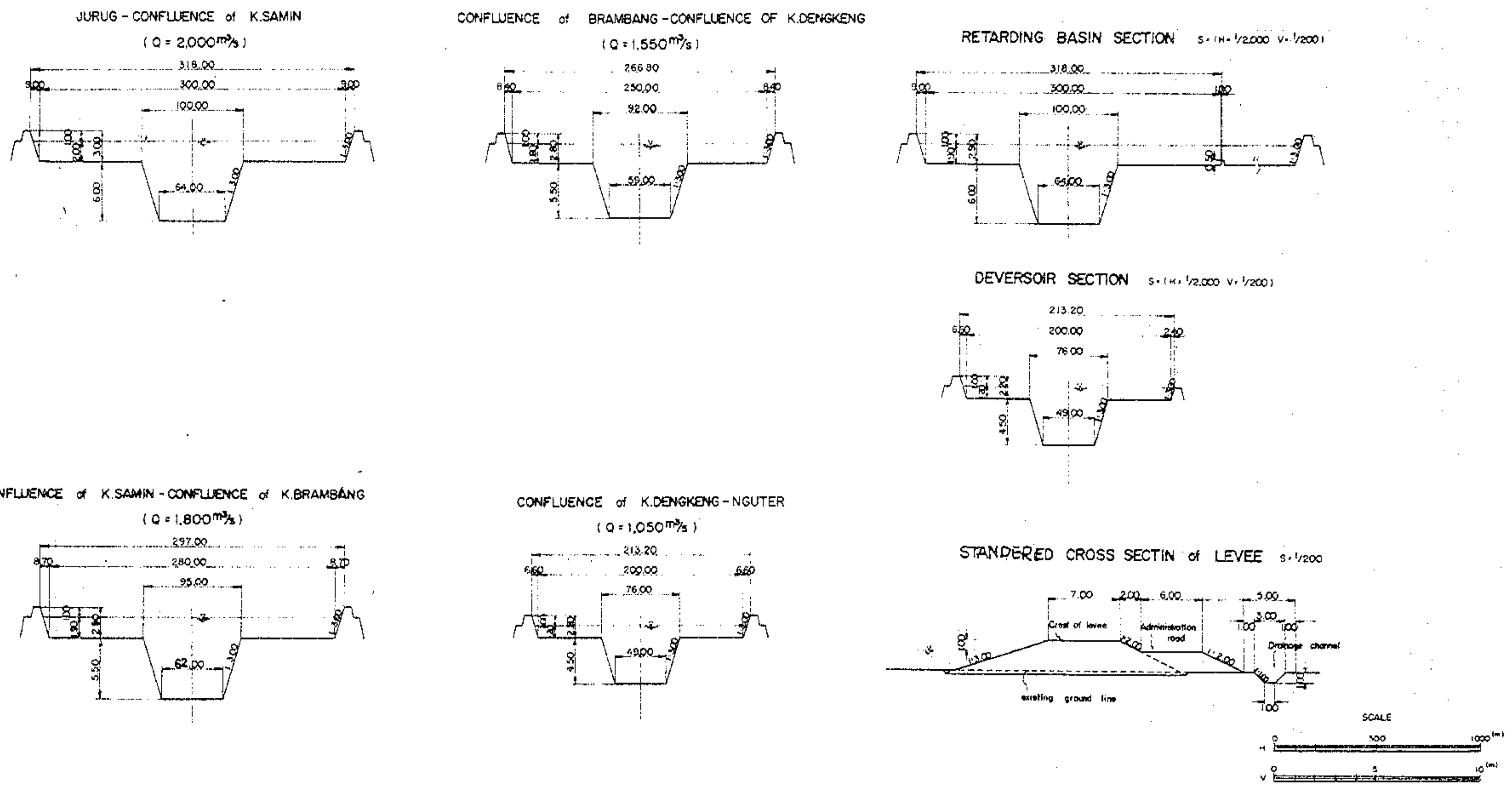


Fig-3.2.10 PROPOSED STANDARD CROSS SECTIONS of SALA RIVER (2000 cm³/s - case)

STANDARD CROSS SECTIONS of BENGAWAN SALA S = (H = 1/2,000 V = 1/200)

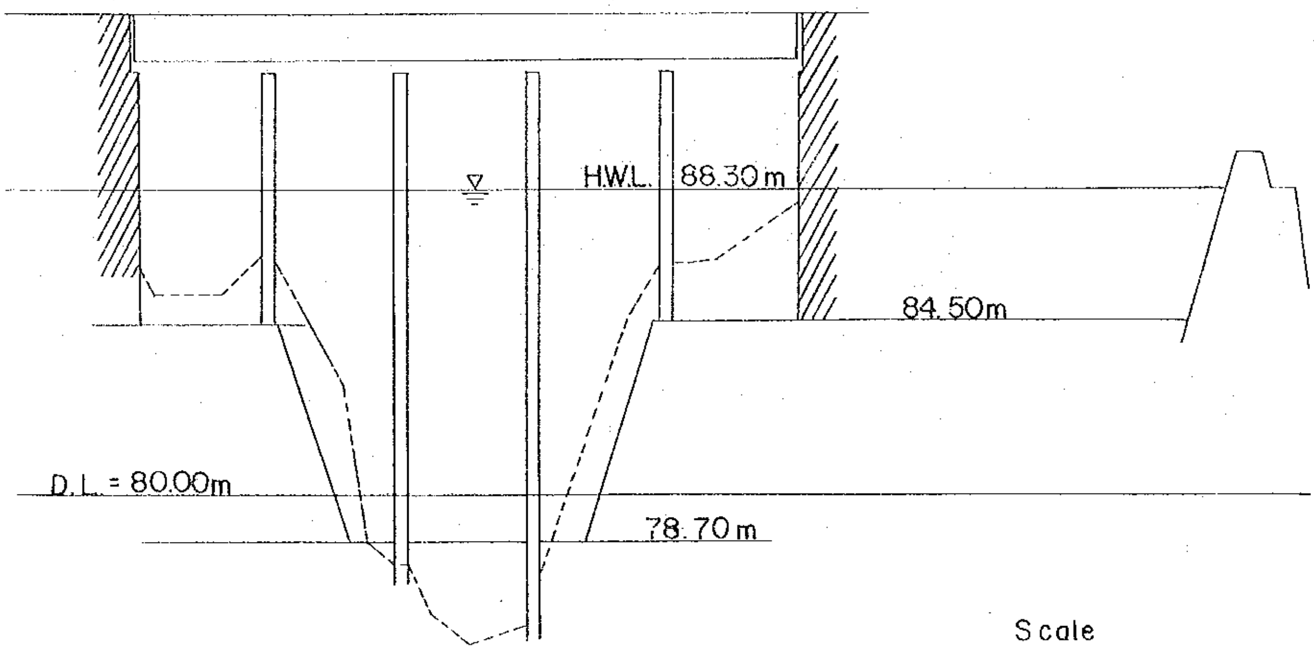


WONOGIRI IRRIGATION AND UPPER SALA RIVER IMPROVMENT
INDONESIA
PROPORSED
STANDARD CROSS SECTION
of BENGAWAN SALA

Date: June 20, 1976 | DWG. No. WE. 16

Fig. 3.2.10 Proposed Standard Cross Section in Bengawan Sala
(2,000 m^3/s - Case)
(Proposed Cross Section at Bridge Site (a))

Jurug rail road bridge



Scale
H : 1/2000
V : 1/ 200

Jurug road bridge

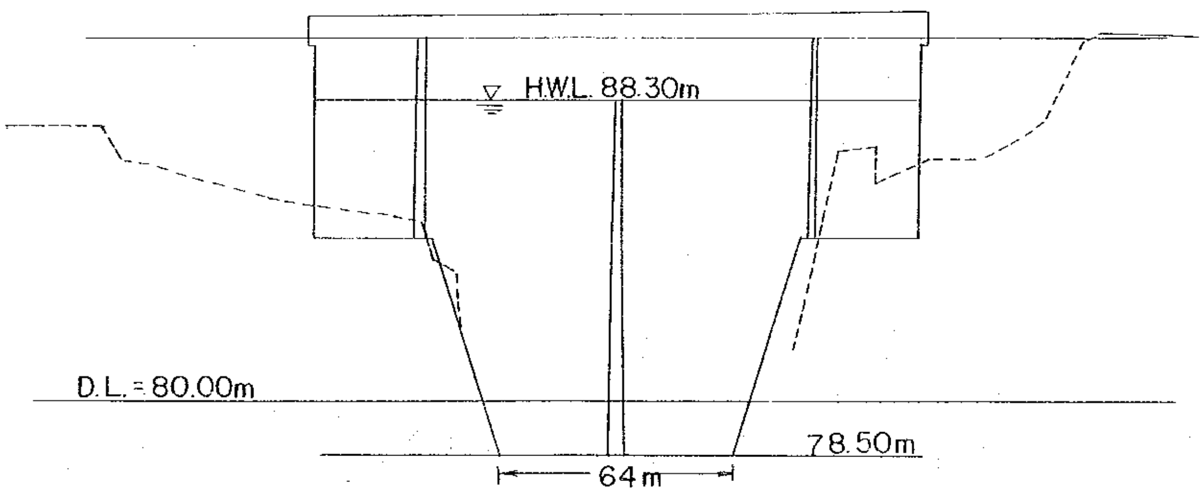
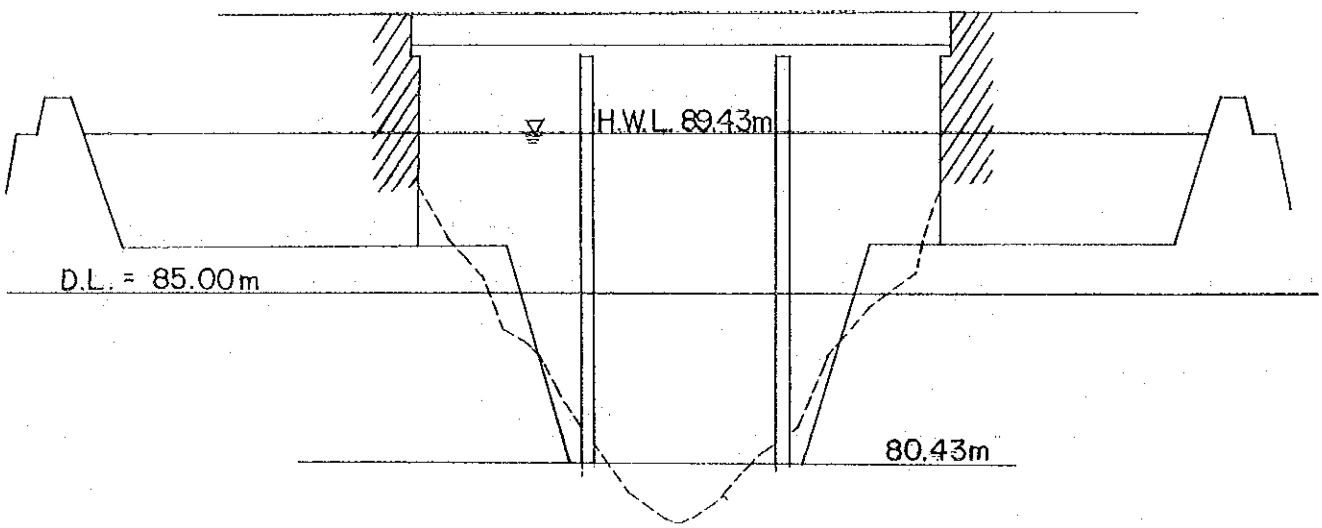


Fig. 3.2.10 Proposed Standard Cross Section in Bengawan Sala
(2,000 m^3/s - Case)
(Proposed Cross Section at Bridge Site (b))

Mojo rail road bridge



Scale
H : 1/2000
V : 1/ 200

Bacem road bridge

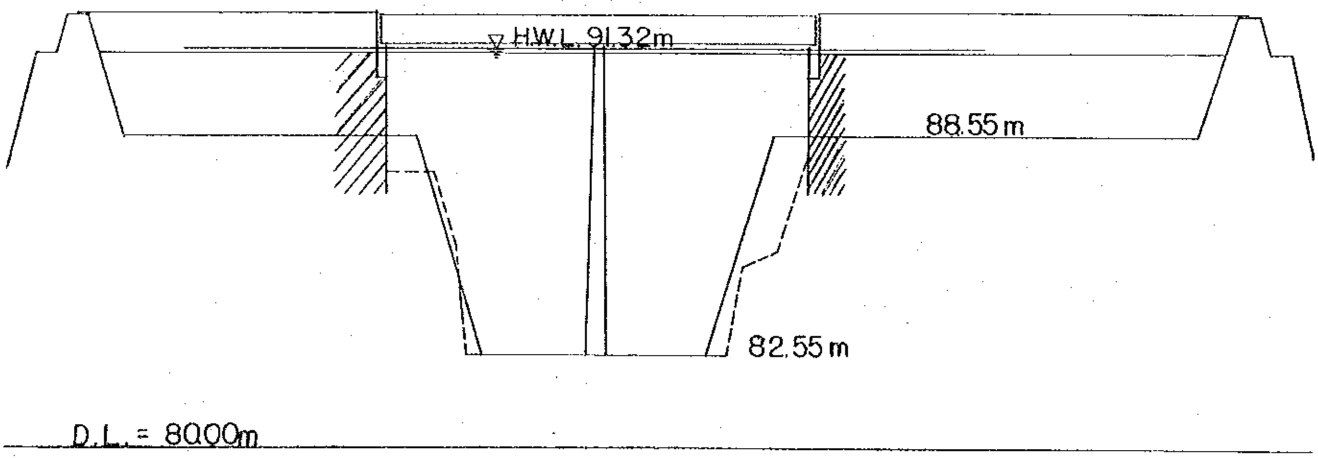


Fig. 3.2.10 Proposed Standard Cross Section in Bengawan Sala
(2,000 m^3/s - Case)
(Proposed Cross Section at Bridge Site (c))

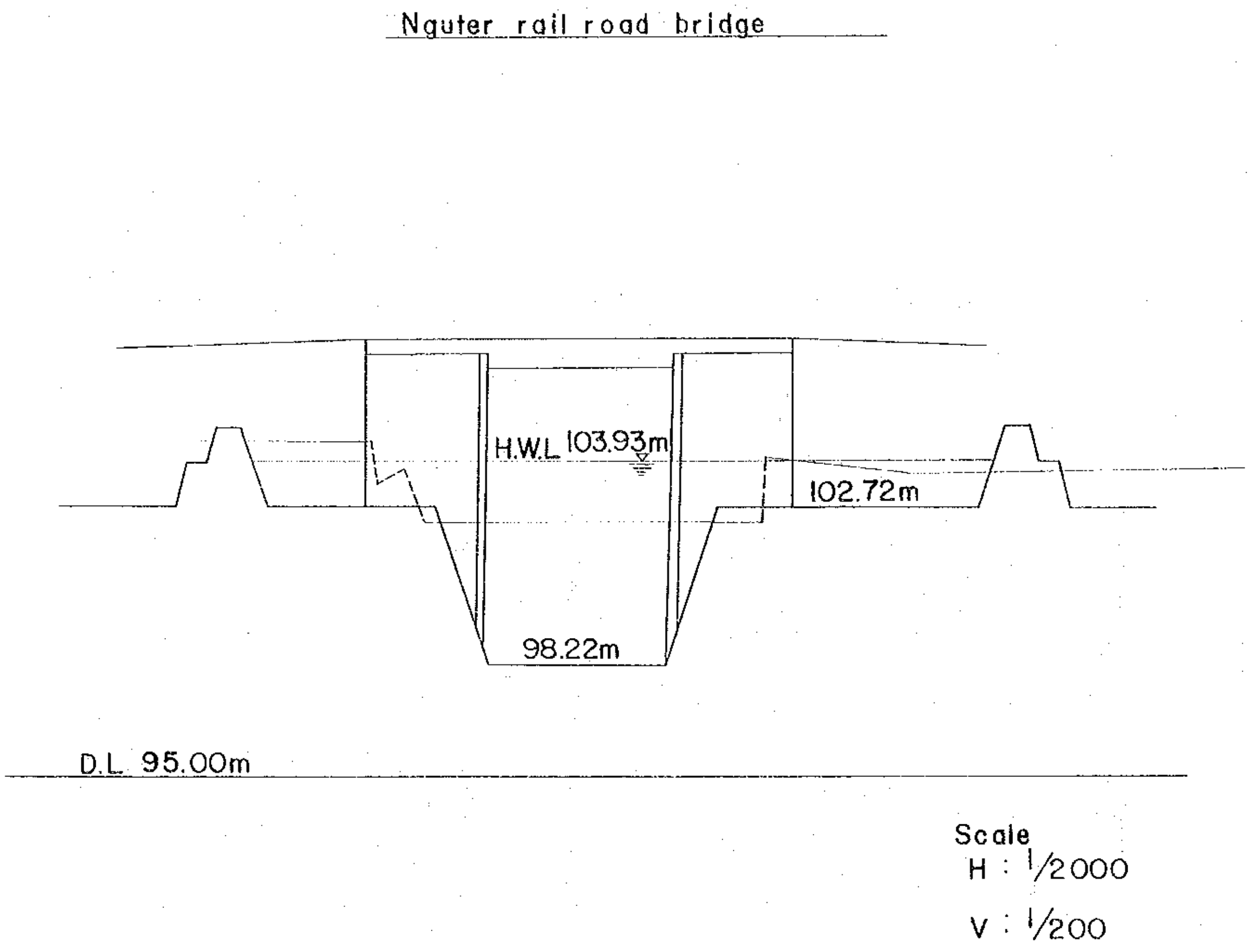
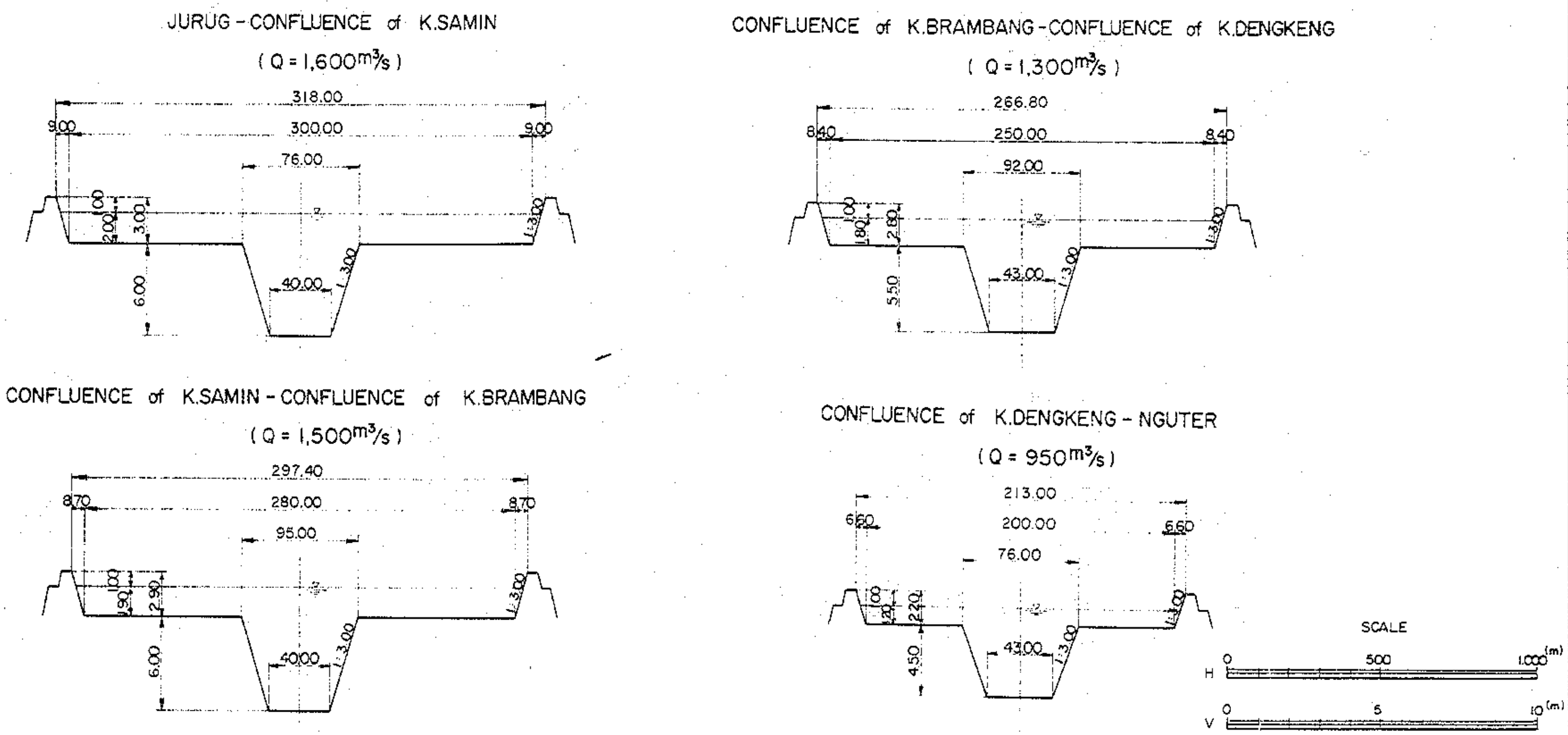


Fig-3.2.11 PROPOSED STANDARD CROSS SECTION of SALA RIVER

S=(H= 1/2,000 .V= 1/200)

(1600 m^3/s - case)



WONOGIRI IRRIGATION AND
UPPER SALA RIVER IMPROVMENT
INDONESIA
PROPOSED
STANDARD CROSS SECTION
of BENGAWAN SALA

Date: J... J. WR. 62

Fig. 3.2.12 Location of Flood control facility

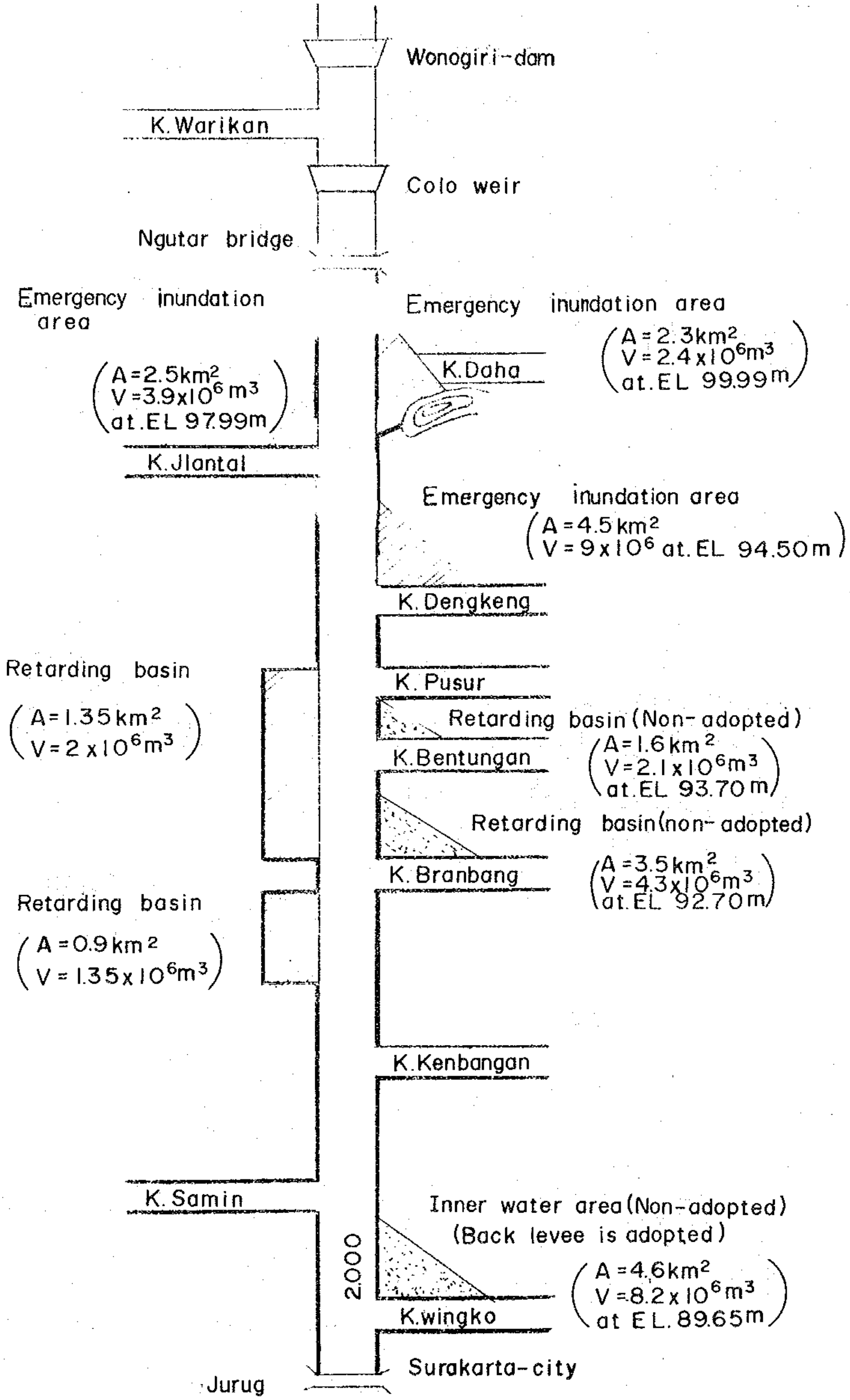
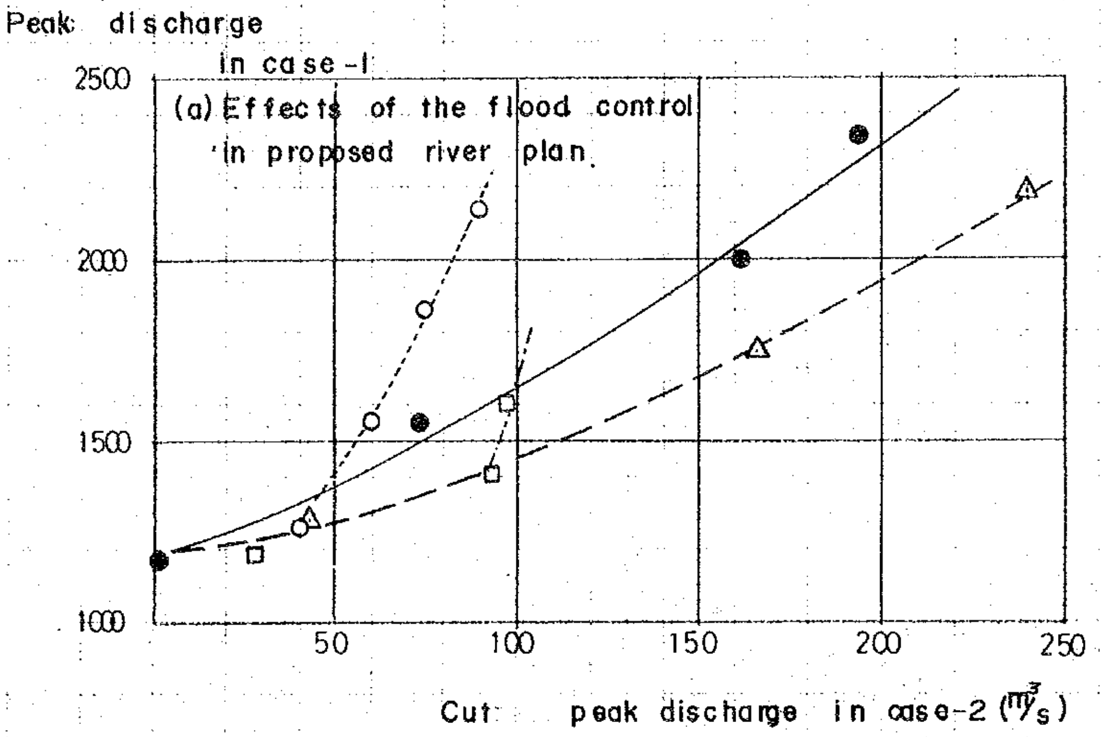


Fig. 3.2.13 Effects of Retarding basin and Open-levee.



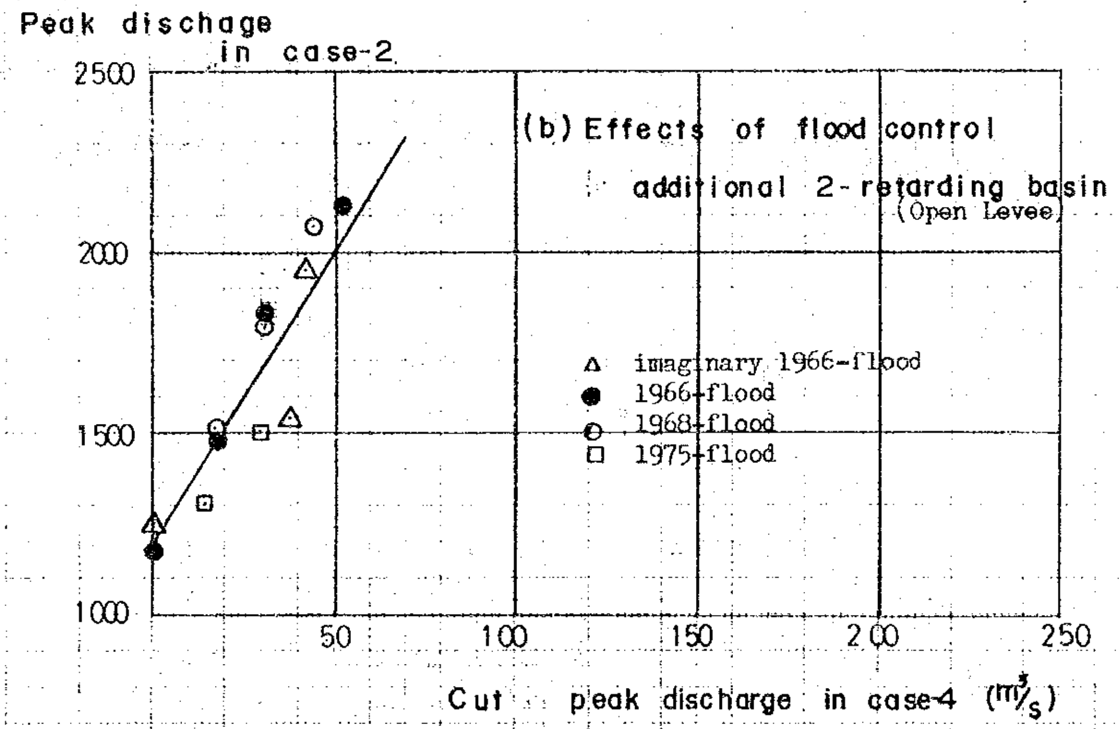


Fig 3.2.14 Probable Discharge in Proposed Channel

(Jurug site)

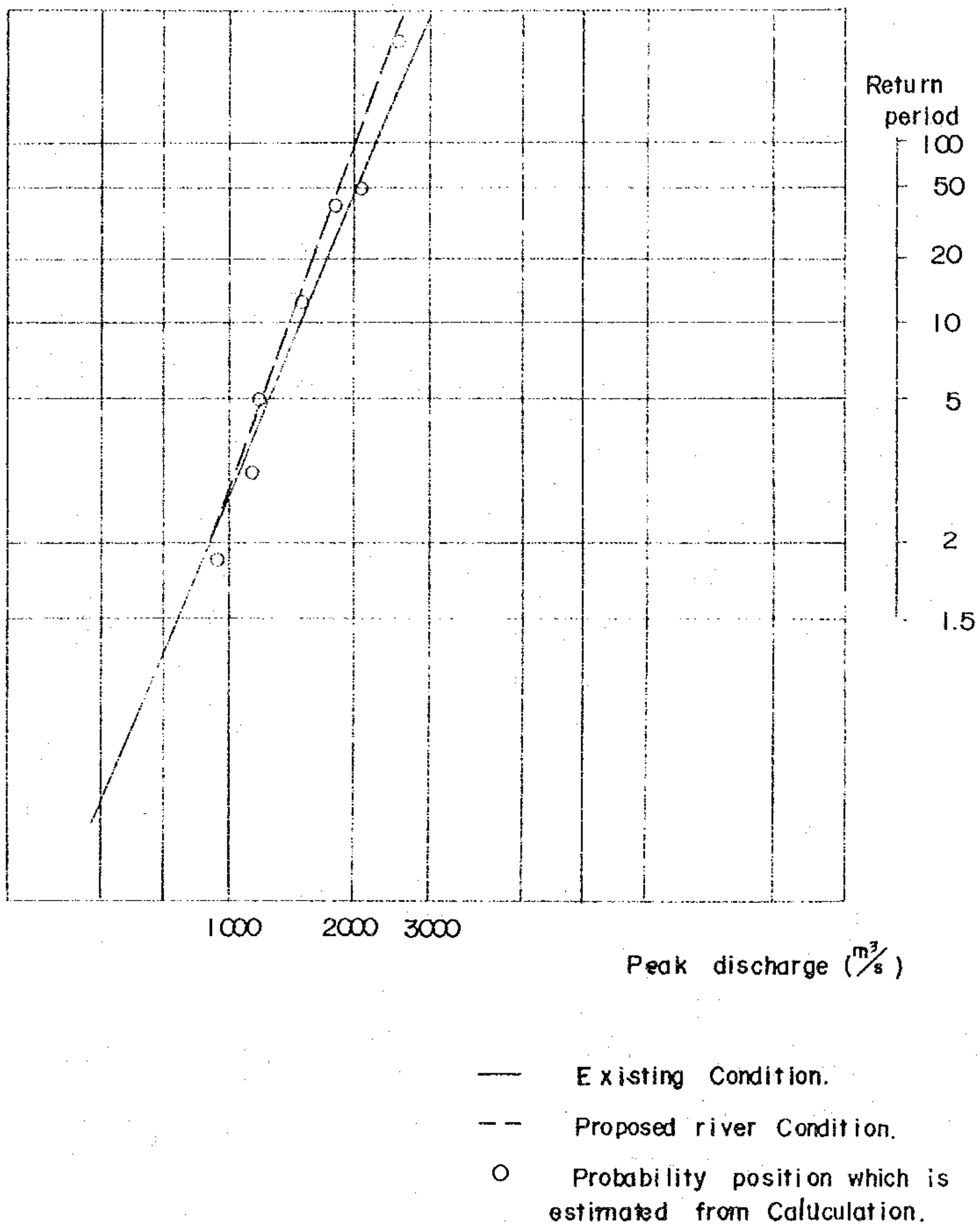
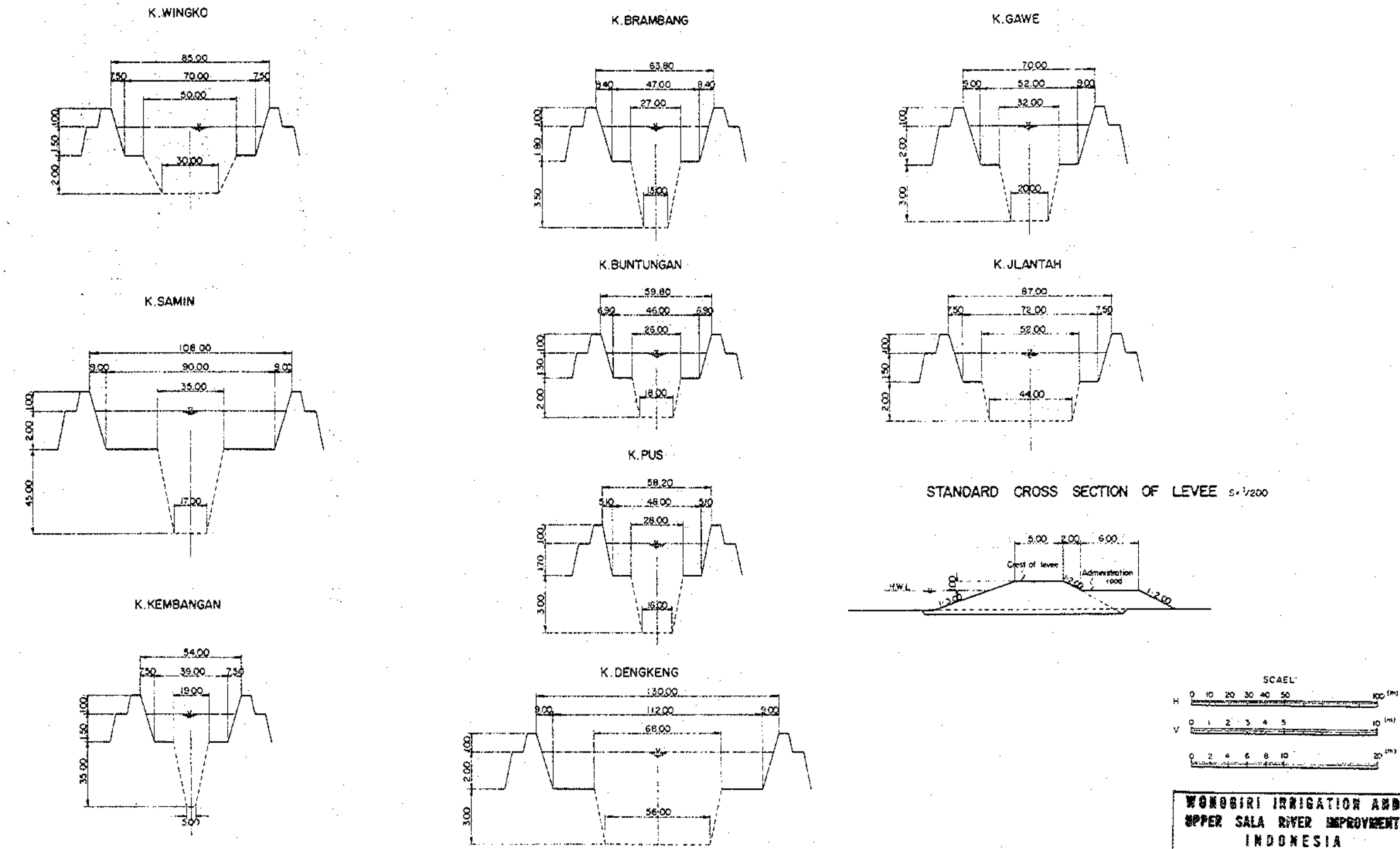


Fig - 3.2.15 STANDARD CROSS SECTIONS of TRIBUTARIES FUTURE S: (H: 1/1,000 V: 1/100)

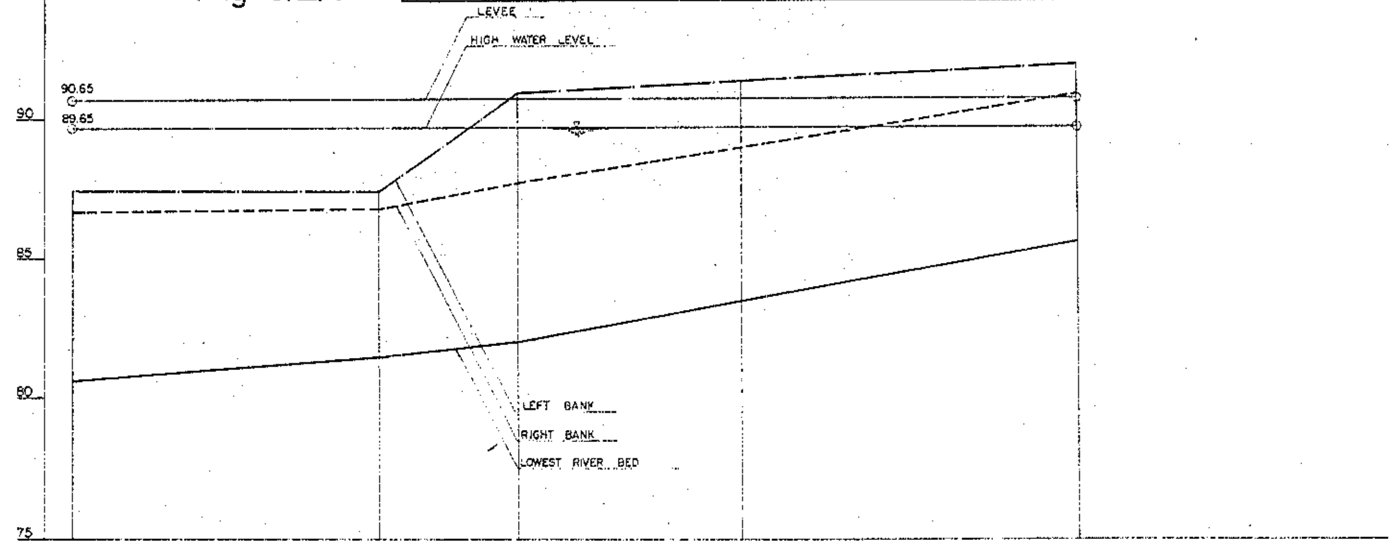


WONOGIRI IRRIGATION AND UPPER SALA RIVER IMPROVEMENT INDONESIA

STANDARD CROSS SECTION of TRIBUTARIES FUTURE

Date: June 28, 1975 | DWG. No. WR 17

Fig-3.2.16 LONGITUDINAL SECTION of K.WINGKO



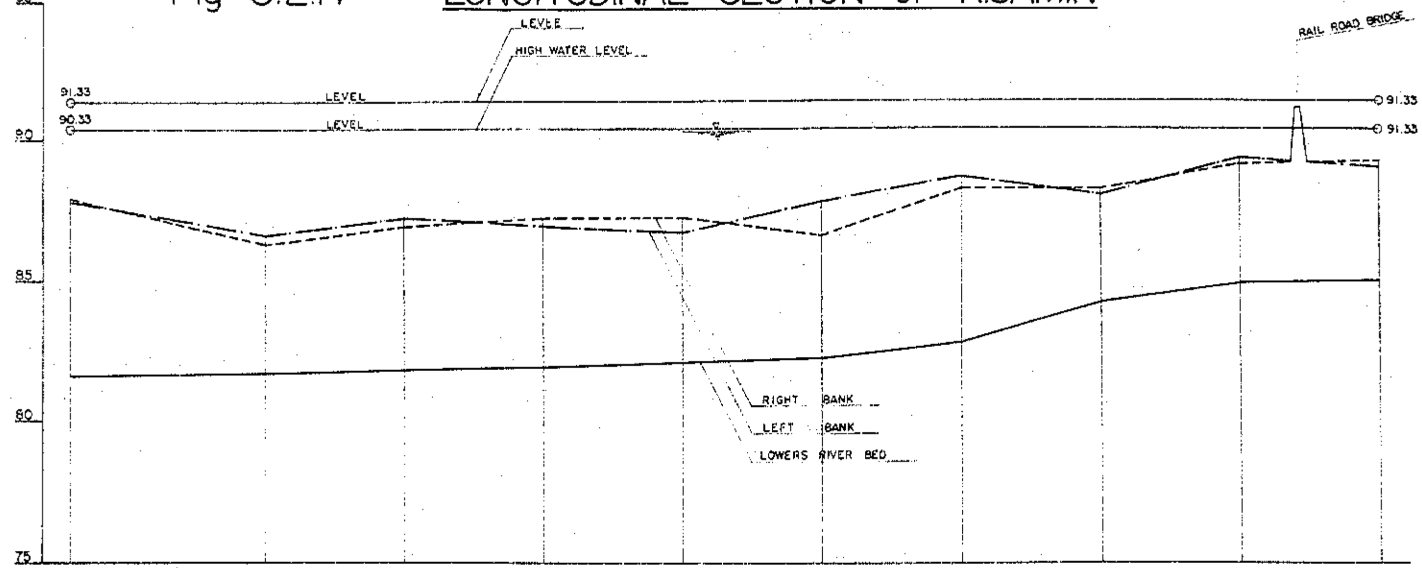
PLANNED ELEVATION (EL. m)	LEVEE	90.65	90.65	90.65	90.65	90.65
	HIGH WATER LEVEL	89.65	89.65	89.65	89.65	89.65
PRESENT ELEVATION (EL. m)	RIGHT BANK	86.70	86.80	87.70	88.95	90.80
	LEFT BANK	87.44	87.40	90.84	91.25	91.82
	LOWEST RIVER BED	80.63	81.45	82.00	83.43	85.60
ACCUMULATED DISTANCE (m)		—	1,100	1,600	2,400	3,600
UNIT DISTANCE (m)		—	1,100	500	800	1,200
CROSS SECTION NO.		0	1	2	3	4

IRRIGATION AND UPPER SOLA RIVER IMPROVMENT INDONESIA

LONGITUDINAL SECTION of K.WINGKO

Date: June 30, 1975 | DWG. No. WR. 7

Fig-3.2.17 LONGITUDINAL SECTION of K.SAMIN



			0	1	2	3	4	5	6	7	8	9
PLANNED ELEVATION (EL.m)	LEVEE		91.33	91.33	91.33	91.33	91.33	91.33	91.33	91.33	91.33	91.33
	HIGH WATER LEVEL		90.33	90.33	90.33	90.33	90.33	90.33	90.33	90.33	90.33	90.33
PRESENT ELEVATION (EL.m)	RIGHT BANK		87.9	86.3	86.9	87.2	87.2	86.6	88.2	88.2	89.1	89.2
	LEFT BANK		87.8	86.6	87.2	86.9	86.7	87.8	88.6	88.0	89.3	89.0
	LOWERS RIVER BED		81.6	81.7	81.8	81.9	82.1	82.2	82.8	84.2	84.9	85.0
ACCUMULATED DISTANCE (m)			–	700	1,200	1,700	2,200	2,700	3,200	3,700	4,200	4,700
UNIT DISTANCE (m)				700	500	500	500	500	500	500	500	500
CROSS SECTION NO.			0	1	2	3	4	5	6	7	8	9

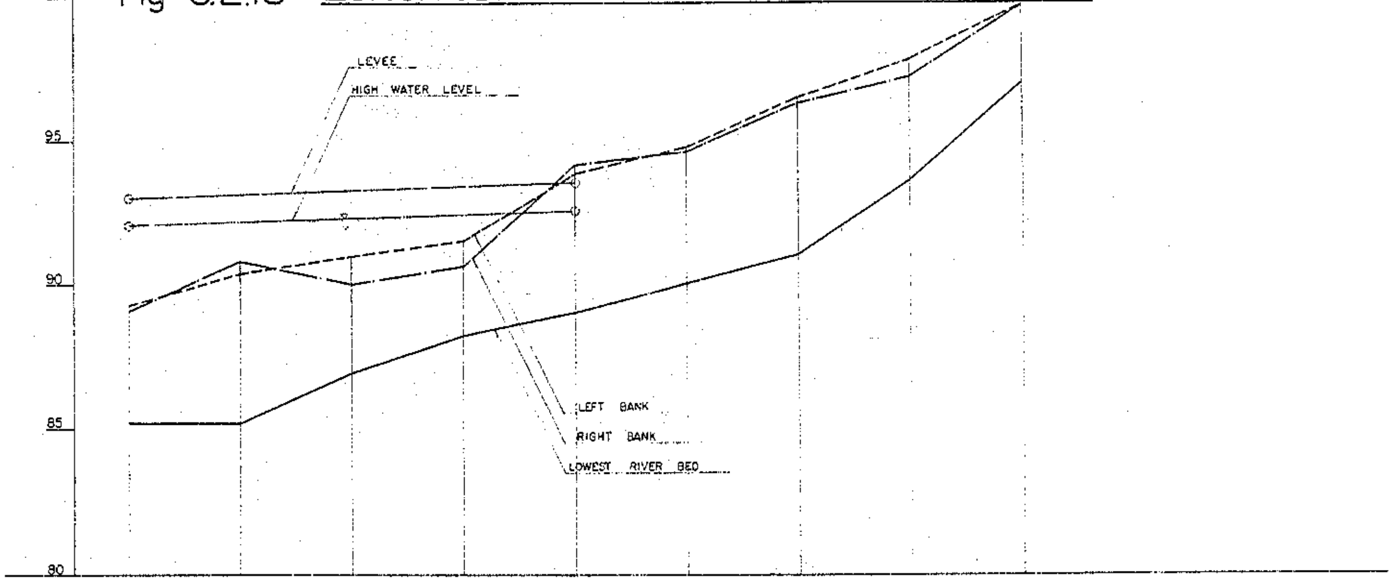
SCALE
H 0 500 1,000 (m)
V 0 5 10 (m)

WONOGIRI IRRIGATION AND UPPER SALA RIVER IMPROVMENT
INDONESIA

LONGITUDINAL SECTION of K.SAMIN

Date: June 30, 1976 | DWG. No. WR 8

Fig-3.2.18 LONGITUDINAL SECTION of K.KEMBANGAN



			0	1	2	3	4	5	6	7	8
PLANNED ELEVATION (EL m)	LEVEE		93.06	93.17	93.27	93.38	93.48	93.64	93.75	93.87	93.98
	HIGH WATER LEVEL		92.06	92.17	92.27	92.38	92.48	92.64	92.75	92.87	92.98
PRESENT ELEVATION (EL m)	RIGHT BANK		89.1	90.8	90.0	90.6	94.1	94.6	96.2	97.1	99.6
	LEFT BANK		89.3	90.4	91.0	91.5	93.8	94.7	96.4	97.7	99.6
	LOWEST RIVER BED		85.2	85.2	86.9	88.2	89.0	90.0	91.0	93.6	96.9
ACCUMULATED DISTANCE (m)			—	400	800	1,200	1,600	2,000	2,400	2,800	3,200
UNIT DISTANCE (m)			—	400	400	400	400	400	400	400	400
CROSS SECTION NO			0	1	2	3	4	5	6	7	8

SCALE

H 0 500 1000 (m)

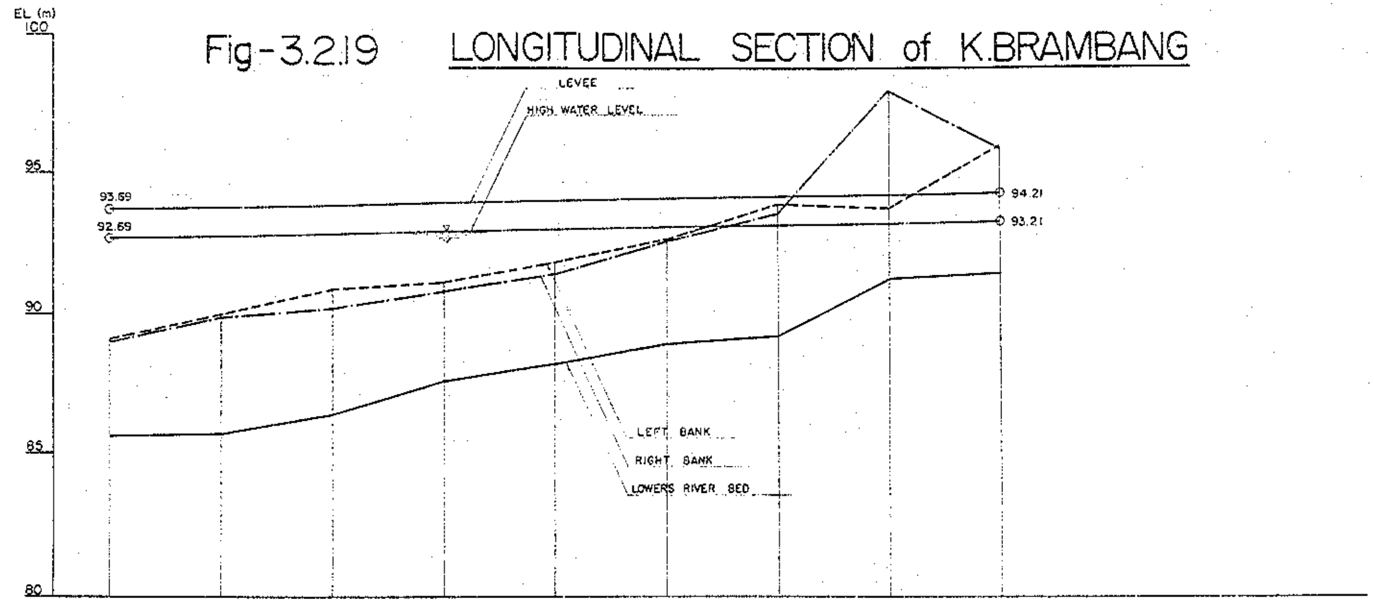
V 0 5 10 (m)

WONOGIRI IRRIGATION AND UPPER SALA RIVER IMPROVMENT INDONESIA

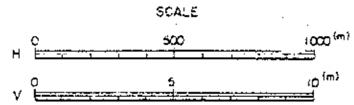
LONGITUDINAL SECTION of K.KEMBANGAN

Date: June 30, 1976 | No. WR 9

Fig-3.2.19 LONGITUDINAL SECTION of K.BRAMBANG



PLANNED ELEVATION (EL m)	LEVEE	93.69	93.76	93.82	93.89	93.95	94.02	94.08	94.15	94.21
	HIGH WATER LEVEL	92.69	92.76	92.82	92.89	92.95	93.02	93.08	93.15	93.21
PRESENT ELEVATION (EL m)	RIGHT BANK	88.5	89.9	90.2	90.8	91.4	92.5	93.5	97.9	95.8
	LEFT BANK	88.6	90.0	90.9	91.1	91.8	92.6	93.8	93.7	95.9
	LOWERS RIVER BED	85.6	85.7	86.4	87.6	88.2	88.9	89.2	91.2	91.4
ACCUMULATED DISTANCE (m)		—	400	800	1,200	1,600	2,000	2,400	2,800	3,200
UNIT DISTANCE (m)		—	400	400	400	400	400	400	400	400
CROSS SECTION NO.		0	1	2	3	4	5	6	7	8

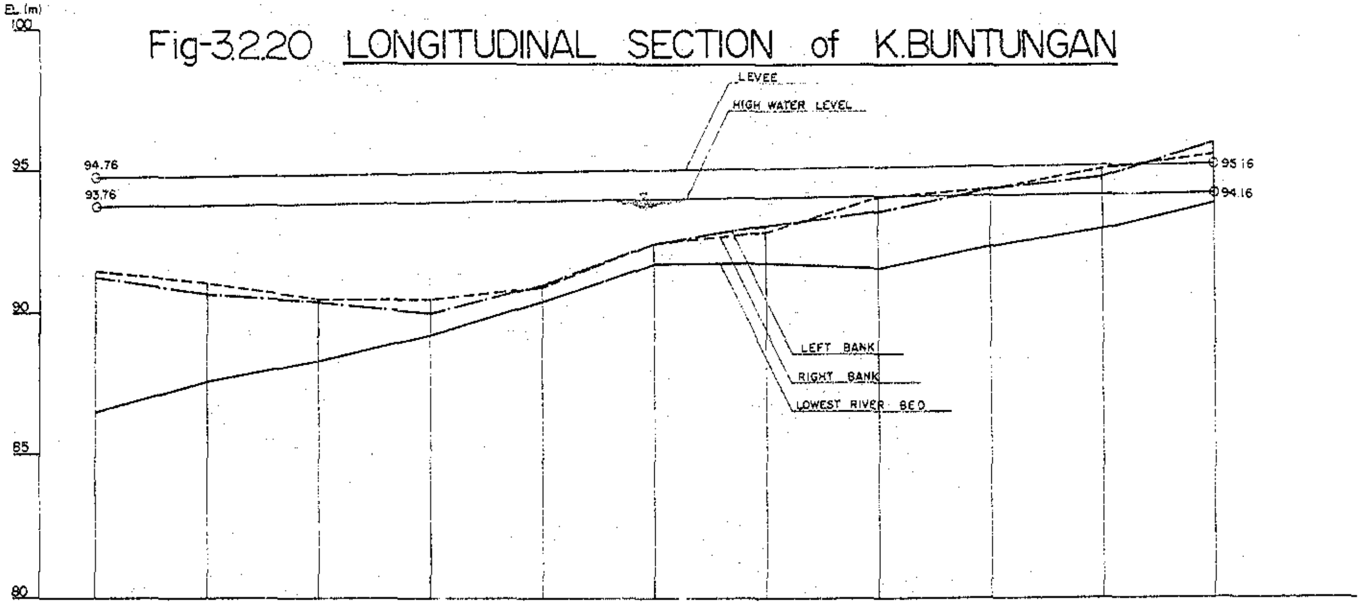


W.JNC ... .SATION AND
UPPER SALA RIVER IMPROVMENT
INDONESIA

LONGITUDINAL SECTION
of K.BRAMBANG

Date: June 3?, 1976 | ... | No. 10

Fig-3.2.20 LONGITUDINAL SECTION of K.BUNTUNGAN



			0	1	2	3	4	5	6	7	8	9	10
PLANNED ELEVATION (EL m)	LEVEE		94.76	94.80	94.84	94.88	94.92	94.96	95.00	95.04	95.08	95.12	95.16
	HIGH WATER LEVEL		93.76	93.80	93.84	93.88	93.92	93.96	94.00	94.04	94.08	94.12	94.16
PRESENT ELEVATION (EL m)	RIGHT BANK		91.5	91.1	90.5	90.5	90.9	92.4	92.6	94.0	94.3	95.0	95.5
	LEFT BANK		91.3	90.7	90.4	90.0	91.0	92.4	93.0	93.5	94.3	94.7	95.9
	LOWEST RIVER BED		86.5	87.6	88.3	89.2	90.4	91.7	91.7	91.5	92.3	92.9	93.9
ACCUMLATED DISTANCE (m)			—	400	800	1,200	1,600	2,000	2,400	2,800	3,200	3,600	4,000
UNIT DISTANCE (m)			—	400	400	400	400	400	400	400	400	400	400
CROSS SECTION NO.			0	1	2	3	4	5	6	7	8	9	10

SCALE

H 0 500 1,000 (m)

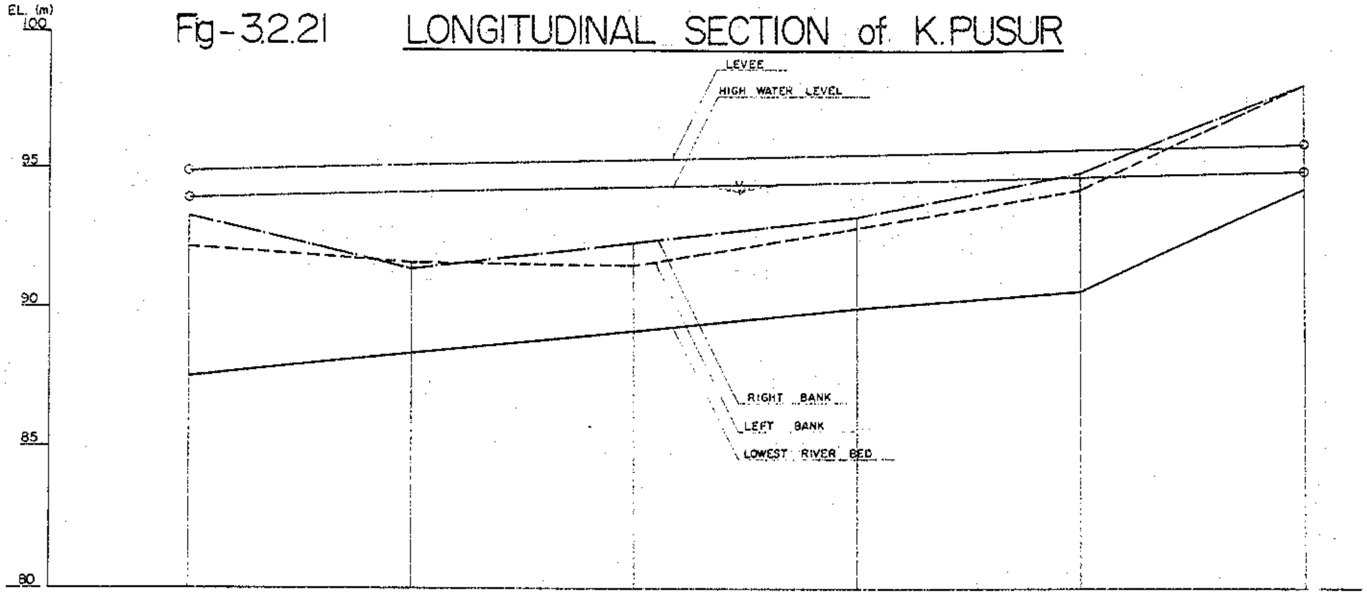
V 0 5 10 (m)

WONOGIRI IRRIGATION AND UPPER SALA RIVER IMPROVEMENT INDONESIA

LONGITUDINAL SECTION of K.BUNTUNGAN

Date: June 18, 1976 | DWG. No. 98.

Fig-3.2.21 LONGITUDINAL SECTION of K.PUSUR



PLANNED ELEVATION (EL.m)	LEVEE	94.88	94.95	95.14	95.34	95.53	95.72
	HIGH WATER LEVEL	93.88	93.95	94.14	94.34	94.53	94.72
PRESENT ELEVATION (EL.m)	RIGHT BANK	92.1	31.5	91.4	92.7	94.1	97.9
	LEFT BANK	93.2	91.3	92.2	93.1	94.7	97.9
	LOWEST RIVER BED	87.5	88.3	89.1	89.9	90.5	94.1
ACCUMULTED DISTANCE (m)		—	800	1,600	2,400	3,200	4,000
UNIT DISTANCE (m)		—	800	800	800	800	800
CROSS SECTION NO.		0	1	2	3	4	5

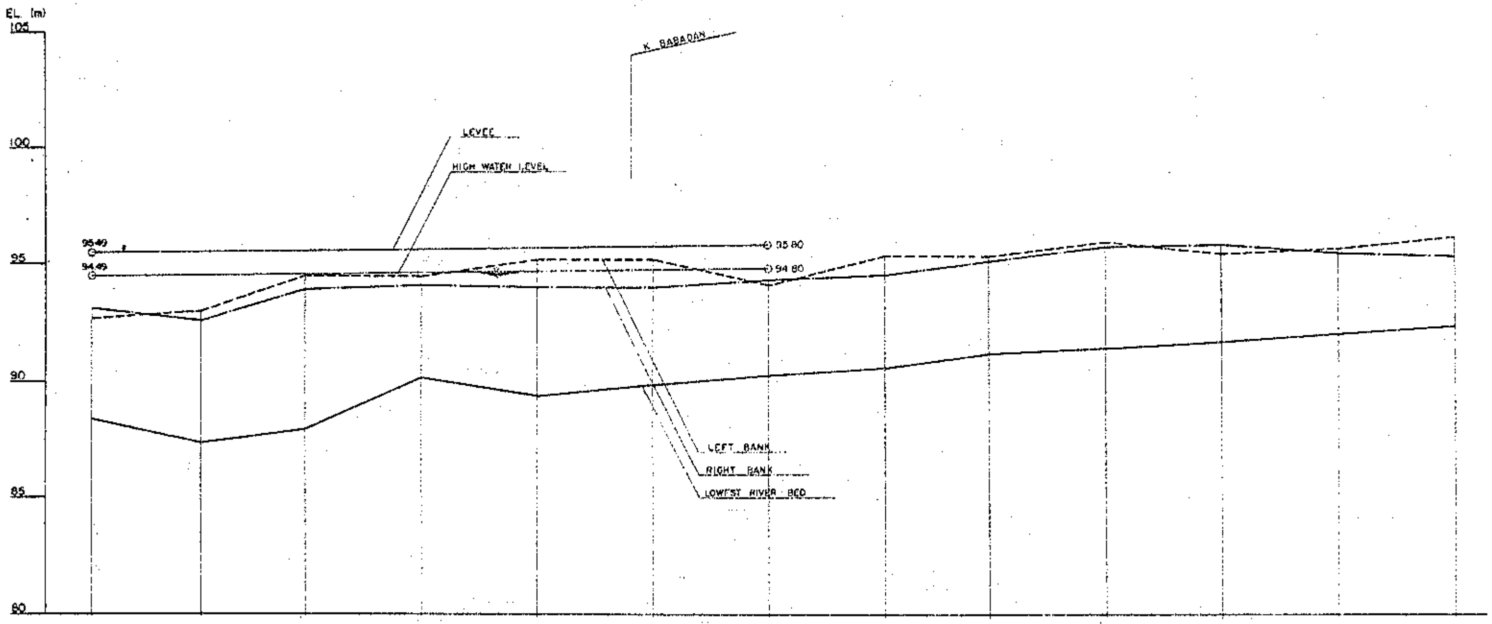
SCALE
H 0 500 1,000 (m)
V 0 5 10 (m)

WONOGIRI IRRIGATION AND UPPER SALA RIVER IMPROVMENT INDONESIA

LONGITUDINAL SECTION of K.PUSUR

Date: June ,, 1976 | DWG. No. WR 12

Fig - 3.2.22 LONGITUDINAL SECTION of K.DENGKENG



		0	1	2	3	4	5	6	7	8	9	10	11	12
PLANNED ELEVATION (EL. m)	LEVEE	95.49	95.54	95.55	95.64	95.69	95.74	95.80	-	-	-	-	-	-
	HIGH WATER LEVEL	94.49	94.54	94.59	94.64	94.69	94.74	94.80	-	-	-	-	-	-
PRESENT ELEVATION (EL. m)	RIGHT BANK	93.1	92.6	93.9	94.1	94.0	94.0	94.3	94.5	95.1	95.7	95.8	95.4	96.3
	LEFT BANK	92.7	93.0	94.5	94.5	95.2	95.2	94.1	95.3	95.3	95.9	95.4	95.6	95.1
	LOWEST RIVER BED	88.4	87.4	88.0	90.2	89.4	89.9	90.3	90.6	91.2	91.4	91.7	92.0	92.3
ACCUMULATED DISTANCE (m)		-	470	920	1,420	1,920	2,420	2,920	3,420	3,870	4,370	4,870	5,370	5,870
UNIT DISTANCE (m)		-	470	450	500	500	500	500	500	450	500	500	500	500
CROSS SECTION NO.		0	1	2	3	4	5	6	7	8	9	10	11	12

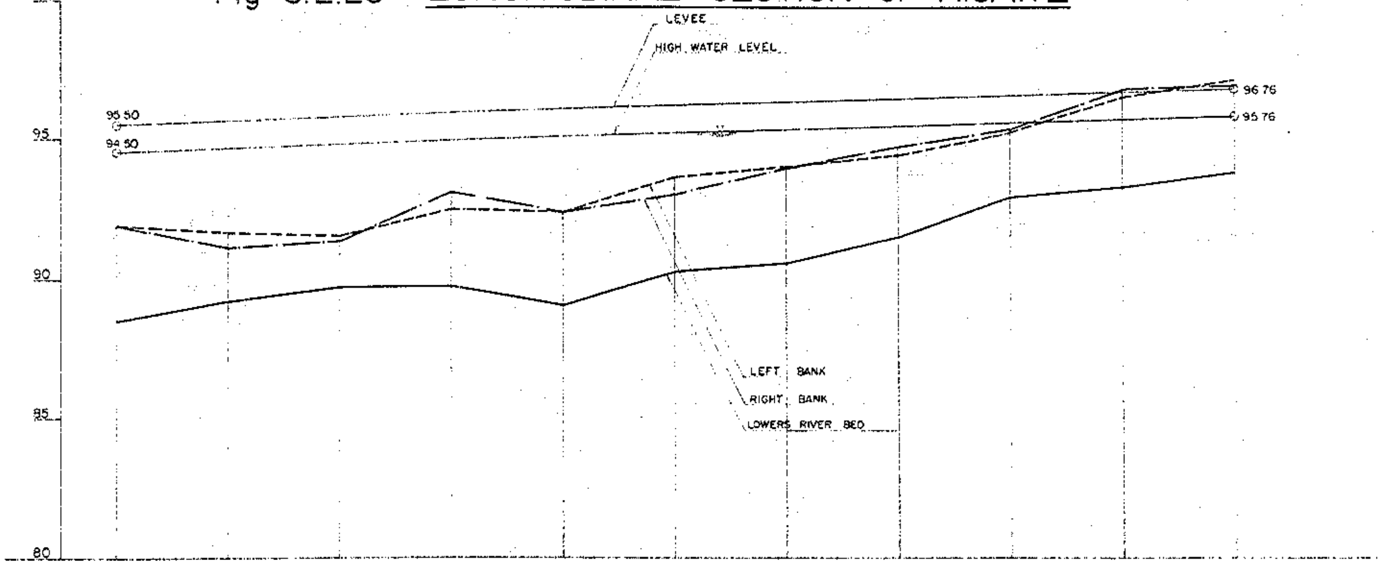
SCALE
H 0 500 1000 (m)
V 0 5 10 (m)

WONOGIRI IRRIGATION AND UPPER SALA RIVER IMPROVEMENT
INDONESIA

LONGITUDINAL SECTION of K.DENGKENG

Date: June 20, 1975 | DWG No. WK 13

Fig-3.2.23 LONGITUDINAL SECTION of K.GAWE



			0	1	2	3	4	5	6	7	8	9	10
PLANNED ELEVATION (EL m)	LEVEE		95.50	95.63	95.75	95.88	96.00	96.13	96.25	96.38	96.51	96.63	96.70
	HIGH WATER LEVEL		94.50	94.63	94.75	94.88	95.00	95.13	95.25	95.38	95.51	95.63	95.76
PRESENT ELEVATION (EL m)	RIGHT BANK		91.9	91.1	91.3	93.0	92.3	92.9	93.8	94.6	95.2	96.7	96.8
	LEFT BANK		91.9	91.6	91.5	92.4	92.3	93.5	93.9	94.3	95.1	96.4	97.0
	LOWERS RIVER BED		88.5	89.2	89.7	89.7	89.0	90.2	90.5	91.4	92.8	93.2	93.7
ACCUMULATED DISTANCE (m)				400	800	1,200	1,600	2,000	2,400	2,800	3,200	3,600	4,000
UNIT DISTANCE (m)				400	400	400	400	400	400	400	400	400	400
CROSS SECTION NO			0	1	2	3	4	5	6	7	8	9	10

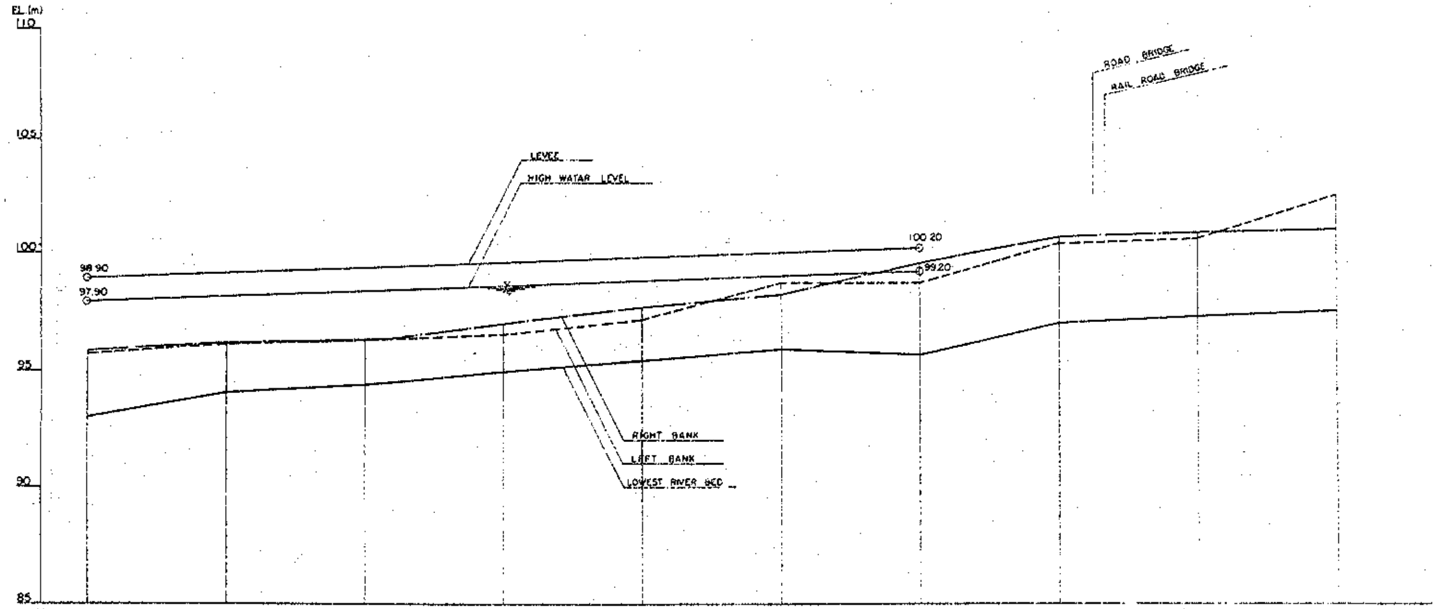
SCALE
H 0 500 1000 (m)
V 0 5 10 (m)

...ATION AND ... RIVER IMPROVMENT
INDONESIA

LONGITUDINAL SECTION of K.GAWE

Date: ... 14

Fig - 3.2.24 LONGITUDINAL SECTION of K.JLANTAH



			0	1	2	3	4	5	6	7	8	9
PLANNED ELEVATION (EL. m)	LEVEE		98.90	99.12	99.33	99.55	99.77	99.98	100.20	-	-	-
	HIGH WATER LEVEL		97.90	98.12	98.33	98.55	98.77	98.96	99.20	-	-	-
PRESENT ELEVATION (EL. m)	RIGHT BANK		95.85	96.14	96.29	96.94	97.53	98.21	99.53	100.68	100.56	101.00
	LEFT BANK		95.75	96.10	96.25	96.50	97.10	98.70	98.70	100.40	100.60	102.45
	LOWEST RIVER BED		93.01	94.06	94.37	94.56	95.32	95.92	95.69	97.00	97.25	97.45
ACCUMULTED DISTANCE (m)			-	600	1,200	1,800	2,400	3,000	3,600	4,200	4,800	5,400
UNIT DISTANCE (m)				600	600	600	600	600	600	600	600	600
CROSS SECTION No			0	1	2	3	4	5	6	7	8	9

SCALE
H 0 500 1000 (m)
V 0 5 10 (m)

WONOGIRI IRRIGATION AND
UPPER SALA RIVER IMPROVMENT
INDONESIA

LONGITUDINAL SECTION
of K.JLANTAH

Date: June 24, 1976 | DWG. No. WR. 15

Fig- 3.2.25 Specific flow ~ Area for a Drainage CHANNEL.

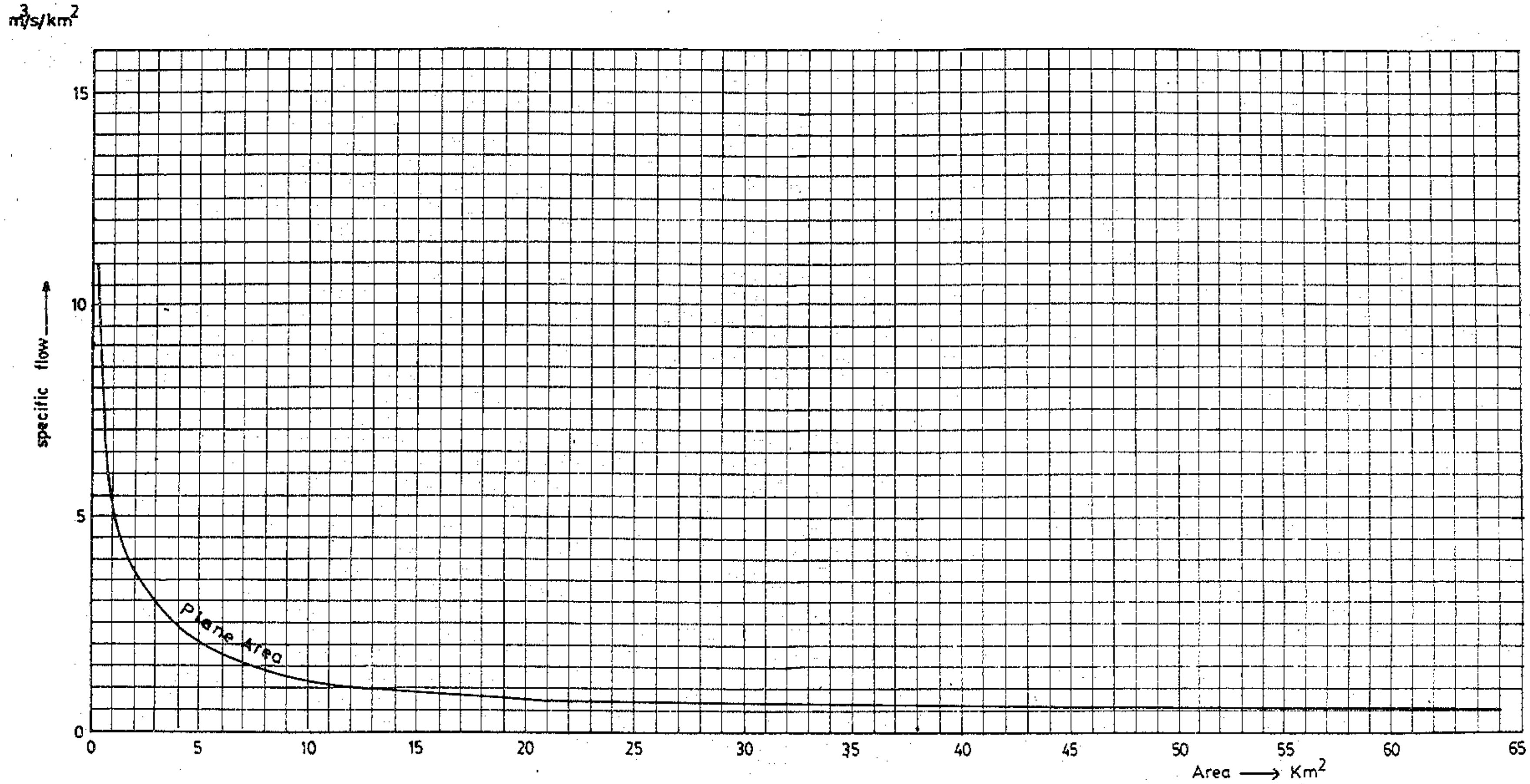


Fig-3.2.26 Location of the Proposed Bridge and Groundsill

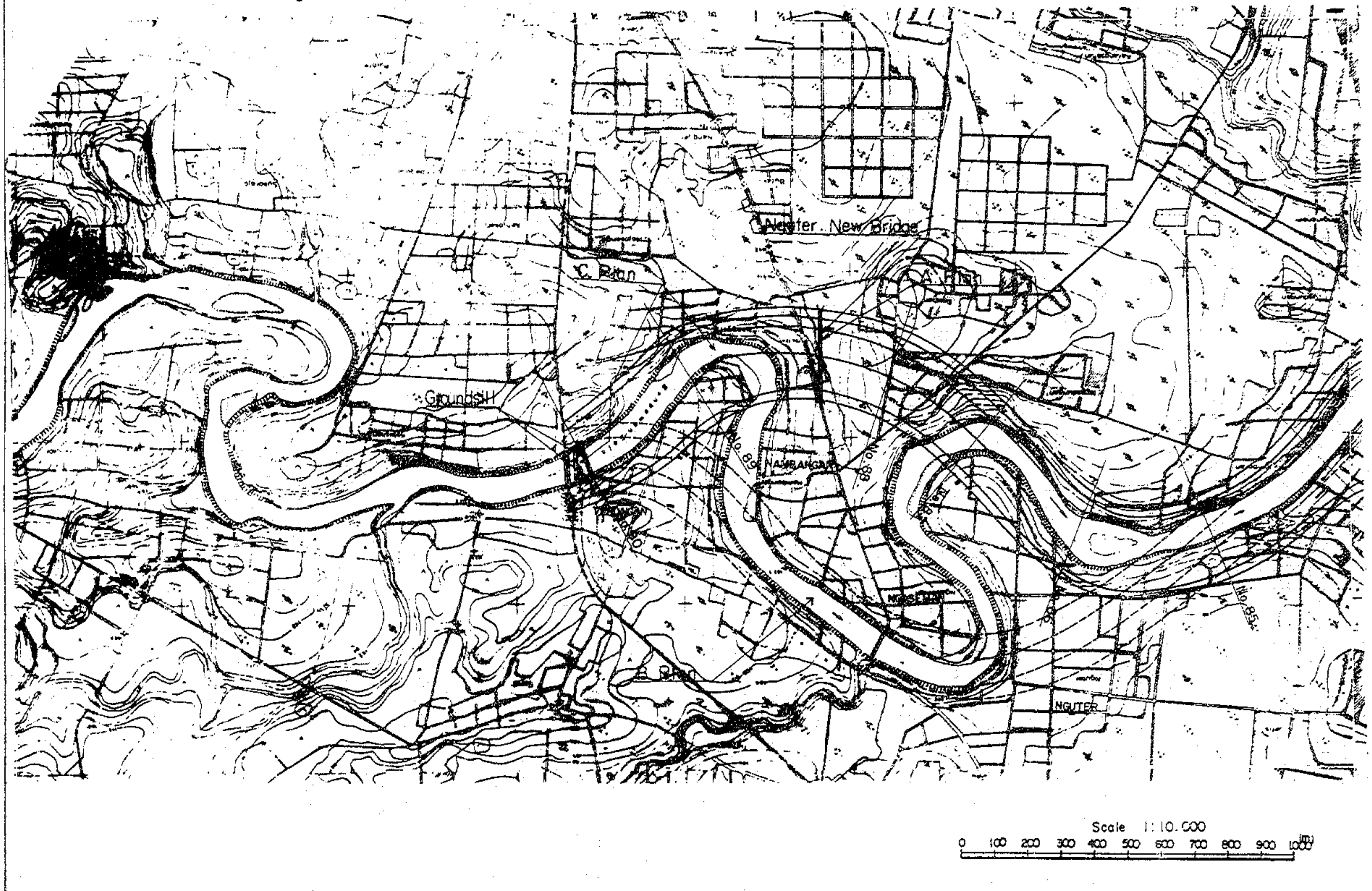


Table 3.2.1 Discharge Capacity at the Proposed Cross Section of Bridge Site

	$Q(m^3/s)$	V(m/s)	$A(m^2)$	I	Bridge Length
Nguter Railway Bridge	1,112	2.06	540	1/1,450	119
Nguter Road Bridge	1,060	2.12	500	1/1,450	106
Bacem Road Bridge	1,720	2.45	840	1/2,000	116
Mojo Railway Bridge	2,390	2.32	1,030	1/2,000	149
Jurug Railway Bridge	3,381	2.30	1,470	1/3,000	178
Jurug Road bridge	3,675	2.50	1,470	1/2,000	169

Note: Q : discharge

V : velocity of flow

A : area of cross - section

I : slope of river bed

Table 3.2.2 Comparison of Construction Cost of the new Bridge near Nguter

Plan	Item	Volume	Construction Cost	Description
A	Excavation	210,000m^3	278,000 US$	in case of including construction cost of new Bridge (L=90m W=5m) over the Existing River 951,000 US$
	Banking	36,000m^3	46,000	
	Bridge	L=209m	539,000	
	Land acquisition		26,000	
	Total		889,000 US$	
B	Excavation	385,000m^3	492,000	
	Banking	66,000m^3	85,000	
	Land acquisition		48,000	
	Total		625,000	
C	Excavation	210,000m^3	278,000	in case of including construction Cost of New Bridge over the existing River 887,000 US$
	Banking	36,000m^3	46,000	
	Bridge	L = 10m	26,000	
	Access road	2,000m	342,000	
	Land acquisition for river channel		26,000	
	Land acquisition for road		6,000	
	Total		724,000	

3.3 FLOOD DAMAGE

3.3.1 General

For the purpose of estimation of flood damage which may occur, the best way is to estimate the potential losses from the actual losses in the past.

Flood damage consists of direct damage, indirect damage and intagible damage. Direct damage includes damages to building and crops plus damages to transportation and utility facilities.

Indirect damages include the travel cost of detouring around the flood area, and the "net loss of normal profit and earning to capital management and labor in the identifiable zone of flood influence", and net increase in the cost of forecasting and warning, evacuating and reoccupying flood-threatened areas are also included.

Intagible damages comprise loss of life, impairment of public health due to the outbreak of contegious diseases, insects, and adverse effect on national economy resulting from temporary closure of major transportation arteries.

In this study, only the damages to building and farm crops are assessed and incorporated in the direct damage. Indirect damage is estimated by applying a fixed percentage to the direct damage in view of the limited time available for the analysis. Intangible damage is excluded in this study.

Average annual flood damage is determined by summing up the potential direct damage from floods of a number of different frequencies, and potential indirect damage is estimated by applying an indirect damage factor.

The so-called Surakarta Area and Sragan Area are defined as follows.

Surakarta Area : from Wonogiri to Jurug including Sala City.

Sragan Area : from Jurug to the confluence of K. Sawur.

For the Surakarta area, the calculation was made on the annual flood lamages under various conditions; such as existing condition after the completion of the main coffer dam for Wonogiri dam, after Wonogiri dam construction and after the river improvement. Thereafter, the calculation was made on the annual benefit from each individual work.

As regards the Sragen area, the calculation of the annual flood damage under the existing condition after completion of the river improvement works was made. Also, the amount of annual decrease in the flood damage resulting from the installation of the upstream flood control facilities such as the dam and the river improvement works was calculated.

3.3.2 Inundation Area

The inundation area and inundation duration of the past floods and the new 1/5,000 topographic map were used to estimate the inundation area in both Surakarta Area and Sragen area for the peak discharges such as 500 m^3/s, 1,000 m^3/s, 1,500 m^3/s, 2,000 m^3/s and 2,500 m^3/s.

Results of the above study are shown in Table 3.3.1.

As the result of the above study, it was estimated that during the inundation duration of three days, the inundation area in Surakarta Area and Sragen Area was 202 km^2 and 97 km^2 respectively at 2,000 m^3/s scale flood which occurred in 1966.

3.3.3 House damage

"House damage" consists of damages to buildings and household-effects. Sala city is the urban area in the inundated area.

House damage was caused mainly to Sala city, so this urban area may well be separated from other inundation areas.

(1) Value of building and household effects.

The values of building and household effects of an average household in Sala city and other areas are presumed as shown in Table 3.3.2 at the investigation conducted this time. The percentage of the buildings in each area has been presumed as shown in Table 3.3.3 based on the data which include the Sala city and the inundation area.

Presumed average amount of assets per household based on the result of the above study is as shown in Table 3.3.4 and the chart below.

(unit :US$/house)

Item	Sala-city	Other inundated areas
Building	3,875	393
Household effects	2,570	221
Total	6,427	614

(2) Damage ratio

Table 3.3.5 shows the flood damage rate of building and household effects, relating to flood depth above floor level. In this table, the damage rate of building is quoted from the rate applied to the case in which the ground gradient of the inundated area is fairly gentle, and the damage rate of household effects is the rate adopted in the "Feasibility Study on Surabaya River Improvement" prepared by the Japanese Survey Team in 1973.

Flood damage rates used in this study are as shown in Table 3.3.6, which are estimated collectively by applying the sharing ratio mentioned in the above sub-section.

(3) House damage potential.

House damage potential will be obtained from the calculation of "average amount of assets per household" which is described in paragraph (1) multiplied by "damage ratio" and as shown in Table 3.3.6. House damage potential per household classified by the depth of inundation is as shown in Table 3.3.7 and the chart below.

(unit :US$/house)

Item	Ponding depth above floor - level (m)						
	0.0 0.5	0.5 1.0	1.0 1.5	1.5 2.0	2.0 2.5	2.5 3.0	Over 3.0
Sala city	476	990	1,414	1,645	1,799	2,005	2,185
Other inundated area	40	83	114	133	146	162	177

Note: Ponding depth means the depth over floor-level.

3.3.4 Crop Damage

Crop Damage is classified into the damage to the paddy field and the Polowijo damage.

Although, after completion of the projects, the whole inundated area in Surakarta area will be included in the project area. On the other hand, in the Sragen area, i.e., only the right-bank side of the inundated area, will be included in the project area. Therefore, the left bank side of the Sragen area will be outside of the project area.

(1) Value of Crops

The present production of paddy per ha. under the existing condition and the harvest of Polowijo have presumed as shown in Table 3.3.8, based on the results of investigation conducted this time. The rates of paddy field and Polowijo under the existing situation are as shown in Table 3.3.9.

In the project area on the right bank in the Sragen area, Polowijo will be switched to the paddy with the expectation of increasing the harvest, and the anticipated harvest has been estimated as shown in Table 3.3.10.

As described above, the harvest per ha. will reach the level as shown in Table 3.3.11 and the chart below.

(unit: US$/ha.)

Item	Condition	Area	Paddy field	Polowijo	Total
Surakarta area	Existing condition	Whole area	367	19	386
Sragen area	Existing condition	Whole area	377	18	395
	After project	Non-project	377	18	395
	Completion	Project area	782	0	782

Note: Sragen area on the right bank side will be included in the irrigation project area

(2) Damage ratio

Damage rate of paddy field correlates with flooding depth, flooding duration and growing stage of crops in the flood season. Table 3.3.12 shows the flood damage rate of paddy adopted in "Feasibility Study on Surabaya River Improvement." Based on this, Table 3.3.13 was prepared for this project area.

The average damage rate is estimated by assuming that the rate for the respective periods of tillering stage, booting stage, heading time and ripening stage was 2 %, 24 %, 35 % and 39 % respectively on the average in flood season.

As to Polowijo crops such as cassava, "peanut and soybean, damages are considered to be larger when compared with paddy under the same condition of inundation because their roots are weak against inundation.

In this study, the flood damage rate usually used in Japan is considered applicable. The flood damage rate of Polowijo crops is summarized in Table 3.3.14.

(3) Crop damage potential

Crop damage potential per ha. is calculated as the equivalent of the sum of the amount of assets of the paddy field per ha. times damage ratio plus the amount of assets of Polowijo crop per ha. times damage ratio. The results of the above calculation are as shown in Table 3.3.15.

3.3.5 Total Damage

1) Direct Damage

The damage to building and farm crops is estimated as the direct damage in the following manner.

Flood damage is obtainable from which the sum of (house damage in inundation area and crop damage per ha.) x (Area classified by flood scale shown in the Table 3.3.1), however, the calculation had been made with paying attention to the following terms.

a) The number of houses per ha. is presumed as follows. It was calculated from the area of the village located in the inundation area, and also the number of houses, density of houses of 3.36 houses/ha. in Surakarta area, and 1.96 houses/ha. in Sragen area.

b) Houses in the inundation area are constructed on the banks about 0.5 m above ground surface for protection against frequent floods. It is considered that the damage to houses is caused when the depth of inundation exceeds 0.5 m.

Based on (a) and (b), the house damage potential for every flood scale is calculated in Table 3.3.16.

c) At present, there are some places in the lower land for which the evaluation of damages to crops caused by frequent floods is almost impossible.

d) Unplanted area in the flood season is estimated at 900 ha. in Surakarta area from the data on crop planting, field survey and the analysis of the flooding conditions in each inundation area.

	Surakarta area	Sragen area
Unplanted area	900 ha.	0

Damage ratio is almost equivalent to 100 % in this area, therefore a damage equivalent to 0.34 x US$$10^6$/year is incurred by the Surakarta Area every year. The flood damage has been calculated for area with the exception of the unplanted area along the river where the inundation depth is largest. The results of calculation are given in Table 3.3.17.

2) Indirect Damage

For the estimate of indirect damage, fixed percentages are applied, namely, 15 per cent to building damage and 10 per cent to crop damage respectively.

3) Total damage

Total damage is calculated as shown in Table 3.3.18. On the other hand, relationship between the discharged scale and the total damage is shown in Fig. 3.3.1 and Fig. 3.3.2.

4) Total damage in landside water

Upon completion of the Wonogiri Dam and the river improvement works flood damage will decrease largely, however, the damage caused by landside water will newly occur in Surakarta area.

There are altogether 15 landside water areas in the Surakarta area. The total damage potential due to the probable discharge at the Jurug site is calculated as shown in Fig. 3.3.3.

The total damage is calculated for 2-cases;

(1) the hydrograph in main channel is fixed to the 1966-Flood having 40-year peak discharge so that the damage to landside water area is calculated severely, (2) the hydrograph is transfigured so that the peak discharge is equal to the N-year peak discharge.

Thus, the problem of the unplanted area in this case is considered to have been solved.

3.4 ANNUAL BENEFIT

Annual benefit, will be given as the difference in the annual damage which is obtained by multiplying the total damage potential by the probable flood rate.

3.4.1 Probable Discharge

Probable discharges under the existing condition and after the completion of main cofferdam and also Wonogiri Dam are as shown in Table 2.6.3.

Probable discharge after the completion of the proposed river improvement is shown in Table 3.2.12. Therefore, how the probable discharge changes with the progress of each stage of construction work is as shown in Figure 3.4.1.

3.4.2 Annual Damage

Annual damage has been calculated for the two areas, namely, (1) Surakarta area, in relation to each stage of construction work; (2) Sragen area under the existing condition and upon completion of the project. The results of calculation are as shown in the following table.

Calculation table concerning the annual damage is shown in Tables 3.4.1 to 3.4.10.

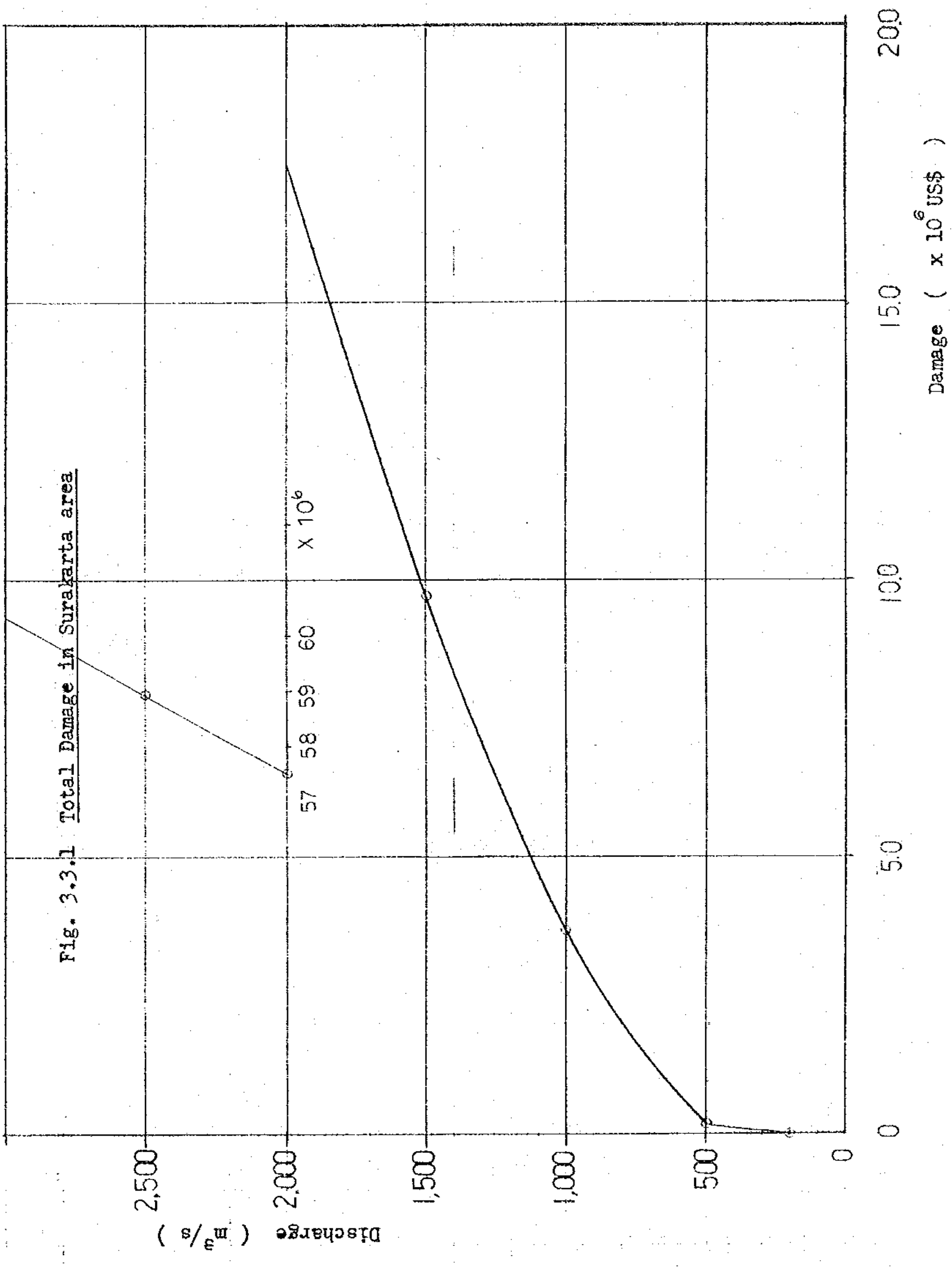


Fig. 3.3.1 Total Damage in Surakarta area

Fig. 3.3.2 Total Damage in Sragen Area

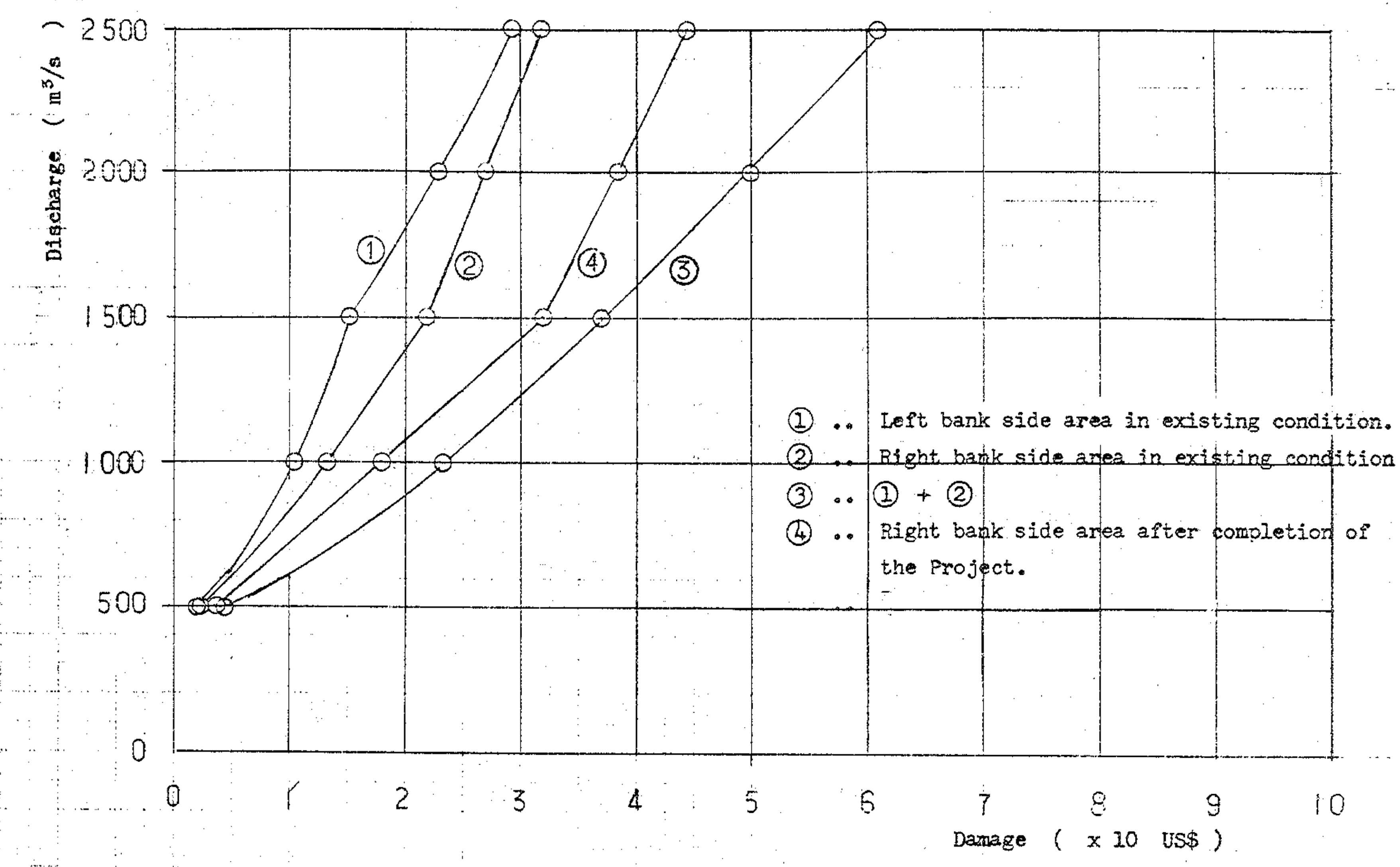


Fig. 3.3.3 Total Inner water Damage in Surakarta area

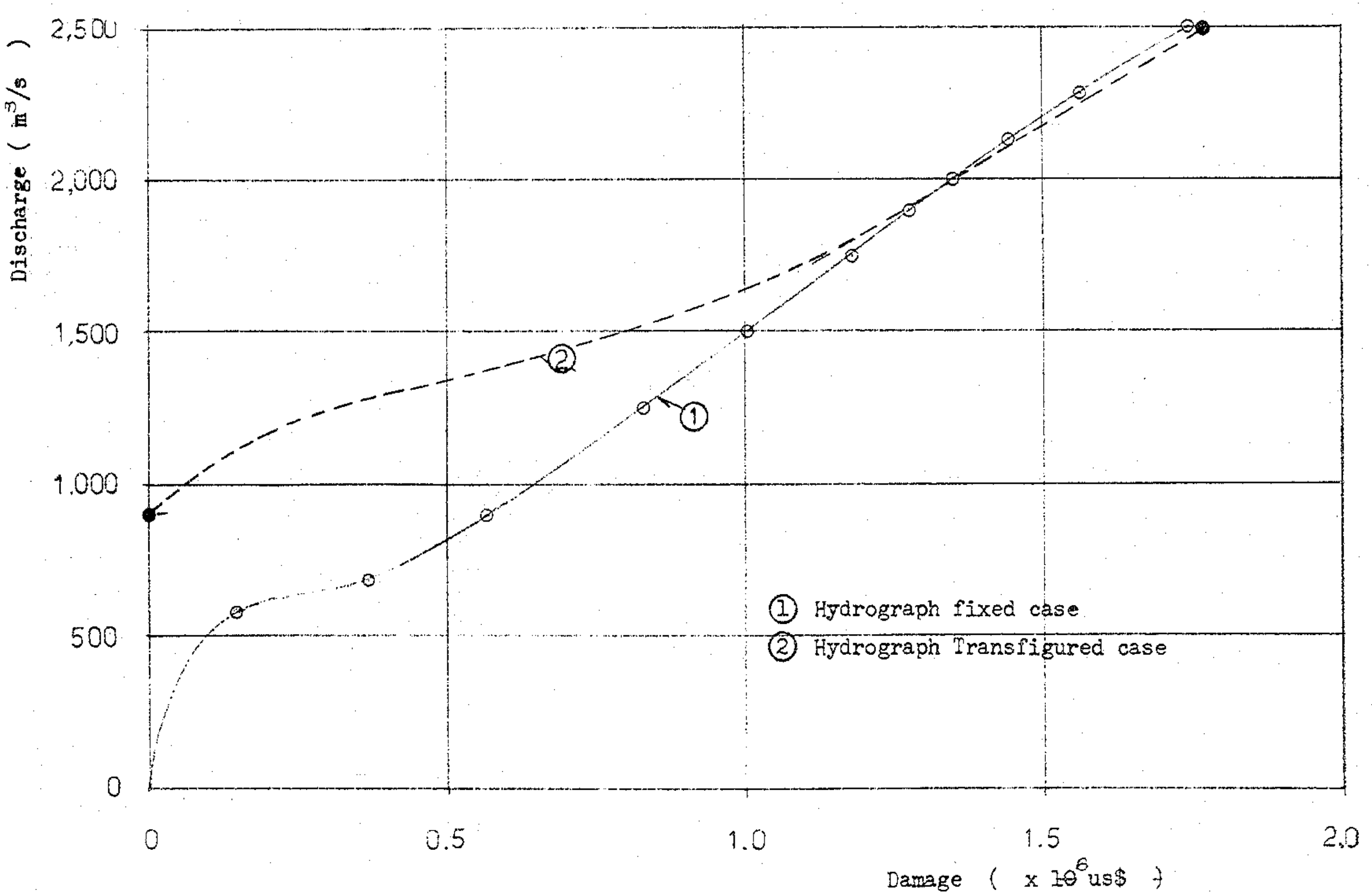


Table 3.3.1(a) Inundation Area under Various Flood Scale

(Surakarta Area)

Discharge at Surakarta (m^3/s)	Flooding Duration (day)	Total Inun-dation Area (ha)	Inundation Area at Every Flooding Depth (ha)									
			0 (m) -0.5	0.5 -1.0	1.0 -1.5	1.5 -2.0	2.0 -2.5	2.5 -3.0	3.0 -3.5	3.5 -4.0	4.0 -4.5	4.5 -5.0
500	1	2,900	2,900									
1,000	2	10,000	1,800	2,200	2,600	3,400						
1,500	3	15,500	1,500	1,800	1,800	2,200	2,200	2,600	3,400			
2,000	3	20,200	1,600	2,600	2,000	1,800	1,800	2,200	2,200	2,600	3,400	
2,500	4	23,200	2,000	2,600	2,600	2,000	1,800	1,800	2,200	2,200	2,600	3,400

(Sragen area in Total)

Discharge at Surakarta (m^3/s)	Flooding Duration (day)	Total Inun-dation Area (ha)	Inundation Area at Every Flooding Depth (ha)									
			0 (m) -0.5	0.5 -1.0	1.0 -1.5	1.5 -2.0	2.0 -2.5	2.5 -3.0	3.0 -3.5	3.5 4.0	4.0 4.5	4.5 5.0
500	1	3,800	1,600	2,200								
1,000	2	6,900	400	1,000	1,400	1,900	2,200					
1,500	3	8,400	900	1,000	1,000	1,400	1,900	2,200				
2,000	3	9,700	500	800	900	1,000	1,000	1,400	1,900	2,200		
2,500	4	10,900	100	800	800	800	900	1,000	1,000	1,400	1,900	2,200

Table 3.3.1(b) Inundation Area under Various Flood Scale

(Sragen Area of Right Bank Side)

Discharge at Surakarta (m^3/s)	Flooding Duration (day)	Total Inundation Area (ha)	Inundation area at Every Flooding Depth(ha)									
			0 (m) -0.5	0.5 -1.0	1.0 -1.5	1.5 -2.0	2.0 -2.5	2.5 -3.0	3.0 -3.5	3.5 -4.0	4.0 -4.5	4.5 -5.0
500	1	2,014	848	1,166								
1,000	2	3,933	228	570	798	1,083	1,254					
1,500	3	4,956	531	590	590	826	1,121	1,298				
2,000	3	5,238	270	432	486	540	540	756	1,026	1,188		
2,500	4	5,668	52	416	416	416	468	520	520	728	988	1,144

(Sragen Area of Left Bank Side)

Discharge at Surakarta (m^3/s)	Flooding Duration (day)	Total Inundation Area (ha)	Inundation Area at Every Flooding Depth (ha)									
			0 (m) -0.5	0.5 -1.0	1.0 -1.5	1.5 -2.0	2.0 -2.5	2.5 -3.0	3.0 -3.5	3.5 -4.0	4.0 -4.5	4.5 -5.0
500	1	1,786	752	1,034								
1,000	2	2,967	172	430	602	817	946					
1,500	3	3,444	369	410	410	574	779	902				
2,000	3	4,462	230	368	414	460	460	644	874	1,012		
2,500	4	5,232	48	384	384	384	432	480	480	672	912	1,056

Table 3.3.2 Value of Building and Household Effects

	Surakarta City		Other Inundation Area	
	Building (US$)	Household effects (US$)	Building (US$)	Household effects (US$)
Farmer's House General	290	150	290	150
Residence	3,610	1,620	750	440
Shop	3,760	12,720	810	2,000
Office	11,190	9,440	2,230	410

Table 3.3.3 Occupancy Ratio

	Surakarta City	Other Inundation Area
Farmer's House General	2%	81%
Residence	88%	17%
Shop	6%	1%
Others	4%	1%

Table 3.3.4(a) Average Value of Building and Household Effects

a) Surakarta City

(US$)

	Building			Household Effect		
	(1)	(2)	3=(1) x (2)	(1)	(2)	3=(1) x (2)
	US$	Nate(%)		US$	Nate(%)	
Farmer's House General	290	2	6	150	2	3
Residence	3,610	88	3,177	1,620	88	1,426
Shop	3,760	6	226	12,720	6	763
Office	11,190	4	448	9,440	4	378
Total			3,857			2,570

Total = 3,857 + 2,570 = 6,427/house

Table 3.3.4(b) Average Value of Building and Household Effects

b) Other Inundation Area

	Building			Household Effect		
	(1)	(2)	3=(1) x (2)	(1)	(2)	3=(1) x (2)
	US$	Nate(%)		US$	Nate(%)	
Farmer's House Gereral	290	81	235	150	281	122
Residence	750	17	128	440	17	75
Shop	810	1	8	2,000	1	20
Office	2,230	1	22	410	1	4
Total			393			221

Total = 393 + 221 = 614/house

Table 3.3.5 Flood Damage Rate of Building and Household Effects (An Instance in Surabaya Area)

Height above Floor Level (m)	Building	Household Effects				
		Farmer's House	Residence	Shop	Office	School & Factory
0 - 0.5	0.05	0.09	0.11	0.08	0.09	0.08
0.5 - 1.0	0.07	0.24	0.29	0.22	0.28	0.24
1.0 - 1.5	0.11	0.33	0.41	0.35	0.42	0.35
1.5 - 2.0	0.11	0.37	0.47	0.44	0.47	0.39
2.0 - 2.5	0.15	0.39	0.49	0.51	0.49	0.40
2.5 - 3.0	0.15	0.39	0.51	0.57	0.49	0.41
Over 3.0	0.22					

Table 3.3.6 Flood Damage Rate of Building and Household Effects (Used in This Study)

Ponding Depth above Floor Level (m)	Surakarta City		Other Inundation Area	
	Building	Household Effects	Building	Household Effects
0 - 0.5	0.05	0.11	0.05	0.09
0.5 - 1.0	0.07	0.28	0.07	0.25
1.0 -- 1.5	0.10	0.40	0.10	0.34
1.5 - 2.0	0.12	0.46	0.12	0.39
2.0 - 2.5	0.14	0.49	0.14	0.41
2.5 - 3.0	0.18	0.51	0.18	0.41
Over 3.0	0.22	0.52	0.22	0.41

Table 3.3.7 Amount of House Damage per Household

Area	Ponding Depth above Floor Level	A	0 - 0.5m		0.5 - 1.0m		1.0 - 1.5m		1.5 - 2.0m		2.0 - 2.5m		2.5 - 3.0m		Over 3.0m	
			B	C	B	C	B	C	B	C	B	C	B	C	B	C
Surakarta City	Building	3,857	0.05	193	0.07	270	0.10	386	0.12	463	0.14	540	0.18	694	0.22	849
	Household Effects	2,570	0.11	283	0.28	720	0.40	1,028	0.46	1,182	0.49	1,259	0.51	1,311	0.52	1,336
	Total	-	-	476	-	990	-	1,414	-	1,645	-	1,799	-	2,005	-	2,185
Other Inundation Area	Building	393	0.05	20	0.07	28	0.10	39	0.12	47	0.14	55	0.18	71	0.22	86
	Household Effects	221	0.09	20	0.25	55	0.34	75	0.39	86	0.41	91	0.41	91	0.41	91
	Total	-	-	40	-	83	-	114	-	133	-	146	-	162	-	177

Note:
A: Average Value (U.S.$)
B: Damage Rate (U.S.$)
C: Damage (U.S.$)

Table 3.3.8 Value of Crop per Hectare

Item	Value	
Paddy Field	200,600 Rp/ha	(483 US$/ha)
Polowijo	33,000 Rp/ha	(80 US$/ha)

Table 3.3.9 Sharing Ratio of Each Field

Item	Paddy Land (%)	Polowijo (%)	Others (%)
Surakarta Area	76	24	-
Sragen Area	78	22	-

Table 3.3.10 Condition in Sragen Area after the Completion of Project

Area	Sharing Ratio		Value of Crop
	Paddy	Polowijo	Paddy
Right Back Area	100 %	0 %	782 US$/ha
Left Bank Area	78	22	483

Table 3.3.11 Average Value of Crops per ha

Item	Surakarta Area			Sragen Area		
	Value of Crops (SU$)	Sharing Ratio (%)	Total (US$)	Value of Crops (US$)	Sharing (%)	Total (US$)
Paddy	483	76	367	483	78	377
Polowijo	80	24	19	80	22	18
Total	563	100	386	563	100	395

Table 3.3.12 Flood Damage Rate of Paddy in Surabaya River Basin

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 3.3.13 Flood Damage Rate of Paddy in This Study

Flooding Depth(m)	Flooding Duration (day)	Tillering Stage *	**	***	Booting Stage *	**	***	Heading Stage *	**	***	Ripening *	**	***	Average
0 - 0.5	1 - 2	6%	2%	0	37%	24%	9	8%	35%	3	2%	39%	1	13%
	3 - 4	9	2	0	42	24	10	22	35	8	4	39	2	20
	5 - 6	14	2	0	45	24	11	25	35	9	6	39	2	22
	Over 7	16	2	0	50	24	13	28	35	10	6	39	2	25
0.5 - 1.0	1 - 2	10	2	0	40	24	10	10	35	3	4	39	2	15
	3 - 4	20	2	0	46	24	11	23	35	8	15	39	6	25
	5 - 6	30	2	1	49	24	11	26	35	9	23	39	9	30
	Over 7	35	2	1	55	24	13	30	35	10	23	39	9	33
Over 1.0	1 - 2	10	2	0	70	24	17	30	35	11	5	39	2	30
	3 - 4	20	2	0	80	24	19	80	35	28	20	39	8	55
	5 - 6	30	2	1	85	24	20	90	35	31	30	39	12	64
	Over 7	35	2	1	95	24	23	100	35	35	30	39	12	70

Note: * Estimated Flood Damage Rate

** Ratio of the Damage for each Stage of Crop

*** Weighted Average

Table 3.3.14 Flood Damage Rate of Polowijo

Flooding Depth (m)	Flooding Duration (day)	Paddy (%)	Polowijo Crops (%)
0 - 0.5	1 - 2	13	27
	3 - 4	20	42
	5 - 6	22	54
	Over 7	25	67
0.5 - 1.0	1 - 2	15	35
	3 - 4	25	48
	5 - 6	30	67
	Over 7	33	74
Over 1.0	1 - 2	30	51
	3 - 4	55	67
	5 - 6	64	81
	Over 7	70	91

Table 3.3.15 Amount of Crop Damage per ha.

Area	flooding duration / depth	1 - 2 days					3 - 4 days					5 - 6 days					Over 7 days				
		A US$	B (%)	C US$	D (%)	Total	A US$	B (%)	C US$	D (%)	Total	A US$	B (%)	C US$	D (%)	Total	A US$	B (%)	C US$	D (%)	Total
Surakarta area	0 - 0.5m	367	13	19	27	53	367	20	19	42	81	367	22	19	54	91	367	25	19	67	104
	0.5 - 1.0m	367	15	19	35	62	367	25	19	48	101	367	30	19	67	123	367	33	19	74	135
	Over 1.0m	367	30	19	51	120	367	55	19	67	215	367	64	19	81	250	367	70	19	91	274
Sragen area (Non-Project area)	0 - 0.5m	377	13	18	29	54	377	20	18	42	83	377	22	18	54	93	377	25	18	67	106
	0.5 - 1.0m	377	15	18	35	63	377	25	18	48	103	377	30	18	67	125	377	33	18	74	138
	Over 1.0m	377	30	18	51	122	377	55	18	67	219	377	64	18	81	256	377	70	18	91	280
Sragen area (Project area)	0 - 0.5m	782	13	0	0	102	782	20	0	0	156	782	22	0	0	172	782	25	0	0	196
	0.5 - 1.0m	782	15	0	0	117	782	25	0	0	196	782	30	0	0	235	782	33	0	0	258
	Over 1.0m	782	30	0	0	235	782	55	0	0	430	782	64	0	0	500	782	70	0	0	547

Note
- A : Average value of Paddy per ha. (US$/ha.)
- B : Flood Damage rate of Paddy crops (%)
- C : Flood Damage value of Polowijo (US$/ha.)
- D : Flood Damage rate of Polowijo (%)
- Total : Total Damage value = (A x B) + (C x D)

Table 3.3.16 (a) Potential House Damage in Surakarta Area

Discharge	Depth / Area	0 - 0.5m			0.5 - 1.0m			1.0 - 1.5m			1.5 - 2.0m			2.0 - 2.5m			2.5 - 3.0m			Over 3.0m			T o t a l	
		A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	C
500 m^3/s	Solo city	-	-	-																			0	0
	Surakarta area	-	-	-																			0	0
	Sub total	-	-	-																			0	0
1,000 m^3/s	Solo city	-	-	-																			0	0
	Surakarta area	7,400	40	296	8,700	83	725	11,400	114	1,300													27,500	2,321
	Sub total	7,400	40	296	8,700	83	725	11,400	114	1,300													27,500	2,321
1,500 m^3/s	Solo city	-	-	-																			0	0
	Surakarta area	6,048	40	242	6,048	83	502	7,392	114	843	7,392	133	983	8,736	146	1,275	11,424	162	1,851				47,040	5,696
	Sub total	6,048	40	242	6,048	83	502	7,392	114	843	7,392	133	983	8,736	146	1,275	11,424	162	1,851				47,040	5,696
2,000 m^3/s	Solo city	-	-	-	-	-	-	26,900	1,414	38,037													26,900	38,037
	Surakarta area	8,736	40	349	6,720	83	558	6,048	114	689	6,048	133	804	7,392	146	1,079	7,392	162	1,198	20,160	177	3,568	62,500	8,245
	Sub total	8,736	40	349	6,720	83	558	32,948	1,528	38,726	6,048	133	804	7,392	146	1,079	7,392	162	1,198	20,160	177	3,568	89,400	46,282
2,500 m^3/s	Solo city	-	-	-	-	-	-	26,900	1,414	38,037													26.900	38,037
	Surakarta area	8,736	40	349	8,736	83	725	6,720	114	766	6,048	133	804	6,048	146	883	7,392	162	1,198	27,552	177	4,877	71,200	9,602
	Sub total	8,736	40	349	8,736	83	725	33,620	1,528	38,808	6,048	133	804	6,048	146	883	7,392	162	1,198	27,552	177	4,877	98,100	47,639

Note : A : Number of House (House)

B : Damage per House (US$/house)

C : Damage ($x10^3$ US$)

Table 3.3.16 - (b) Potential House Damage in Sragen area

(a) Left bank side area

Left

Depth	0 - 0.5m			0.5 - 1.0m			1.0 - 1.5m			1.5 - 2.0m			2.0 - 2.5m			2.5 - 3.0m			Over 3.0m			Total	
Discharge (m3/S)	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	C
500	2,027	40	81																			3,501	81
1,000	843	40	34	1,180	83	98	1,601	114	183	1,854	133	247										5,815	562
1,500	804	40	32	804	83	67	1,125	114	128	1,527	133	203	1,768	146	258							6,751	688
2,000	721	40	29	811	83	67	902	114	103	902	133	120	1,262	146	184	1,713	162	278	1,984	177	351	8,746	1,132
2,500	753	40	30	753	83	62	753	114	86	847	133	113	941	146	137	941	162	152	5,175	177	915	10,257	1,495

(b) Right bank side area

Right

Depth	0 - 0.5m			0.5 - 1.0m			1.0 - 1.5m			1.5 - 2.0m			2.0 - 2.5m			2.5 - 3.0m			Over 3.0m			Total	
Discharge (m3/S)	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	C
500	2,285	40	91																			3,947	91
1,000	1,117	40	45	1,564	83	130	2,123	114	242	2,458	133	327										5,586	744
1,500	1,156	40	46	1,156	83	96	1,619	114	185	2,197	133	292	2,544	146	371							9,713	990
2,000	847	40	34	953	83	79	1,058	114	121	1,058	133	141	1,482	146	216	2,011	162	326	2,328	177	412	10,266	1,329
2,500	815	40	33	815	83	68	815	114	93	917	133	122	1,019	146	149	1,019	162	165	5,605	177	993	11,107	6,623

Note A : Number of House (house)

B : Damage per House (US$/house)

C : Damage ($x10^3$ US$)

Table 3.3.17 Amount of Crop Damage for Every Flood Scale

a) Surakarta area (x10^3 US$)

Discharge (m^3/S)	0.0 - 0.5m			0.5 - 1.0m			Over 1.0m			Total	
	A	B	C	A	B	C	A	B	C	A	C
500	(900)	(386)	(347)	-			-			(2,700)	(453)
	2,000	53	106	-			-			2,000	106
1,000	1,800	53	95	2,200	62	136	6,000	120	720	10,000	951
1,500	1,500	81	122	1,800	101	182	12,200	215	2,623	15,500	2,927
2,000	1,600	81	207	2,600	101	263	16,000	215	3,440	20,200	3,910
2,500	2,000	81	162	2,600	101	263	18,200	215	3,913	23,200	4,338

b) Sragen left bank side area (x10^3 US$)

Discharge (m^3/S)	0.0 - 0.5m			0.5 - 1.0m			Over 1.0m			Total	
	A	B	C	A	B	C	A	B	C	A	C
500	752	54	41	1,034	63	65	-	-	-	1,786	106
1,000	172	54	9	430	63	27	2,365	122	289	2,967	325
1,500	369	83	31	410	103	42	2,665	219	584	3,444	657
2,000	230	83	19	368	103	38	3,864	219	846	4,462	903
2,500	48	83	4	384	103	40	4,800	219	1,051	5,232	1,095

c) Sragen right bank side area (x10^3 US$)

Discharge (m^3/S)	0.0 - 0.5m			0.5 - 1.0m			Over 1.0m			Total	
	A	B	C	A	B	C	A	B	C	A	C
500	848	54	46	1,166	63	73	-	-	-	2,014	119
1,000	228	54	12	570	63	36	3,135	122	382	3,933	430
1,500	531	83	44	590	103	61	3,835	219	840	4,956	945
2,000	270	83	22	432	103	44	4,536	219	993	5,238	1,059
2,500	52	83	4	416	103	43	5,200	219	1,139	5,668	1,186

d) Sragen right bank side area (with Project) (x10^3 US$)

Discharge (m^3/S)	0.0 - 0.5m			0.5 - 1.0m			Over 1.0m			Total	
	A	B	C	A	B	C	A	B	C	A	C
500	848	102	86	1,166	117	136	-	-	-	2,014	222
1,000	228	102	23	570	117	67	3,135	235	737	3,933	827
1,500	531	156	83	590	196	116	3,835	430	1,649	4,956	1,848
2,000	270	156	142	432	196	85	4,536	430	1,950	5,238	2,077
2,500	52	156	8	416	196	82	5,200	430	2,236	5,668	2,326

Note : A = Inundation area (ha)
B = Amount of crop damage per ha (US$)
C = Total crop damage (10^3 US$)

Table 3.3.18 Total Flood Damage

a) Surakarta Area (x10^3 US$)

Discharge at Surakarta	Direct Damage		Indirect Damage		Total
	Building	Crop	Building (15%)	Crop (10%)	
m^3/S					
500	0	453	0	45	151
1,000	2,318	951	348	95	3,712
1,500	5,690	2,927	854	293	9,764
2,000	46,282	3,910	6,942	391	57,525
2,500	47,639	4,338	7,146	434	58,945

b) Sragen Left bank side Area (x10^3 US$)

Discharge at Surakarta	Direct Damage		Indirect Damage		Total
	Building	Crop	Building (15%)	Crop (10%)	
m^3/S					
500	81	106	12	11	210
1,000	562	325	84	33	1,004
1,500	688	657	103	66	1,514
2,000	1,132	903	170	90	2,295
2,500	1,495	1,095	224	110	2,924

c) Sragen Right bank side Area (x10^3 US$)

Discharge at Surakarta	Direct Damage		Indirect Damage		Total
	Building	Crop	Building (15%)	Crop (10%)	
m^3/S					
500	91	119	14	12	236
1,000	744	430	112	43	1,329
1,500	990	945	149	95	2,179
2,000	1,329	1,059	199	106	2,693
2,500	1,623	1,186	243	119	3,171

d) Sragen Right bank side Area (with Project) (x10^3 US$)

Discharge at Surakarta	Direct Damage		Indirect Damage		Total
	Building	Crop	Building (15%)	Crop (10%)	
m^3/S					
500	91	222	14	22	349
1,000	744	827	112	83	1,766
1,500	990	1,848	149	185	3,172
2,000	1,329	2,077	199	208	3,813
2,500	1,623	2,326	243	233	4,425

(unit: x US$ 10^6/year)

	Construction Stage				Inner Water
	Stage 0	Stage 1	Stage 2	Stage 3	
Surakarta area	6.56	4.97	4.27	0.15	0.87 (0.17)*
Sragen R - area	1.98	-	-	2.72	
L - area	1.58			1.58	
T - area	3.56			4.30	

Note: Stage 0 ..... Existing condition
Stage 1 ..... Main coffer dam + Existing river
Stage 2 ..... Wonogiri dam + Existing river
Stage 3 ..... Wonogiri dam + Proposed river improvement
R-area = Right bank side area
L- " = Left "
T- " = Total area
* Inner water damage ... 0.87 = Hydrograph fixed case
" ... 0.17 = Hydrograph transfigured case

3.4.3 Annual Benefit

(1) Annual Benefit in the construction stages.

From the above table, the annual benefit (B_1) is 1.59 x US$ 10^6/year upon completion of the main coffer dam.

$$B_1 = (6.56 - 4.97) \times 10^6 = 1.59 \times \text{US\$ } 10^6/\text{year}.$$

Annual Benefit (B_2) upon completion of Wonogiri dam is 2.29 x US$ 10^6/year, the difference between this and the figure upon completion of Coffer dam is 0.70 x US$ 10^6/year

$$B_2 = (6.56 - 4.27) \times 10^6 = 2.29 \times \text{US\$ } 10^6/\text{year}$$

Thus, the Annual Benefit (B_3) due to the completion of proposed river improvement will be 5.54 x US$ 10^6/year when landside water damage is taken into consideration.

$$B_3 = (6.56 - 0.15 - 0.87) = 5.54 \times \text{US\$ } 10^6/\text{year}$$ ...minimum case

$$B_3 = (6.56 - 0.15 - 0.17) = 6.24 \times \text{US\$ } 10^6/\text{year}$$ ...maximum case

(2) Increase of annual benefit with the construction.

The rate of increase of the annual benefit in working period of 6 years after 1978 is provided as follows. Annual benefit from the proposed river improvement could be

$B_3 - B_2 = 3.25 \times US\$ \, 10^6$/year... (Annual benefit minimum case)

The annual benefit obtainable after the completion of each work section is calculated by multiplying the ratio of the area of each work section to the entire inundation area by $3.25 \times US\$ \, 10^6$/year.

(unit: $\times$ US\$ 10^6/year)

Year	1978	1979	1980	1981	1982	1983
Work section No.	1	2	3	4	5	6
Inundation area ratio	34%	16%	10%	16%	13%	11%
Annual benefit in work section	1.10	0.52	0.33	0.52	0.42	0.36
Annual benefit from main coffer dam	1.59	1.59	1.59	-	-	-
Annual benefit from Wonogiri dam	-	-	-	2.29	2.29	2.29
Yearly increase	2.69	0.52	0.33	1.22	0.42	0.36
Annual Total	2.69	3.21	3.54	4.76	5.18	5.54

(3) Negative benefit in Sragen area

The Right-bank side will be included in the irrigation area of this project in Sragen area.

However, since no flood control facility is constructed, its annual damage will increase. The negative benefit in Sragen area is given as $0.72 \times US\$ \, 10^6$/year from the table presented.

3.4.4 Effects of Improved river to the downstream area

The influence of channel inprovement in the Surakarta area to be exerted on the Sragen area could be estimated from the difference between the annual damage caused under the existing condition (Case 1) and the annual damage caused after the completion of the dam construction and river improvement work (Case 2). The value of the latter is obtained by multiplying the potential damage under the existing condition in Sragen area by the probable discharge rate at the time of completion of the dam construction and the river improvement work.

The table below shows the comparison of the annual damages in Sragen area obtained.

The difference between the two kinds of annual damages is almost negligable. Since the increase of damage which is caused by increased frequency of middle class floods and the decrease of damage from large scale floods due to the effect of flood control cancel each other, it is considered that the river improvement work on the upstream will not exert any adverse influence on the Sragen area.

(unit : x US$ 10^6/year)

	Case 1	Case 2	Difference
Annual damage in Sragen area	3.56	3.56	0

Note : Case 1 = {damage potential under existing condition} x {probable discharge under existing condition}

Case 2 = { " } x {probable discharge after river improvement}

Fig. 3.4.1 Probable Discharge at Surakarta

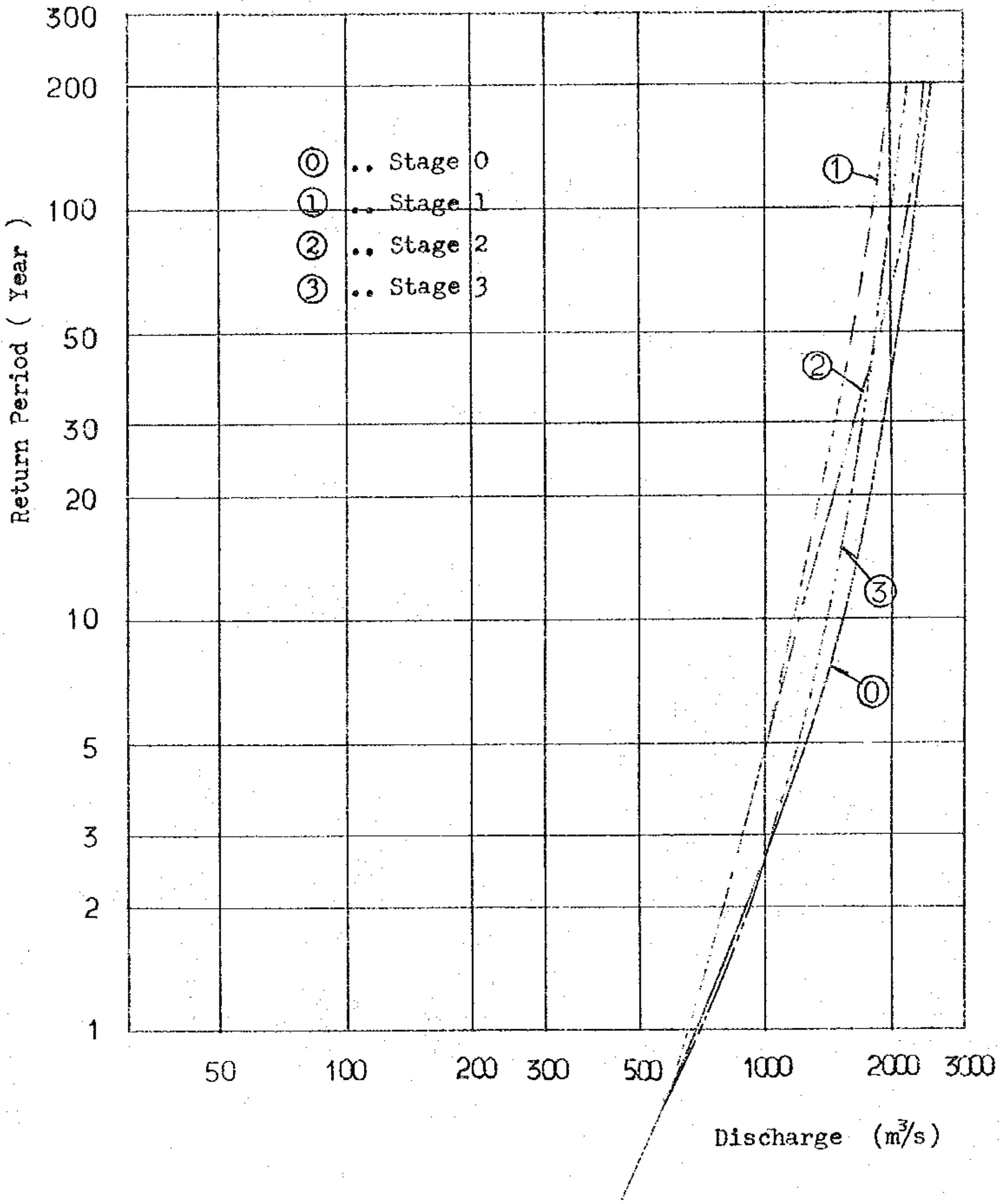


Table 3.4.1 Average Annual Damage under Existing Condition

(Surakarta Area)

Probability	Probability of Occurrence	Discharge (m^3/s)	Amount of Flood Damage $x10^6$ (US$)	Average Flood Damage $x10^6$ (US$)	Average Annual Flood Damage $x10^6$ (US$)	Accumulated Average Annual Flood Damage $x10^6$ (US$)	Unplantable Area Damage $x10^6$ (US$)	Total $x10^6$(US$)
	1.500	580	0.60	0.60	0.90	0.90	0.34	1.24
1/1		680	1.20					
	0.500			1.95	0.98	1.88	"	2.22
1/2		900	2.70					
	0.300			4.21	1.26	3.14	"	3.48
1/5		1,250	5.70					
	0.100			7.73	0.77	3.91	"	4.25
1/10		1,500	9.76					
	0.050			11.78	0.59	4.50	"	4.84
1/20		1,750	13.80					
	0.0167			15.05	0.25	4.75	"	5.09
1/30		1,900	16.30					
	0.0083			36.92	0.31	5.06	"	5.40
1/40		2,000	57.53					
	0.0083			57.73	0.48	5.54	"	5.88
1/60		2,130	57.92					
	0.0067			58.14	0.39	5.93	"	6.27
1/100		2,300	58.35					
	0.0050			58.65	0.29	6.22	"	6.56
1/200		2,500	58.94					

Table 3.4.2 Average Annual Damage under Stage 1

(Surakarta Area)

Probability	Probability of Occurrence	Discharge (m^3/s)	Amount of Flood Damage $x10^6$ (US$)	Average Flood Damage $x10^6$ (US$)	Average Annual Flood Damage $x10^6$ (US$)	Accumulated Average Annual Flood Damage $x10^6$ (US$)	Unplantable Area Damage $x10^6$ (US$)	Total $x10^6$(US$)
	1.500	580	0.6	0.60	0.90	0.90	0.34	1.24
1/1		680	1.2					
	0.500			1.60	0.80	1.70	"	2.04
1/2		800	2.0					
	0.300			2.85	0.86	2.56	"	2.90
1/5		1,000	3.7					
	0.100			4.70	0.47	3.03	"	3.37
1/10		1,170	5.7					
	0.050			6.75	0.34	3.37	"	3.71
1/20		1,360	7.8					
	0.0167			9.90	0.17	3.54	"	3.88
1/30		1,650	12.0					
	0.0083			13.13	0.11	3.65	"	3.99
1/40		1,800	14.25					
	0.0083			35.89	0.30	3.95	"	4.29
1/60		2,000	57.53					
	0.0067			57.80	0.39	4.34	"	4.68
1/100		2,200	58.06					
	0.0050			58.35	0.29	4.63	"	4.97
1/200		2,400	58.64					

Table 3.4.3 Average Annual Damage under Stage 2

(Surakarta Area)

Probability	Probability of Occurrence	Discharge (m^3/s)	Amount of Flood Damage $x10^6$ (US$)	Average Flood Damage $x10^6$ (US$)	Average Annual Flood Damage $x10^6$ (US$)	Accumulated Average Annual Flood Damage $x10^6$ (US$)	Unplantable Area Damage $x10^6$ (US$)	Total $x10^6$(US$)
	1.50	580	0.60	0.60	0.90	0.90	0.34	1.24
1/1		680	1.20					
	0.50			1.60	0.80	1.70	"	2.04
1/2		800	2.00					
	0.30			2.85	0.86	2.56	"	2.90
1/5		1,000	3.70					
	0.10			4.58	0.46	3.02	"	3.36
1/10		1,170	5.45					
	0.05			6.55	0.33	3.35	"	3.69
1/20		1,350	7.65					
	0.0167			8.48	0.14	3.49	"	3.83
1/30		1,470	9.30					
	0.0083			9.85	0.08	3.57	"	3.91
1/40		1,550	10.40					
	0.0083			11.33	0.09	3.66	"	4.00
1/60		1,670	12.25					
	0.0067			13.25	0.09	3.75	"	4.09
1/100		1,800	14.25					
	0.0050			35.89	0.18	3.93	"	4.27
1/200		2,000						

Table 3.4.4 Average Annual Damage under Stage 3

(Surakarta Area)

Probability	Probability of Occurrence	Discharge (m^3/s)	Amount of Flood Damage $x10^6$ (US$)	Average Flood Damage $x10^6$ (US$)	Average Annual Flood Damage $x10^6$ (US$)	Accumulated Average Annual Flood Damage $x10^6$ (US$)	Unplantable Area Damage $x10^6$ (US$)	Total $x10^6$(US$)
	1.500							
1/1								
	0.500							
1/2								
	0.300							
1/5								
	0.100							
1/10								
	0.050							
1/20								
	0.0167							
1/30								
	0.0083							
1/40								
	0.0083							
1/60								
	0.0067	2,000	0.00					
1/100				28.90	0.15	0.15	0.00	0.15
	0.0050	2,200	57.80					
1/200								

Table 3.4.5 Average Annual Damage in Landside-Water Area

(Surakarta Area)

Probability	Probability of Occurrence	Discharge (m^3/s)	Amount of Flood Damage x10^6 (US$)	Average Flood Damage x10^6 (US$)	Average Annual Flood Damage x10^6 (US$)	Accumulated Average Annual Flood Damage x10^6 (US$)	Unplantable Area Damage x10^6 (US$)	Total x10^6 (US$)
	1.500	580	0.142	0.142	0.213	0.213	0.00	0.217
1/1		680	0.369					
	0.500			0.469	0.234	0.447	0.00	0.447
1/2		900	0.568					
	0.300			0.698	0.209	0.656	0.00	0.656
1/5		1,250	0.828					
	0.100			0.936	0.094	0.750	0.00	0.750
1/10		1,500	1.043					
	0.050			1.113	0.055	0.805	0.00	0.805
1/20		1,750	1.183					
	0.0167			1.229	0.021	0.826	0.00	0.826
1/30		1,900	1.274					
	0.0083			1.312	0.011	0.837	0.00	0.837
1/40		2,000	1.349					
	0.0083			1.397	0.012	0.849	0.00	0.849
1/60		2,130	1.445					
	0.0067			1.505	0.010	0.861	0.00	0.861
1/100		2,300	1.565					
	0.0050			1.656	0.0083	0.869	0.00	0.869
1/200		2,500	1.745					

Table 3.4.6 Average Annual Damage under Existing Condition

(Sragen Left Bank Side Area)

Probability	Probability of Occurrence	Discharge (m^3/s)	Amount of Flood Damage x 10^6 US$	Average Flood Damage x 10^6 US$	Average Annual Flood Damage x 10^6 US$	Accumulated Average Annual Flood Damage x 10^6 US$
	1.500	580	0.37	0.3700	0.555	0.555
1/1		680	0.57			
	0.500			0.735	0.368	0.923
1/2		900	0.90			
	0.300			1.100	0.330	1.253
1/5		1,250	1.30			
	0.100			1.405	0.141	1.394
1/10		1,500	1.51			
	0.050			1.715	0.086	1.480
1/20		1,750	1.92			
	0.0167			2.030	0.034	1.514
1/30		1,900	2.14			
	0.0083			2.220	0.018	1.532
1/40		2,000	2.30			
	0.0083			2.375	0.020	1.552
1/60		2,130	2.45			
	0.0067			2.565	0.017	1.569
1/100		2,300	2.68			
	0.0050			2.800	0.014	1.583
1/200		2,500	2.92			

Table 3.4.7 Average Annual Damage under Existing Condition

(Sragen Right Bank Side Area)

Probability	Probability Occurrence	Discharge (m^3/s)	Amount of Flood Damage $\times 10^6$ US$	Average Flood Damage $\times 10^6$ US$	Average Annual Flood Damage $\times 10^6$ US$	Accumulated Average Annual Flood Damage $\times 10^6$ US$
	1.500	580	0.44	0.440	0.660	0.660
1/1		680	0.66			
	0.500			0.900	0.450	1.110
1/2		900	1.14			
	0.300			1.450	0.435	1.545
1/5		1,250	1.76			
	0.100			1.970	0.197	1.742
1/10		1,500	2.18			
	0.050			2.305	0.115	1.857
1/20		1,750	2.43			
	0.0167			2.505	0.042	1.899
1/30		1,900	2.58			
	0.0083			2.635	0.022	1.921
1/40		2,000	2.69			
	0.0083			2.745	0.023	1.944
1/60		2,130	2.80			
	0.0067			2.890	0.019	1.963
1/100		2,300	2.98			
	0.0050			3.075	0.015	1.978
1/200		2,500	3.17			

Table 3.4.8 Average Annual Damage after Completion of the Project

(Sragen Right Bank Side Area)

Probability	Probability Occurrence	Discharge (m^3/s)	Amount of Flood Damage x 10^6 US$	Average Flood Damage x10^6 US$	Average Annual Flood Damage x 10^6 US$	Accumulated Average Annual Flood Damage x 10^6 US$
	1.500	580	0.60	0.600	0.900	0.900
1/1		680	0.87			
	0.500			1.185	0.593	1.493
1/2		900	1.50			
	0.300			2.000	0.600	2.093
1/5		1,250	2.50			
	0.100			2.835	0.284	2.377
1/10		1,500	3.17			
	0.050			3.345	0.167	2.544
1/20		1,750	3.52			
	0.0167			3.615	0.060	2.604
1/30		1,900	3.71			
	0.0083			3.760	0.031	2.635
1/40		2,000	3.81			
	0.0083			3.905	0.032	2.667
1/60		2,130	4.00			
	0.0067			4.100	0.027	2.694
1/100		2,300	4.20			
	0.0050			4.315	0.022	2.716
1/200		2,500	4.43			

Note: Left Bank Area is the Out-side Area of Project.

Table 3.4.9 Average Annual Damage under Existing Condition

(Sragen (Left + Right) Bank Area)

Probability	Probability Occurrence	Discharge (m^3/s)	Amount of Flood Damage x 10^6 US$	Average Flood Damage x 10^6 US$	Average Annual Flood Damage x 10^6 US$	Accumulated Average Annual Flood Damage x 10^6 US$
	1.500	580	0.81	0.81	1.215	1.215
1/1		680	1.23			
	0.500			1.635	0.818	2.033
1/2		900	2.04			
	0.300			2.550	0.765	2.798
1/5		1,250	3.06			
	0.100			3.375	0.338	3.136
1/10		1,500	3.69			
	0.050			4.020	0.201	3.337
1/20		1,750	4.35			
	0.0167			4.535	0.076	3.413
1/30		1,900	4.72			
	0.0083			4.855	0.040	3.453
1/40		2,000	4.99			
	0.0083			5.120	0.043	3.496
1/60		2,130	5.25			
	0.0067			5.455	0.036	3.532
1/100		2,300	5.76			
	0.0050			5.875	0.029	3.561
1/200		2,500	6.09			

Table 3.4.10 Average Annual Damage after the Completion of the Proposed River Improvement

(Sragen (Left + Right) Bank Side Area)

Probability	Probability of Occurrence	Discharge (m^3/s)	Amount of Flood Damage x10^6 (US$)	Average Flood Damage x10^6 (US$)	Average Annual Flood Damage x10^6 (US$)	Accumulated Average Annual Flood Damage x10^6 (US$)
	1.500	585	0.84	0.84	1.260	1.260
1/1		700	1.33			
	0.500			1.69	0.845	2.105
1/2		900	2.04			
	0.300			2.48	0.744	2.849
1/5		1,200	2.92			
	0.100			3.19	0.319	3.168
1/10		1,400	3.45			
	0.050			3.71	0.186	3.354
1/20		1,600	3.97			
	0.0167			4.10	0.068	3.422
1/30		1,700	4.22			
	0.0083			4.35	0.036	3.458
1/40		1,800	4.47			
	0.0083			4.60	0.038	3.496
1/60		1,900	4.72			
	0.0067			4.86	0.033	3.529
1/100		2,000	4.99			
	0.0050			5.21	0.026	3.555
1/200		2,200	5.43			

3.5 RIVER IMPROVEMENT DESIGN

3.5.1 Cross Section

The standard section is a compesite cross section as shown in Fig. 3.2.10. The low-water channel is roughly equivalent to the width of the existing river (80 - 100 m), and the channel section is planned to be capable of withstanding the flood which occurs once every two years or so. ($Q = 900\ m^3/s$ at Jurug site).

The major bed width at each side of the repair section on the upstream side is made rather 62 m, and at 100 m on the downstream side.

The reason is that allowance is made for new unpredictable meandering due to short-cutting of the channel as well as for the possibility of increasing the width of the low-water channel to cope with the possible increase of the flood flow rate.

The slope of the low-water channel is designed to be moderate at 1:3 for increasing its safety against erosion by the stream.

Banking is done mainly by using the soil obtainable from the excavation of the low-water channel short-cut, the low-water channel, and the major bed. Since the soil dug from the river bed is sandy, it is to be used for the construction of the maintenance road inside the bank. The total volume of excavation is approx. 6,800,000 m^3, while the soil to be used for banking is approx. 6,000,000 m and an allowance of approx. 13% is considered sufficient, the balance being nearly ideal.

Most of the banking material is silty sand or silty clay, not considered very good as banking material.
The average values obtained through the soil test are as follows.

Optimum water content ratio	$W \doteqdot 30\%$
Wet density	$t \doteqdot 1.75\ t/m^3$
Maximum drying density	$d \doteqdot 1.35\ t/m^3$
Coefficient of permeability	$k \doteqdot 1 \times 10^{-7}$ cm/sec.
Internal friction angle	$\phi \doteqdot 12^{o}$
Cohesion	$C \doteqdot 3.7\ t/m^2$

The banking material is high in fine grain content and the degree of compaction is low, hence it is weak to stream and rain water being subject to erosion. The permeability coefficient is low and the cut-off effect is high, but the surface of slope is subject to cracking on drying.

For the safety of the levee, therefore, the cross sectional area is designed to be increased as far as the amount of the banking soil is balanced with that of the cut-out soil. The outside slope of levee is designed to be 1;3.0, the crest width is 7.0 m and the inside of slope of levee 1:2.0, and the width of the maintenance road 6.0 m.

3.5.2 Study of the Stability of the Levee

The stability calculation is made for the standard section of a levee $H \doteqdot 5.00$ m. The stability is determined in terms of safety factor on the basis of the rotary moment (M) caused by the top soil and the resistance moment (Mr) caused by the adhesive force and frictional force acting at the sliding face under assumption of a rould sliding surface.

$$\frac{Mr}{M} = Fs \text{ (safety factor)}$$

As to soil conditions, calculation is made on the basis of follwing values obtained as rough average at the soil test.

wet density	$t = 1.75\ t/m^3$
internal friction angle	$\phi = 12^o$
adhesive force	$c = 3.7\ t/m^2$

The calculation is made with the aid of a computer.

1) Calculation conditions

Calculation is to be made for the following cases.

a. Immediately after completion of the embankment

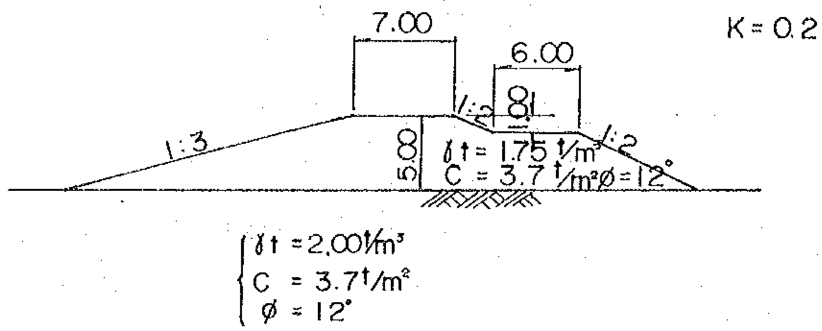
b. When the water level is H.W.L.

c. When the water level has suddenly dropped from H.W.L. to L.W.L.

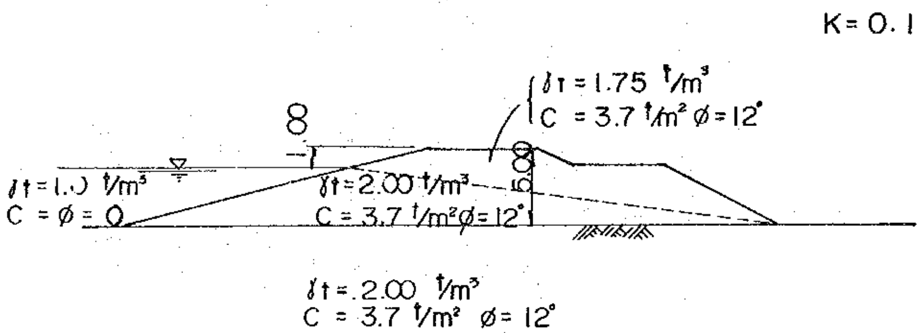
As to "a" and "b", calculation is to be made where damage has been caused to the inside and outside slope of the embankment, while as to "c", calculation is to be made where damage has been caused to the outside slope of the levee.

In case of "a", it was taken as $k = 0.2$ and in case of "b" and "c" as $k = 0.1$. The sliding face is in contact with the foundation subgrade. The soil conditions for each case are as illustrated below.

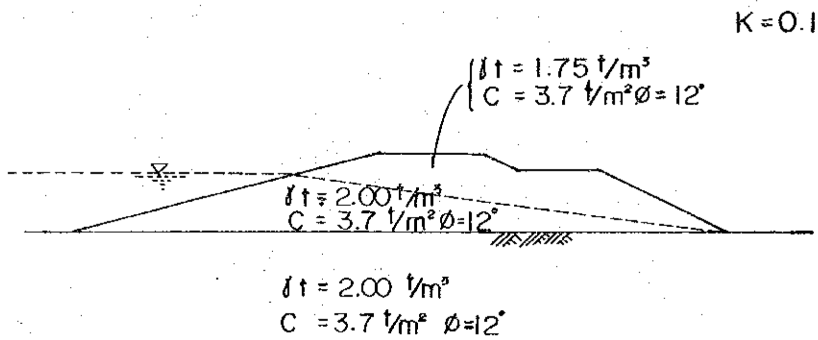
a. Immediately after completion of the embankment



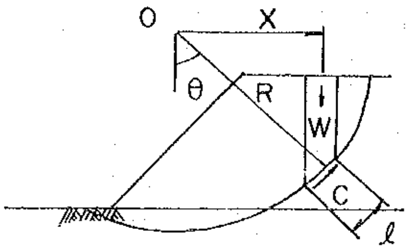
b. When the water level is H.W.L.



c. When the water level has dropped from H.W.L. to L.W.L.



2) Calculation formula



c: adhesive force of soil on the assumed sliding face

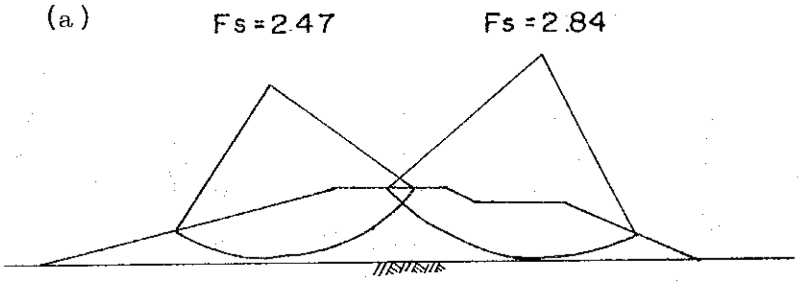
ϕ: internal frictional force of soil on the assumed sliding face

W: weight of fine-particled soil

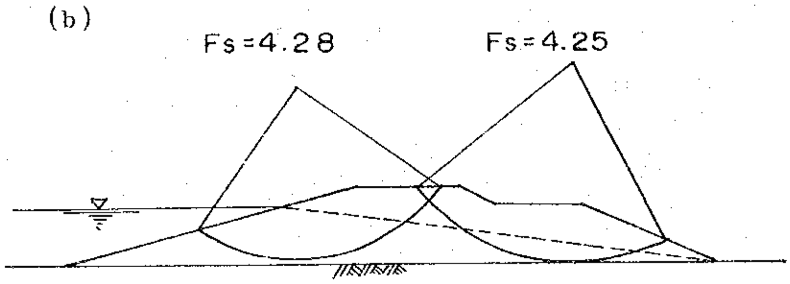
Fs: safety factor

3) Result

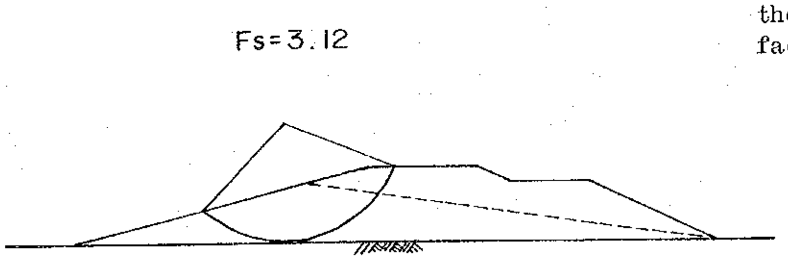
(a)



(b)



(c)



* Fs indicates the minimum value, and the arc the sliding face.

These results show that in every case the safety factor is ensured.

As to (1) outside face of slope of the levee immediately after completion, for which the safety factor is lowest, given below is the result of the study made on the values c and ϕ' of the banking soil varying within given ranges.

Table 3.5.1 Relationship Fs and ϕ'' in Parameter C

C	ϕ''	Fs	C	ϕ''	Fs	C	ϕ''	Fs
0	5	0.05	1.0	20	0.89	3.0	15	2.13
0	10	0.10	2.0	5	1.33	3.0	20	2.14
0	15	0.16	2.0	10	1.40	3.7	5	2.41
0	20	0.21	2.0	15	1.47	3.7	10	2.48
1.0	5	0.69	2.0	20	1.54	3.7	15	2.55
1.0	10	0.75	3.0	5	2.11	3.7	20	2.62
1.0	15	0.82	3.0	10	2.12			

a) Using the above table, the relationship between Fs and ϕ has been made into a graph with C as parameter, and the result shows that for the condition of Fs 1.0 is satisfied on the line which connects C = 1.5, $\phi' = 5^{o}$ with C = 1.2, $\phi = 20^{o}$ in Fig. 3.5.1.
The result of the calculation is shown in Figs. 3.5.2 to 3.5.4.

3.5.3. Bank Protection

The river to be improved is a primitive river extremely meandering and almost untouched artificially save for a section of stone pitching and several pile groins. Both levees of the river are bluff at almost right angle due to erosion almost everywhere. In the flow attaking place there is noted a marked progress of erosion, its extent being 2-3 m/year at some places.

Under the present river improvement work the repair is made to correct meandering by short-cutting, and continuous levees were constructed on both banks.

The condition of the river changes from year to year due to the severe meandering, and it is almost impossible to predict such meandering. The possibility of migration of flow attaking point, widening of the low-water channel to cope with the anticipated increase in future of the flood discharge, etc. are taken into account, and it is considered advisable not to make the low-water channel revetment so solid.

Under the current river improvement work, the following measures for the protection of the river are being taken.

a. Sodding

The bank protection in the embanked sections is by sodding as flood covering the major bed is recorded once every 2 years or so.

b. Stone pitching revetment

Stone pitching is for directly protecting the banks of the low-water channel from scour and collapse and also for regulating the flow direction of the stream, and is to be constructed at the following places. (See Fig. 3.2.6 and DWG. No. RW.55)

i) The section at where the tributary joins the main river and the section on the opposite bank of tributary mouth where supposed to be directly subjected to jointed stream.

ii) Up-and downstream sides of bridge.

c. Wire basket revetment

Compared with stone pitching this is flexible and is easy to repair, and is to be constructed at the following places. (See Fig. 3.2.6 and DWG. NO. 56)

i) As transition sections on the up- and down-stream sides of a stone pitching over a length of 5.0 - 10.0 m.

ii) On portions of the low-water channel crossing the old meandering river course. (Short-cut portions)

d. Groin

Groins are the permeable groins and the impermeable groins.

The permeable groin is for lowering the flow velocity and inducing sedimentation of silt and sand carried by the stream. Hence the resistance to stream is smaller than that of impermeable groin but safer.

To the impermeable groins belong mattress groin, crib groin, etc., which have a large resistance to the stream. It is, therefore, difficult to maintain such a groin which is subject to scouring on the up- and downstream sides or in front of the groin.

Here a pile groin which is a kind of permeable groin is used. There are empirical formula for determination of the proper length and interval of groins. Actually it is ideal to determine the proper position, length and interval of such groins on the basis of the observed result of hydraulic model test.

i) Flow attaking place on bends of a low-water channel

ii) Where in a linear section of a low-water channel the low-water channel width is large and a sand bar is necessary.

The length of the groin is made ℓ = 20 m.
The relationship between ℓ and interval of pile L is shown by the following empirical formula.

concaved bank section: L = (1.5 - 2.0) m

convexed bank section: L = 3 m

Linear section: L = (2.5 - 3.0) m

The groin interval along the channel chosen for the present section is;

concaved bank section: L = 1.5 ℓ = 30 m

convexed bank section: L = 3.0 ℓ = 60 m

Liner section: L = 2.0 ℓ = 40 m

There are two alternative types of groins, namely, wooden pile groin and concrete pile groins, which are arranged alternately for lowering the working cost and facilitating the future maintenance. Also wire cylinders are used for foot protection of groins. (See DWG. No. RW. 52, 53, 54)

3.5.4. Landside Water Drainage Facilities

The Sala river, which is to be improved over a reach of approx. 33 km, is going to have continuous levees upon completion of the said work.

It is, therefore, necessary to provide drainage sluiceways and drainage channels for drainage of landside basin. As to the calculation of discharge, the whole landside basins are to be divided into 32 areas and the discharge from the individual areas are to be determined according to the "Indonesian Standards of Drainage".

The discharge in the said Standards are given in the form of a diagram (Fig. 3-2-25) showing the relationship between drainage area and specific discharge. The result of the drainage discharge calculation is as shown in Table 3.5.3.

The sluiceway is designed for 6 typical discharge rates, namely, Q = 8.0 m^3/s, Q = 12.0 m^3/s, Q = 16.0 m^3/s, Q = 20.0 m^3/s, Q = 28.0 m^3/s and Q = 40.0 m^3/s, the construction cost .vs. discharge capacity graph is prepared and the amounts of work required for the respective discharge rates are estimated.

The result is given in Fig. 3.5.5. and Table 3.5.3.

3.5.5. Groundsill

The so-called "Groundsill" will be constructed to traverse the channel for the purpose of prevention of erosion of the river bed. Most of the sediment which has been sent down to the Nguter railway bridge at present will be deposited in the Wonogiri reservoir and the Colo afterbay after the construction of the Wonogiri dam and the Colo weir. In anticipation of the lowering of the low water channel bed, the Groundsills will be constructed 20 m downstream of the Nguter railway and road bridges to protect their foundations. The locations of the Groundsills are shown in Fig. 3.2.26.

"The Groundsill II" will be constructed experimentally at three (3) places for the purpose of fixing the course of the low water channel. Structure of the above is shown in DWG. No. RW-55.

3.5.6. Others

a) Boat station

At present, there are about 26 boat stations on the river. No special facilities are being provided for boat stations and some simplified bamboo wharves are found here and there. This river improvement plan does not include the construction of wharves, and the existing boat stations are to be reinforced with wire basket revetment at the point of sluiceway. It would be advisable for the people who live there to construct simple bamboo wharves by themselves in order not to exert any bad influence on the revetment.

b) Bathing place for buffalo

Various places along the river are being used as bathing places for buffaloes. It is feared that the constructed levee may be damaged as the buffaloes walk on it at random.

Therefore, in this river improvement plan, step ways for sluiceway administration as well as the passage for buffaloes will be constructed. The step ways will be widened to 3.0 m. 35 sluiceways are required. There are not many bathing places for buffaloes, but it will be possible to provide sufficient number of bathing places by using other tributaries and drainage channel inside the levee.

3.5.7 Measurement and Quantities of the major works:

1.	Excavation	6,814,000 m^3
2.	Banking	5,998,000 m^3
3.	Spoil Bank	703,000 m^3
4.	Bank Protection	
	Stone Pitching	125,000 m^2
	Wire Basket	103,300 m^2
	Groin (Wooden)	259 Place
	Groin (Concrete)	136 Place
	Sodding	1,819,000 m^3
5.	Groundsill	5 Place
6.	Sluiceway	32 Place
7.	Intercepting Drainage Channel	254,000 m^3
8.	Road for Construction Work	10,000 m
9.	Bridge (Nguter)	1 Place

3.6 RIVER IMPROVEMENT WORK

3.6.1 Survey of Present Circumstances

a) Machinery and equipment

Machinery and equipment in possession are as shown in Table 3.6.1.

Table 3.6.1 Machineries and Equipment in Possession

No.	Item	Specification		Q'ty
1	Drag-line	Linkbelt LS78J	(6 ton)	2
2	Bulldozer	CAT D3	(6 ton)	2
3	Bulldozer	CAT D6	(11 ton)	1
4	Bulldozer	Komatsu D60A	(15 ton)	1
5	Dozer-shovel	CAT 931	(0.7 m^3)	3
6	Excavator	Yutani-Poclain TY45	(0.3 m^3)	1
7	Excavator	Yutani-Poclain TC50	(0.35 m^3)	1
8	Air Compressor	Schramm 250 (ft/m^3)		2
9	Sheep's-foot Roller			2
10	Tamping Rammer			7
11	Vibration Roller	2 Drum	1 ton	1
12	Dump Track	3.5 ton		5
13	Dump Track	5.0 ton		8
14	Water Pump	6" ø		1
15	Water Pump	4" ø		1
16	Pile Driver			1

Fig.3.5.1 The relationship Fs and ø with Parameter C

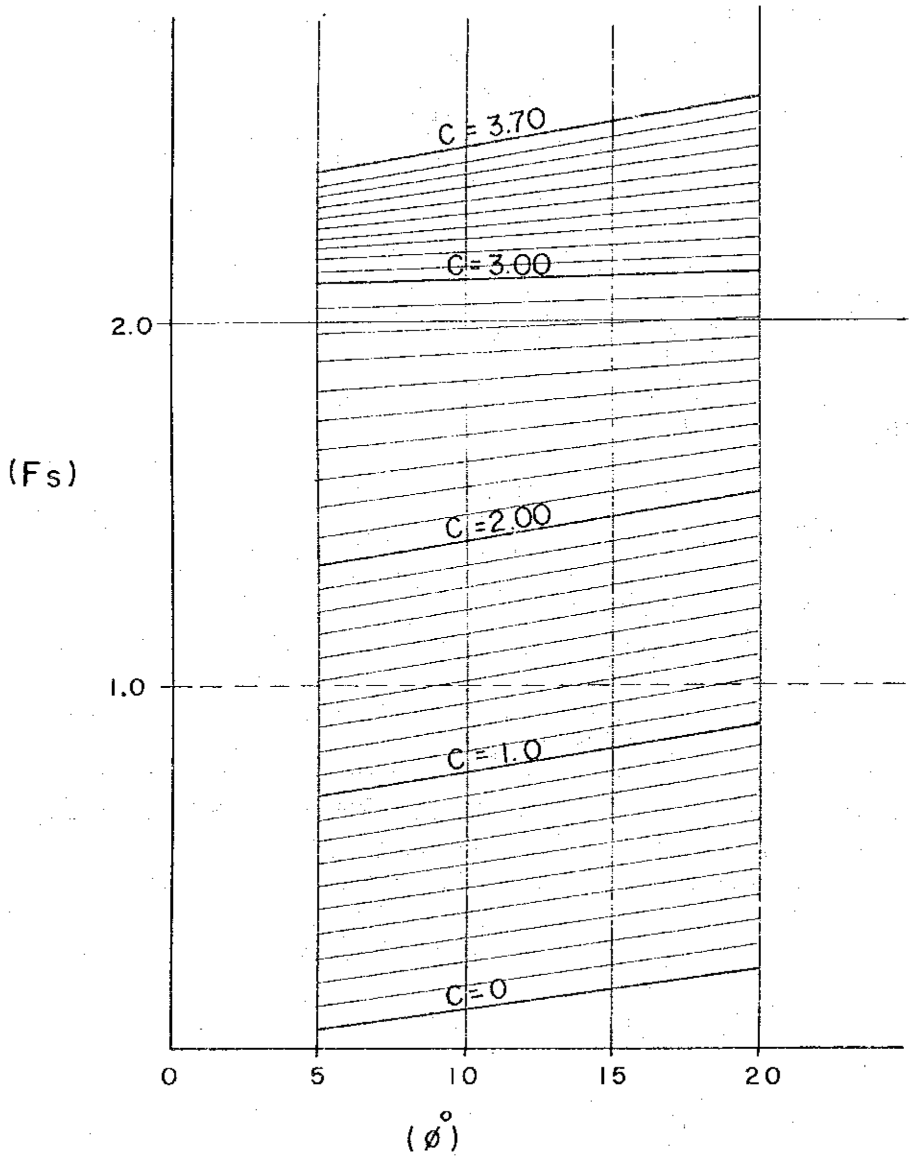
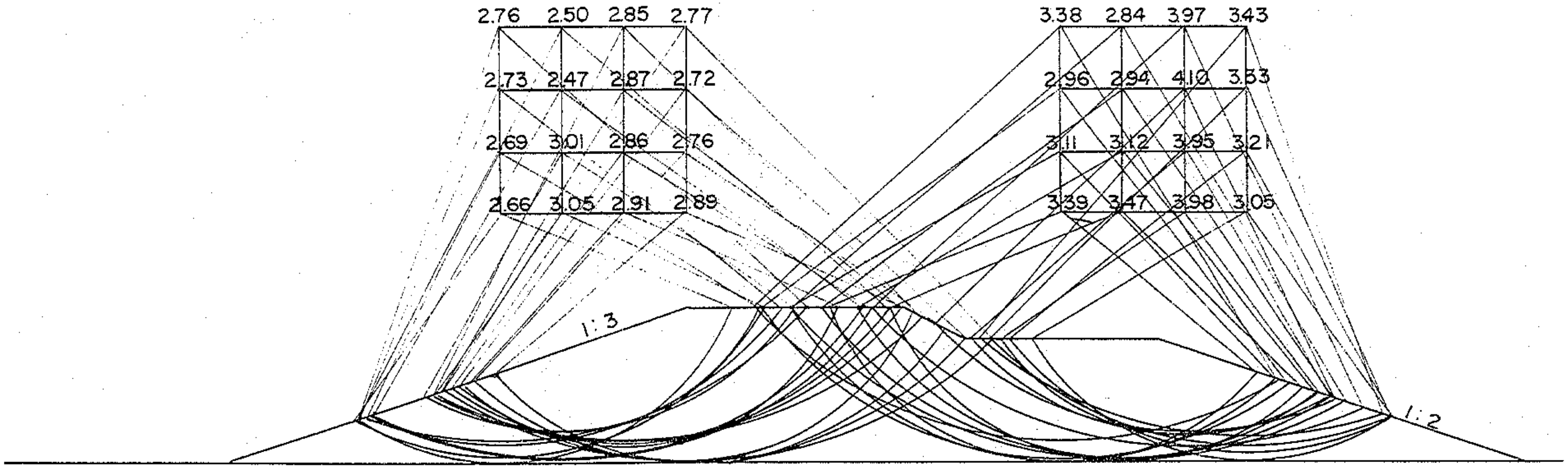
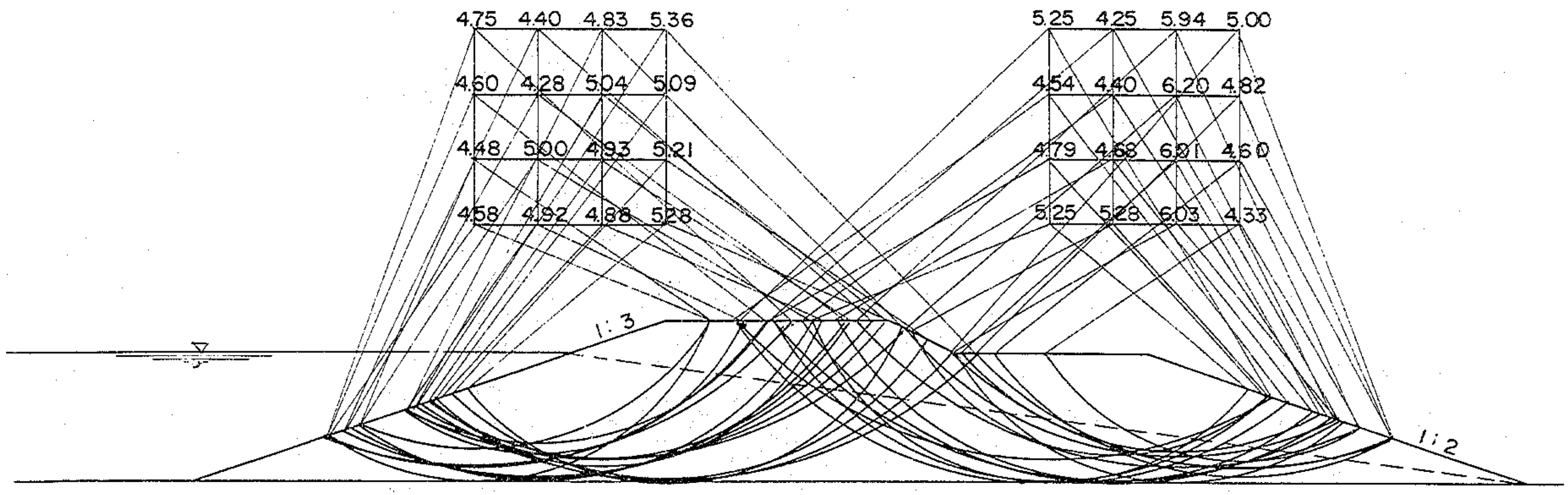


Fig. 3.5.2 Safety factor of Bank immediately after the bank construction



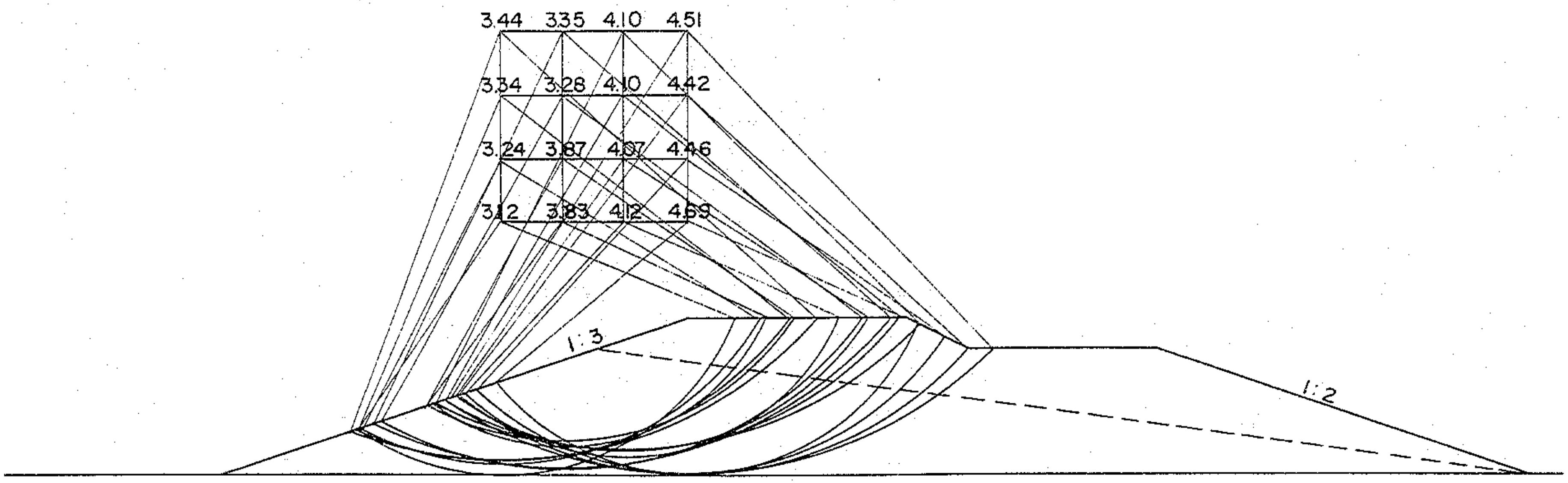
S = 1/200

Fig. 3.5.3 Safety factor of Bank in case of the H.W.L stage condition



S = 1/200

Fig. 354 Safety factor of Bank in case of Sudden stage down from H.W.L to L.W.L.



S = 1/200

Fig. 3.5.5 Construction Cost ~ Dischaig Capacity
Of a sluiceway

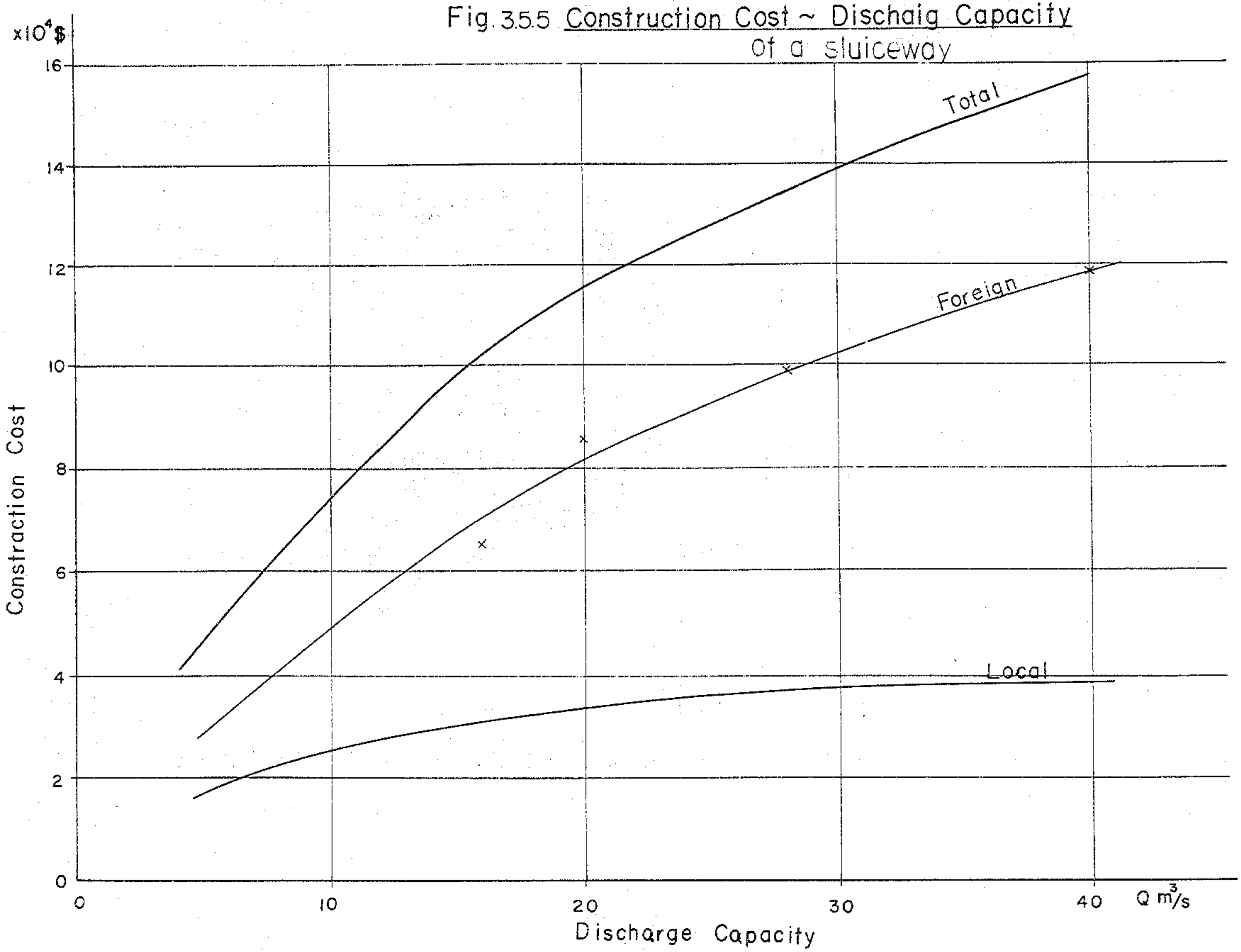


Table 3.5.2 Amount of the Bank Protection No. 1

Location No.	Stone Piching (m)	Wire Basket (m)	Groin (Wooden) (point)	Groin (Conrete) (point)
No. 1- 20 - No. 1+ 20	L. 20.0	L. 20.0		
	R. 20.0	R. 20.0		
No. 2+ 50 - No. 3+ 60			R. 3	R. 4
No. 3+ 80 - No. 3+240		L.160.0		
No. 4+110 - No. 4+200		L. 90.0		
No. 4+120 - No. 4+250		R.130.0		
No. 6+ 60 - No. 6+180		R.120.0		
No. 6+120 - No. 7+100		L.220.0		
No. 9+150 - No.10+ 60			L. 4	L. 5
No. 9+250 - No.10+ 10			R. 2	R. 3
No.10+250 - No.11+120			R. 2	R. 3
No.11+140 - No.11+410			L. 5	L. 5
No.12+270 - No.12+310	L. 20.0	L. 20.0		
	R. 20.0	R. 20.0		
No.13+150 - No.14+140	R.390.0	R. 20.0		
No.13+220 - No.14+160	L.420.0	L. 40.0		
No.15+160 - No.16+ 60		L.150.0		
No.16+200 - No.17+ 40			L. 4	L. 5
No.17+220 - No.17+380		L.160.0		
No.18+ 30 - No.19+110	R.510.0	R.250.0		
No.18+ 90 - No.19+ 80	L.370.0	L. 20.0		
No.19+200 - No.20+180			R. 3	R. 3
No.20+200 - No.21+ 20		R. 90.0		
No.21+200 - No.22+ 60		R. 90.0		
No.23+ 0 - No.24+ 10			L. 6	L. 7
No.23+110 - No.23+170		R. 60.0		
No.24+200 - No.25+ 60			R. 1	R. 2
No.25+200 - No.25+240	L. 20.0	L. 20.0		
	R. 20.0	R. 20.0		
No.27+290 - No.28+150			L. 5	L. 5
No.28+230 - No.29+130		R.160.0		
No.29+230 - No.30+240			R. 4	R. 5
No.30+310 - No.31+130		R.300.0		
No.31+240 - No.32+180			L. 3	L. 4
No.32+250 - No.33+160		R.260.0		
No.32+340 - No.33+290	L.320.0	L. 40.0		

No. 2

Location No.	Stone Piching (m)	Wire Basket (m)	Groin (Wooden) (Point)	Groin (Concrete) (Point)
No.34+130 - No.34+370			R. 2	R. 3
No.37+130 - No.38+230	R.170.0	R.250.0		
No.37+150 - No.38+250	L.570.0	L. 40.0		
No.39+230 - No.39+310		R. 80.0		
No.40+ 20 - No.40+130		R.110.0		
No.40+380 - No.41+180			R. 3	R. 3
No.41+380 - No.42+ 30		R.170.0		
No.43+ 0 - No.43+120		R.120.0		
No.44+ 0 - No.44+290	L.360	L. 40.0		
No.44+ 0 - No.44+270	R.300	R.120.0		
Sub Total	3,530 m	3,410 m		
Area	66,400 m^2	64,100 m^2	47 Point	57 Point
No.44+380 - No.45+180	R.230.0	100.0		
No.44+410 - No.45+270	L.440.0	40.0		
No.46+ 70 - No.47+ 0		L.170.0		
No.47+330 - No.48+ 80			L. 2	L. 3
No.47+330 - No.48+ 80			R. 2	R. 3
No.48+ 90 - N-.48+520	L.730.0	L. 40.0		
No.48+ 90 - No.49	R.640.0	30.0		
Sub Total	2,040.0 m	380 m		
Area	35,500 m^2	6,600 m^2	4 Point	6 Point
No.49+130 - No.49+260		L.130.0		
No.49+330 - No.49+420		R. 90.0		
No.49+330 - No.49+690			L. 3	L. 4
No.49+500 - No.50+120			R. 2	R. 3
No.50+170 - No.50+320		R.150.0		
No.51+230 - No.52+210			L. 6	L. 6
No.52+490 - No.53+180			R. 2	R. 3
No.54 - No.55+310		R.590.0		
No.54+160 - No.55+120			L. 4	L. 5
No.55+510 - No.56+120			L. 4	L. 4
No.56+380 - No.57+120			R. 2	R. 3
No.57+320 - No.58+180		R.260.0		

No. 3

Location No.	Stone Piching (m)	Wire Basket (m)	Groin (Wooden) (Point)	Groin (Concrete) (Point)
No.59+160 - No.59+460			R. 3	R. 3
No.60+130 - No.61+180			L. 8	L. 8
No.61+220 - No.62+180		R.230.0		
No.63+210 - No.64+110		R.230.0		
No.64+180 - No.66	L.490.0	20.0		
No.64+190 - No.66+ 80	R.740.0	40.0		
No.67+310 - No.68+210			R. 3	R. 4
No.70+160 - No.78+ 30			L. 75	
No.71+ 0 - No.78+ 0			R. 70	
No.78+300 - No.78+520		L.220.0		
No.79+190 - No.80+ 30		L.300.0		
No.80+190 - No.82+ 10			R. 5	R. 6
No.82+560 - No.83+180			L. 6	L. 7
No.84+ 50 - No.84+290			R. 2	R. 3
No.85+170 - No.86+280			R. 5	R. 5
No.86+360 - No.87+ 70		R.130.0		
No.88+310 - No.88+670			L. 6	L. 7
No.89+ 30 - No.89+210			R. 2	R. 2
Sub Total	1,230 m	2,390 m	208 Point	73 Point
Area	17,500 m^2	33,900 m^2		
Total	6,800 m	6,180 m		
	125,300 m^2	103,300 m^2	259 Point	136 Point

Table 3.5.3 Discharge of Sluiceway

No.	Location		Drainage Area (km^2)	Discharge (m^3/s)	Length of Drainage Channel (m)
1	Bengawan Sala	.L.No. 7	3.7	9.3	0
2	"	.L.No.10	16.0	14.4	1,500
3	"	.R.No. 2	7.0	11.2	900
4	"	.R.No. 7	10.0	12.0	1,300
5	"	.R.No.13	9.7	11.6	3,400
6	K. Wingko	.R.No. 1	67.5	37.1	2,200
7	K. Samin	.R.No. 1	6.8	10.9	500
8	"	.L.No. 4	19.7	15.8	1,200
9	"	.L.No. 3	23.9	17.9	0
10	Bengawan Sala	.R.No.21	2.8	8.5	3,900
11	"	.L.No.24	8.0	11.6	1,300
12	"	.L.No.29	6.4	10.9	1,700
13	K. Kembangan	.R.No. 0	3.0	9.0	1,300
14	Bengawan Sala	.R.No.34	8.6	11.6	4,400
15	K. Brambang	.R.No. 1	9.0	12.2	0
16	"	.R.No. 0	1.5	6.1	1,800
17	K. Buntungan	.R.No. 1	2.0	7.4	5,500
18	Bengawan Sala	.R.No.48	14.2	14.2	3,500
19	K. Pusur	.R.No. 0	3.2	9.3	1,700
20	K. Gawe	.R.No. 0	1.8	6.8	1,000
21	K. Dengkeng	.L.No. 5	2.7	8.6	1,000
22	"	.R.No. 1	4.4	9.9	7,700
23	Bengawan Sala	.R.No.56	4.7	9.9	4,000
24	K. Jlantah	.R.No. 1	2.6	8.3	0
25	"	.L.No. 2	10.0	12.0	2,500
26	"	.L.No. 0	15.7	14.1	3,100
27	Bengawan Sala	.L.No.63	2.7	8.4	3,700
28	"	.R.No.78	6.0	10.8	3,300
29	"	.L.No.76	0.6	4.2	1,100
30	"	.L.No.79	2.0	7.4	1,700
31	"	.L.No.83	45.0	27.0	2,600
32	"	.R.No.76	25.0	17.5	1,200

Table 3.5.4 Construction Cost of Each Sluiceway

No.	Drainage Area	Discharge (m^3/s)	Total Cost (US$)	F/C (US$)	L/C (US$)
1	3.7	9.3	70.000	46,000	24,000
2	16.0	14.4	95,000	65,000	30,000
3	7.0	11.2	80,000	54,000	26,000
4	10.0	12.0	84,000	56,000	28,000
5	9.7	11.6	82,000	55,000	27,000
6	67.5	37.1	152,000	114,000	38,000
7	6.8	10.9	78,000	52,000	26,000
8	19.7	15.8	101,000	70,000	31,000
9	23.9	17.9	109,000	76,000	33,000
10	2.8	8.5	65,000	43,000	22,000
11	8.0	11.6	82,000	55,000	27,000
12	6.4	10.9	78,000	52,000	26,000
13	3.0	9.0	68,000	45,000	23,000
14	8.6	11.6	82,000	55,000	27,000
15	9.0	12.2	85,000	57,000	28,000
16	1.5	6.1	52,000	33,000	19,000
17	2.0	7.4	60,000	39,000	21,000
18	14.2	14.2	94,000	64,000	30,000
19	3.2	9.3	70,000	46,000	24,000
20	1.8	6.8	56,000	36,000	20,000
21	2.7	8.6	66,000	43,000	23,000
22	4.4	9.9	73,000	48,000	25,000
23	4.7	9.9	73,000	48,000	25,000
24	2.6	8.3	64,000	42,000	22,000
25	10.0	12.0	84,000	56,000	28,000
26	15.7	14.1	94,000	64,000	30,000
27	2.7	8.4	65,000	43,000	22,000
28	6.0	10.8	78,000	54,000	24,000
29	0.6	4.2	41,000	26,000	15,000
30	2.0	7.4	60,000	39,000	21,000
31	45.0	27.0	132,000	96,000	36,000
32	25.0	17.5	107,000	75,000	32,000
Total			2,580,000	1,747,000	833,000
Average			80,625 $/Place	54,594 $/Place	26,031 $/Place

The river improvement plan is of a huge scale requiring earthwork of approx. 6,800,000 m3 and excavation work comprising approx. 6,000,000 m^3 of soil. The river improvement work can not be executed without having sufficient quantity of efficient machinery and equipment. More machinery and equipment should be supplied under this river improvement plan.

b) Material investigation

Materials available in Indonesia are as follows:

i) Stone

Stones for stone pitching, wire basket and wire cylinder will be supplied from the upper stream of the tributaries of the Sala river.

ii) Aggregates

Fine aggregates (sand) have been dug at the bed of the Sala river. Coarse aggregates (gravel) will be supplied from the upper reach of the tributaries. The estimated amount to be excavated daily is not sufficient, however, the amount of concrete to be used under this river improvement plan will not be large for each site, therefore there will be no shortage of aggregates.

iii) Wooden pile and board

Wooden piles and boards will be supplied from the area which will be submerged after the completion of the Wonogiri Dam. Coconut tree, teak tree, mahoui tree and other fruits tree are available in the adjacent area. Approx. 7,200 wooden piles will be used in total. Coconut trees available in the area will be approx. 165,000. Other timbers can be obtained in sufficient quantities. Lumbering is necessary for producing planks.

iv) Bamboo

Bamboo will be supplied from the abovementioned area, and there are plenty of bamboos near the job site.

v) Lawn or grass

Grasses for sodding are plentiful near the job site. The total area for sodding is approx. 1,820 x 10^3 m^2, land for cultivation of grasses for sodding may be required for constant supply of required quantity.

c) Investigation of present condition of the road

The length of the existing river for river improvement is about 55 km, and it will be about 33 km after the completion of the river improvement works. The roads which can be used in the construction of the project have been checked. New roads may have to be constructed and the existing roads reinforced because the roads are narrow and are in poor condition. Bridges over the channels are too old. The results of the investigation are as shown in DWG. No. RW-59 and DWG. No. RW-61.

3.6.2 Construction Work

a) Work order and examination of work measurement

The construction period estimated is 6 years. The construction work will be separated into 6 sections and the work will be carried out only during the dry season of May to October.

Major works of the river improvement will be "banking" and "excavation," and the work section will be separated to unify the quantity of banking. (See Fig. 3.6.1, Table 3.6.3 and Table 3.6.4.)

In principle, the work will be started with the excavation of the downstream, and No. 1 work section is set on the right bank side of K. Samin and the reach between No. 19 + 50 m -- No. 49 + 50 m of the right bank side of the main channel where the benefit from the levee is the largest. On the other hand, with the exception of the above, construction of the revetment, as the first annual work of the year, should be carried out on the immediate upstream of the confluence of K. Dengkeng and main river where erosion has progressed and the river is meandering considerably. For the other work sections, work will be done from the downstream in the order of II, III, IV, V, VI. The revetment and groin will be constructed upon completion of the excavation of the low-water channel, and the sluiceway will be constructed prior to the construction of the levee.

b) Selection and combination of the heavy equipment

i) Conditions

. Annual working days

6 months x 30 days = 180 days

. Annual work plan

Although work sections I to VI have been planned, work section IV which involves the largest volume of work has been used as the standard when making the plan.

. Annual work measurement

Excavation:	River bed	933,800 m^3
	Low water channel	339,100 m^3
	High water bed	97,600 m3
	Total	1,370,500 m^3
Banking:	{ Main stream levee	103,800 m^3
	{ Maintenance road	28,400 m^3
	{ Tributary levee	848,600 m^3
	{ Maintenance road	247,400 m^3
	Total	1,228,200 m^3
Other:	Stripping	145,400 m^3
Slope area of levee		416,900 m^3

ii) <u>Combination of the heavy equipment</u>

Heavy equipment usable are as follows:

Excavation: Bulldozer
Truck shovel

Loading: Tractor shovel
Drag shovel

Carry & transportation: Bulldozer (Short range)
Damp truck (Long distance)

Spreading roll-filling: Bulldozer

Slope compaction of levee: Vibration roller

iii) Calculation

Approx. 50% of river bed excavation (467,000 m^3) and high water bed excavation (97,600 m3) (564,000 m3 in total) will be carried out with the bulldozer. Remaining river bed excavation (466,800 m^3) and low-water channel excavation (339,100 m^3) (805,900 m^3 in total) will be carried out with the drug-shovel. Transportation of approx. 35% (410,000 m^3) of excavated soil will be carried out with the bulldozer, and the remaining soil (960,500 m^3) will be transported by the damp truck. Spread roll-filling with bulldozer. Excavated soil will be diverted to banking soil, stripping comprising 132,800 m^3 will be excavated with the bulldozer, and then transported and thrown away by damp truck.

. Daily job measurement

1) Bulldozer (excavation)

$$\frac{564{,}600\ m^3}{180\ \text{days}} = 3{,}140\ m^3/\text{day}$$

2) Drag shovel (excavation and loading)

$$\frac{805{,}900\ m^3}{180\ \text{days}} = 4{,}480\ m^3/\text{day}$$

3) Bulldozer (carry)

$$\frac{410{,}000\ m^3}{180\ \text{days}} = 2{,}280\ m^3/\text{day}$$

4) Tractor shovel (loading, soil q'ty, bed & lower channel)

$$\frac{145{,}400\ m^3}{180\ \text{days}} = 810\ m^3/\text{day}$$

5) Dump truck (transportation, soil from bed, lower channel)

$$\frac{960{,}500\ m^3}{180\ \text{days}} = 5{,}340\ m^3/\text{day}$$

6) Bulldozer excavation (stripping)

$$\frac{145{,}400\ m^3}{180\ \text{days}} = 810\ m^3/\text{day}$$

7) Tractor shovel (loading·stripping)

$$\frac{145{,}400\ m^3}{180\ \text{days}} = 810\ m^3/\text{day}$$

8) Dump truck (transportation·stripping)

$$\frac{145{,}400\ m^3}{180\ \text{days}} = 810\ m^3/\text{day}$$

9) Bulldozer (spreading roller-fill)

$$\frac{1{,}400{,}000\ m^3}{180\ \text{days}} = 7{,}780\ m^3/\text{day}$$

10) Vibration roller (roller-fill for face of slope)

$$\frac{416{,}900\ m^3}{180\ \text{days}} = 2{,}320\ m^3/\text{day}$$

. Calculation of job measurement

1) Excavation, bulldozer $V = 3,140\ m^3/day$
21-ton class bulldozer used
Average push: distance $L = 20.0$ m

$$Q = \frac{q \times f \times e \times 60}{Cm}$$

Herewith Q = job measurement (m^3/h)
q = push q'ty per time (capacity 4.13)
(capacity $4.13\ m^3 \times 0.96 = 3.96\ m^3$)
f = function soil q'ty calculated ($\fallingdotseq 0.86$)
e = job efficiency ($\fallingdotseq 0.90$)
Cm = cicle time (min.)

$$Q = \frac{3.96 \times 0.86 \times 0.90 \times 60}{0.99} = 172.8\ m^3/h$$

Obtaining, necessary equipment as the daily operation hour is 6 hours.

Necessary equipment $\frac{3,140}{6 \times 172.8} = 3$ each

2) Drag shovel excavation & loading $V = 4,480\ m^3/day$
Bucket capacity $1.2\ m^3$ drag shovel used

$$Q = \frac{q \times k \times f \times E \times 3,600}{Cm}$$

Herewith Q = job measurement per hour (m^3/h)
q = bucket capacity ($= 1.2\ m^3$)
k = bucket function ($= 0.90\ m^3$)
f = calculated function for soil ($= 0.80\ m^3$)
E = job efficiency ($= 0.70\ m^3$)
Cm = cycle time ($= 25$ sec)

$$Q = \frac{1.2 \times 0.90 \times 0.80 \times 0.70 \times 3,600}{25} = 87.1\ m^3/h$$

daily operation time is 6 hours as the necessary

equipment $\frac{4,480}{6 \times 87.1} = 8.6 = 9$ each

3) Bulldozer carry $V = 2,280\ m^3/day$

21-ton class bulldozer is used
Average carry distance $L = 40$ m

$$Q = \frac{q \times f \times E \times 60}{Cm}$$

herewith Q = job measurement per hour (m^3/h)
q = push q'ty per time
(capacity $4.13\ m^3 \times 0.88 = 3.63\ m^3$)
f = function calculated from soil ($= 1.0$)
E = job efficiency ($= 0.90$)
Cm = cycle time (min.)
$Cm = 0.037 \times L + 0.25 = 1.73$ min.

$$Q = \frac{3.63 \times 1.0 \times 0.90 \times 60}{1.73} = 113.3\ m^3/h$$

Daily operation time 6 hours

$$\text{Necessary equipment} = \frac{2,280}{6 \times 113.3} = 3.35 = 4 \text{ each}$$

4) Tractor shovel (loading) $V = 860\ m^3/day$

Bucket capacity 2.0 m^3 tractor shovel used

$$Q = \frac{q \times K \times F \times E \times 3,600}{Cm}$$

herewith Q = job measurement per hour (m^3/h)
q = bucket capacity (= 2.0 m^3)
K = bucket function (= 1.10 m^3)
F = function calculated from soil (= 0.80)
E = job efficiency (0.70)
Cm = cycle time

$$Cm = m \times e + t_1 + t_2$$

herewith m = function from tractor shovel (= 1.9 sec/m)
e = distance of one way carry (= 10.0 m)
t_1 = shoveling period (= 15 sec)
t_2 = loading, change gear, lost time (20 sec)
$Cm = 1.9 \times 10.0 + 15 + 20 = 54$ sec

$$Q = \frac{2.0 \times 1.10 \times 0.80 \times 0.70 \times 3,600}{54} = 82.1\ m^3/h$$

Daily operation time is 6 hour

$$\text{Necessary equipment} = \frac{860}{6 \times 82.1} = 1.7 = 2 \text{ each}$$

5) Dump truck transportation $V = 5,340\ m^3/day$

8-ton dump truck is used
Average transport distance $L = 2,000$ m

$$Q = \frac{q \times f \times E \times 60}{Cm}$$

herewith Q = soil q'ty transported (m^3/h)
q = loaded soil q'ty per one load (= 5.19 m^3)
f = function calculated from soil q'ty(= 0.80)
E = job efficiency (= 0.90)
Cm = cycle time

$$Cmd = \frac{Cms \cdot n}{60Es} + (T_1 + t_1 + T_2 + t_2)$$

herewith Cmd = cycle time of dump truck (min.)
Cms = necessary period, one cycle of loading equip. (54 sec)
n = loading equip. cycle needed one dump truck (day)

however $n = \frac{qd}{gs \cdot K}$

qd = dump truck loading soil q'ty (= 5.19 m^3)
gs = bucket capacity of loading equip. (= 2.0 m^3)
K = function from bucket (= 1.10)

$$n = \frac{5.19}{2.0 \times 1.10} = 2.36$$

Es = job efficiency of loading equip. (= 0.70)

T_1, T_2 = dump truck both ways running period (min.)

$$T_1 = \frac{2.0}{1.5} \times 60 = 8.0 \text{ min.}$$

$$T_2 = \frac{2.0}{2.0} \times 60 = 6.0 \text{ min.}$$

t_1 = unloading period (= 1.0 min.)

t_2 = preparation period after arrival (0.5 min.)

$$Cm = \frac{54 \times 2.36}{60 \times 0.7} + (8.0 + 1.0 + 6.0 + 0.5) = 18.5 \text{ min.}$$

$$Q = \frac{5.19 \times 0.80 \times 0.90 \times 60}{18.5} = 12.1 \text{ m}^3\text{/h}$$

Daily operation time is 7 hours

$$\text{Necessary equipment} = \frac{5,340}{7 \times 12.1} = 63 \text{ each}$$

6) Bulldozer excavation (stripping) V = 810 m^3/h

8-ton class bulldozer used
Average push q'ty per time distance 12.0 m

$$Q = \frac{q \times f \times E \times 60}{Cm}$$

herewith Q = job measurement (m^3/h)
q = one excavation & q'ty
(capacity 1.10 m^3 x 0.98 = 1.08 m^3)
f = function calculated soil q'ty (= 0.80)
E = job efficiency (= 0.90)
Cm = cycle time (min.)

$$Cm = 0.37 \times 12 + 0.25 = 0.69$$

$$Q = \frac{1.08 \times 0.80 \times 0.90 \times 60}{0.69} = 67.6 \text{ m}^3\text{/h}$$

Daily operation time is 6 hours

$$\text{Necessary equipment} = \frac{810}{6 \times 67.6} = 2.0 = 2 \text{ each}$$

7) Tractor shovel loading (stripping) $V = 810\ m^3/day$

Bucket capacity 2.0 m^3 tractor shovel used
Job measurement per hour = 82.1 m^3/h

Necessary equipment $= \frac{810}{6 \times 82.1} = 1.6 = 2$ each

8) Dump truck transportation (stripping) $V = 810\ m^3/day$

8-ton dump truck used
Average transport - throughway distance L = 500 m

$$Q = \frac{q \times f \times E \times 60}{Cm}$$

$q = 5.19\ m^3$
$f = 0.80$
$E = 0.90$

$$CmD = \frac{Cms \cdot n}{60\ Es} + (T_1 + t_1 + T_2 + t_2)$$

$Cms = 54$ sec
$n = 2.36$
$Es = 0.70$
$T_1 = \frac{0.5}{15} \times 60 = 2.0$ min.
$T_2 = \frac{0.5}{20} \times 60 = 1.5$ min.
$t_1 = 1.0$ min.
$t_2 = 0.5$ min.

$$CmD = \frac{54 \times 2.36}{60 \times 0.70} + (2.0 + 1.0 + 1.5 + 0.5) = 8.03 \text{ min.}$$

$$Q = \frac{5.19 \times 0.80 \times 0.90 \times 60}{8.03} = 27.9\ m^3/h$$

Daily operation time is 7 hours

Necessary equipment $= \frac{810}{7 \times 27.9} = 4.1 = 4$ each

9) Bulldozer spreading

2-ton class bulldozer used

$$Q = \frac{q \times f \times E \times 60}{Cm}$$

herewith Q = job measurement per hour (m^3/h)
q = push q'ty per time (= 3.96 m^3)
f = function calculated from soil q'ty (= 0.80)
E = job efficiency (= 0.90)
Cm = cycle time
$Cm = 0.037\ L \times 0.25 = 0.99$ min.

$$Q = \frac{3.96 \times 0.80 \times 0.90 \times 60}{0.99} = 172.8\ m^3/h$$

Daily operation time is 6 hours

Necessary equipment $= \frac{7,780}{6 \times 172.8} = 7.5 = 8$ each

10) Bulldozer roller-fill $V = 7,780\ m^3/day$

21-ton class bulldozer used

$$Q = \frac{V \times W \times H \times F \times E \times 1,000}{N}$$

herewith
- Q = job measurement per hour (m^3/h)
- V = job velocity (= 3.0 km/h)
- H = thickness stripping (= 0.50 m)
- F = function calculated from soil q'ty (= 0.80)
- W = available width of compaction (= 0.90)
- N = frequency roller fill (= 5)
- E = job efficiency (= 0.80)

$$Q = \frac{3.0 \times 0.90 \times 0.50 \times 0.80 \times 0.80 \times 1,000}{5} = 172.8\ m^3/h$$

Daily operation time is 6 hours

Necessary equipment $= \frac{7,780}{6 \times 172.8} = 7.5 = 8$ each

11) Vibration roller slope face roller fill

$A = 2,320\ m^2/day$

4.5t vibration roller used

Stretched by 8t bulldozer

$$Q = \frac{V \times W \times E}{N}$$

herewith
- Q = job measurement per hour (m^3/h)
- V = job velocity (= 3.0 km/h)
- W = available width of compaction (= 1,140 m)
- E = job efficiency (= 0.80)
- N = roller fill frequency (= 6)

$$Q = \frac{3.0 \times 1,140 \times 0.80 \times 1,000}{6} = 456\ m^2/h$$

Daily operation time is 6 hours

Necessary equipment $= \frac{2,320}{6 \times 456} = 0.85 = 1$ each

Equipment used for the river improvement work are as follows.

Overall, earthworks are as follows.

Excavation

1.	River bed		4,375,600 m^3
2.	Low-water channel		1,600,800 m^3
3.	High water bed		838,000 m^3
		Total	6,814,400 m^3

Banking

1.	Main channel levee		2,787,200 m^3
2.	Main river maintenance road		770,000 m^3
3.	Tributary levee		1,917,300 m^3
4.	Tributary maintenance road		570,900 m^3
5.	Others		12,600 m^3
		Total	5,998,000 m^3

Spoil Bank

1.	Main channel stripping		407,900 m^3
2.	Tributary "		295,300 m^3
		Total	703,200 m^3

The following equipment will be used.

Excavation

1.	Bulldozer, 21 ton, excavated	2,000,000 m^3
	Bulldozer, 21 ton, carry	2,000,000 m^3
2.	Truck Shovel 2.0 m^3 excavated	3,407,200 m^3
	Dump truck 8 ton transported	3,407,200 m^3
3.	Bulldozer 21 ton excavated	1,407,200 m^3
	Tractor, Shovel, 21 ton, excavated	1,407,200 m^3
	Dump truck, 21 ton, transported	1,407,200 m^3

Banking

1.	Bulldozer, 21 ton, spread	5,982,400 m^3
2.	Bulldozer, 21 ton, Roller fill	5,982,400 m^3
3.	Vibration Roller Rollerfill (8 ton Bulldozer stretch)	1,819,000 m^3

Spoil Bank

1.	Bulldozer, 8 ton, excavated	703,200 m^3
2.	Tractor Shovel 2.0 m loading	703,200 m^3
3.	Dump Truck, 8 ton, Transported	703,200 m3
4.	Bulldozer, 8 ton, Spread	703,200 m^3

c) Construction schedule

Excluding one year preparation period, the construction of the project is scheduled for 6 years commencing from 1978. Construction is scheduled to be performed in dry seasons taking a low efficiency in wet seasons into consideration. Annual work days are 195 days including 15 days for preparation. Fig. 6.3.2 shows a work schedule of the river improvement works.

3.6.3 Examination of the Heavy Equipments and the Material

Although Work measurement of major works are shown in paragraph 3.3.7 herewith examination for the heavy equipments and the material will be provided regards the major work.

The major work and required equipments and materials are as follows.

1.	Excavation	6,814,000 m^3
2.	Banking	5,998,000 m3
3.	Spoil Bank	703,000 m^3
4.	Concrete (used for Sluiceway and Stone Piching)	40,000 m^3
5.	Cement	11,000 t
6.	Reinforcement bar (used for Sluiceway)	700 t
7.	Aggregate (used for Concrete)	52,000 m^3
8.	Stone (used for Stone Piching ∅35 cm)	25,000 m^3
9.	Concrete Pile (L=9.00 m, used for Groin)	2,000 pieces
10.	Concrete Pile (L=6.00 m, used for Groin)	1,400 pieces
11.	Wooden Pile (used for Groin and Sluiceway)	13,000 pieces
12.	Wire Cylinder (Groin foot protection)	21,000 pieces
13.	Wire Basket (Revetment, foot protection)	62,000 pieces
14.	Stone (used for Wire Cylinder and Wire Basket)	82,000 m^3

15.	Sluice gate (Steel 32 place)	180 t
16.	Administration Bridge (used for Sluiceway)	100 t
17.	Bridge (Nguter Bridge, Super structure)	500 t
18.	Sodding (Banking protection)	1,819,000 m^2
19.	Others (Bamboo, Scaffold log, etc.)	1 set
20.	Construction Machinery and Equipment	1 set

The item 1 excavated soil of the above will be diverted to the item 2 banking of all.

Distance of excavated soil carried is approx. 20 km. The item 3 Spoil Bank will be used for diversion as for the filling of the old river course. The item 4 Concrete are to be used for the back-fill of stone pitching and also the body concrete of sluiceway.

The item 5 Cement & the item 6 Reinforcement bar are hard to obtain in Indonesia so that imported material would be used. The item 7 Aggregate and the item 8 - 14 Stone would be collected from the upper reach of the tributaries. And the item 11 Wooden Pile will be supplied from the inundation area on the upper stream of the Wonogiri Dam.

The item 9 - 10 Concrete pile, 12 Wire Cylinder, 13 Wire Basket are hard to obtain as they are not manufactured locally, so some imported materials will be used. The item 15 Sluice gate, 16 Administration Bridge, 17 Bridge, including beams are all made of steel, and those must be imported.

The item 18 Sodding Material will be collected from the place around Work section.

The item 20 Construction machinery and equipment are not manufactured in Indonesia, so these will be imported. Although, some heavy equipment are under the control of Proyek Bengawan Sala Office. These are mostly for other projects, and its diversion to this river improvement work will be very difficult. Therefore, all heavy equipment for this river improvement work should be imported.

3.7 LANDS AND HOUSES TO BE ACQUIRED

The land necessary for improvement of the river is as follows;

Main channel	Approx. 660 ha
Tributary	" 200 ha
Total	Approx. 860 ha

The total area of 860 ha consists of about 420 ha of Sawah and about 440 ha of Kampong area. Since the land to be aquired is limited to the area for the banking total land acquisition is estimated as follows:

Purchase Area

Sawah	Approx. 160 ha	
Kampong	" 160 ha	(Yard)
Total	Approx. 320 ha	

Number of houses to be moved for the implementation of the proposed river improvements is 2,300. The breakdown of which is shown the in the following table.

	Number of Houses		
Kinds	Banking Area	Highwater Channel & Retarding Basin	Total
Bamboo	612	1,044	1,656
Wood	85	145	230
Brick	153	261	414
Total	850	1,450	2,300

Note: The table above was compiled from the results of processing of the data gathered from 35 villages in the Surakarta area.