REPUBLIC OF INDONESIA

FEASIBILITY REPORT

ON

SURABAJA RIVER IMPROVEMENT PROJECT

FEBRUARY 1973

OVERSEAS TECHNICAL COOPERATION AGENCY
GOVERNMENT OF JAPAN

REPUBLIC OF INDONESIA

FEASIBILITY REPORT

ON

SURABAJA RIVER IMPROVEMENT PROJECT

FEBRUARY 1973

OVERSEAS TECHNICAL COOPERATION AGENCY

GOVERNMENT OF JAPAN

PREFACE

At the request of the Republic of Indonesia, the Government of Japan agreed to undertake an overall planning cum feasibility study on the Surabaja River Improvement Project in East Java Province, and entrusted the implementation of the study to the Overseas Technical Cooperation Agency, an executing organization of the Japanese Government.

The Agency made a contract for the study with NIKKEN Consultants, Inc., Tokyo, and dispatched a study team comprising eight experts headed by Dr. Seiichi Sato, vice president of the said company to the Republic of Indonesia over a period from December 21, 1971 to March 19, 1972.

The study team conducted field investigation in cooperation with the Indonesian Authorities concerned. After its return to Japan, the team made various studies and analysis of the data, materials and opinions collected in Indonesia, and completed the draft report. On 26th, November, 1972, the team again visited Indonesia with the draft report and had discussions about its contents with Indonesian Authorities. Taking their opinions into consideration the team, revised the draft report. As a result, the report has been finalized and is herewith submitted to the Indonesian Government.

Meanwhile, the Agency organized an Inspection Committee comprising five men of learning and experience, headed by Mr. Akira Miyazaki, Ministry of Construction, for advices to the study team on the technical problems of the Project.

I sincerely hope that the report will contribute to the implementation of the Project and serve in promoting friendly relations between Indonesia and Japan.

Finally, I wish to take this opportunity to express our heartfelt gratitude to the officials of the Republic of Indonesia and the staff of the Japanese Embassy in Jakarta for the wholehearted support and cooperation extended to the team throughout the study period.

Keiichi Tatsuke
Director General,
Overseas Technical Cooperation Agency,
Tokyo, Japan

February 1973

LETTER OF TRANSMITTAL

February, 1973

Mr. Keiichi Tatsuke
Director General
Overseas Technical Cooperation Agency
Tokyo, Japan

Dear Sir:

This is the final report of the feasibility study of the Surabaja River Improvement Project prepared by the Japanese Study Team which was organized and despatched by the Overseas Technical Cooperation Agency, Japan.

The team made the studies including data collection and surveying for about three months from December 21, 1971 to March 13, 1972 in Indonesia in cooperation with the counterpart team which was organized by the Ministry of Public Works and Power, the Government of Indonesia. During the study, meetings for discussion were held four times in Djakarta and Surabaja between the study team and the counterparts. Further studies were made in Japan in connection with analysis of collected data, planning of the project, and justification of feasibility.

After finishing the draft final report, meetings were held in Djakarta and Surabaja from November 30 to December 9, 1972 for the discussions on the improvement works to be adopted in the present phase between the study team and the counterparts/officials. Taking into consideration the results of disccusions, the final report has been completed and is herewith submitted to the Overseas Technical Cooperation Agency.

On the other hand, several meetings of the Inspection Committee composed of the Japanese authorities concerned were held in Tokyo for the disccusions on the present project and the final report was approved by it.

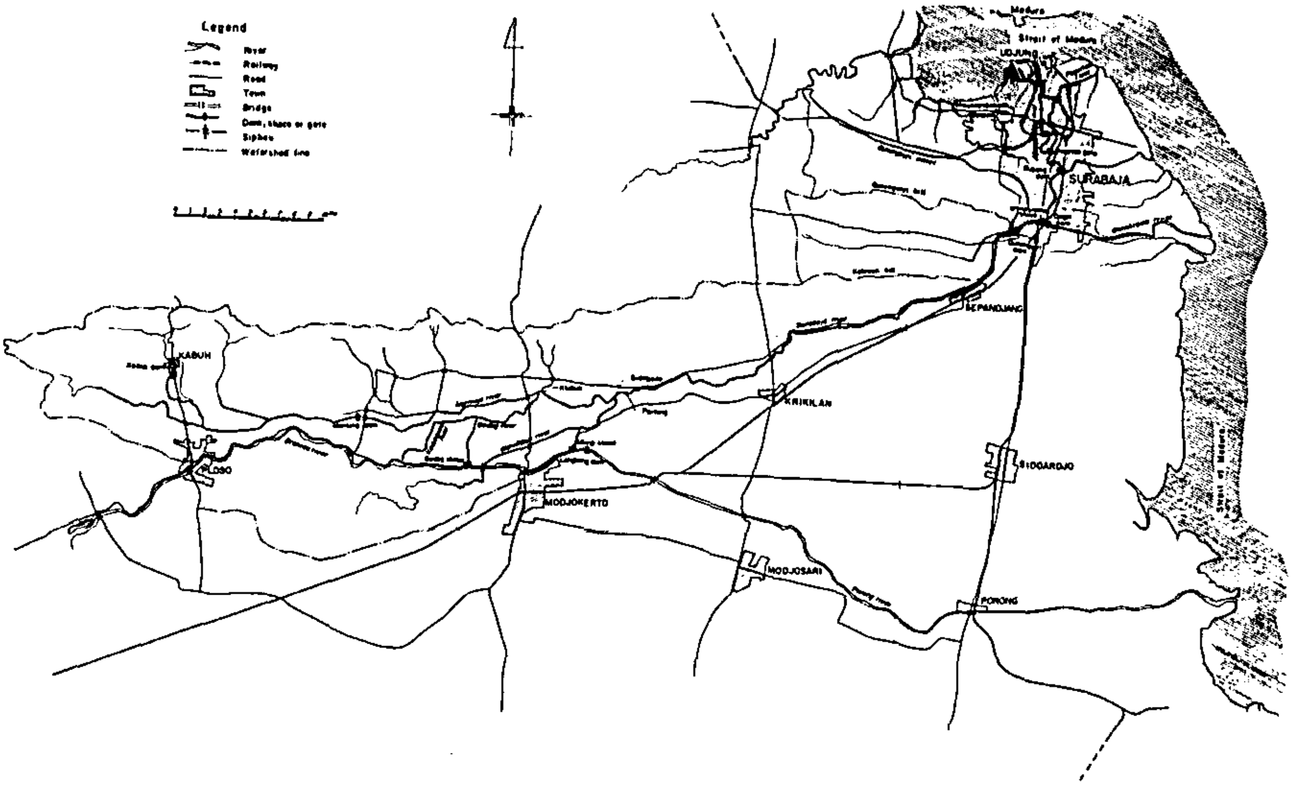
The Japnese study team wishes to express its deep appreciation to the counterparts/officials in the Directrate General of Water Resources Development and the Irrigation Service in East Java Province, the staffs of the Japanese Embassy in Djakarta and the Japanese Consul in Surabaja, the staffs of the Overseas Technical Cooperation Agency and the officials of the Japanese authorities concerned for their kind cooperation and support in performing the study.

Yours faithfully,

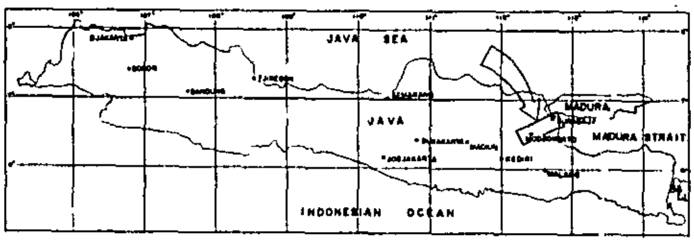
Dr. Seiichi Sato
Leader
Japanese Study Team
for the Surabaja River
Improvement Project

The Mas River

BASIN MAP



Legend
River
Railway
Road
Town
Bridge
Dam, sluice or gate
Siphon
Watershed line
KABUH
PLOSO
MODJOKERTO
KRIKILAN
SEPANDJANG
SURABAJA
UDJUNG
Strait of Madura
SIDOARDJO
MODJOSARI
PORONG



JAVA SEA
JAVA
MADURA
MADURA STRAIT
INDONESIAN OCEAN

ABBREVIATION AND UNIT

DPUT	Departmen Pekerdjaan Umum Dan Tenaga Listrik (Department of Public Works and Power)
DRSAD	Directorate of River and Swampy Area Development
DPPDT	Dinas Pengairan Propinsi Djawa Timur
OTCA	Overseas Technical Cooperation Agency, Japan
SHVP	Surabaja Haven Vloed Peil
BOD	Biochemical oxygen demand
^oC	Degree(s) centigrade
cm	Centimeter(s)
cm^2	Square centimeter(s)
cms, or m^3/s	Cubic meter per second
El	Elevation
ft	Foot
gr	Gram(s)
ha	Hectare(s) = 2.47 acres
hr	Hour(s)
HP	Horse power
HWL	High water level
in	Inch
kg	Kilogram(s)
km	Kilometer(s) = 0.62 miles
km^2	Square kilometer(s) = 0.386 square miles
kt	knot
kw	Kilowatt(s)
kwh	Kilowatt-hour(s)
lb	Pound(s) = 0.4536 kg
l/s	Liter per second
l/s/ha	Liter per second per hectare
m	Meter(s) = 3.28083 feet
m^2	Square meter(s)
m^3	Cubic meter(s)
min	Minute(s)
mm	Millimeter(s) = 0.04 inches
mm^2	Square millimeter(s)

m/s	Meter per second
Ph	Potential hydrogen
ppm	Part per million
qul	quintal = 100 kg
sec	Second(s)
ton	Metric ton(s) = 1,000 kg
yr	year
%	Percent
Rp	Rupiah(s)
$	U.S. dollar(s) = ¥ 308 = Rp 415
¥	Japanese yen

C O N T E N T S

VOLUME I

MAIN REPORT

PART 3. WATER REQUIREMENT

VOLUME II

STUDY AND DATA

PART 4. STUDY

PART 5. DATA LIST

VOLUME I

MAIN REPORT

I. INTRODUCTION

This is the final report of the feasibility study on the Surabaja River Improvement Project prepared by the Japanese Study Team which was organized and despatched by the Overseas Technical Cooperation Agency, Japan at the request of the Government of Indonesia.

The objective of the study is to formulate a plan for (1) protecting Surabaja City from a menace of flooding of the Surabaja River, (2) relieving Surabaja City from habitual inundations caused by local heavy rainfall in the city area, (3) relieving Surabaja City from habitual inundations resulting from devastation of sea dike, (4) rehabilitating or improving superannuated hydraulic structures so as to facilitate water intake as well as drainage and to enable to control floods, (5) protecting the Marmojo basin from habitual inundations, and (6) estimating the water requirements, and finally to justify the economical feasibility of the plan formulated.

The study team was composed of the following ten experts:

Dr. Seiichi Sato	Leader	NIKKEN Consultants, Inc.
Ken Soejima	Adviser	The Ministry of Construction.
Dr. Shoichiro Nakagawa	Adviser	The Ministry of Agriculture and Forestry.
Sumihisa Ohira	River planning	Tokyo Construction Consultants, Co., Ltd.
Yasuo Iwasaki	Hydrology	Nippon Koei Co., Ltd.
Noboru Jitsuhiro	Hydraulics	NIKKEN Consultants, Inc.
Jiro Okazaki	Hydraulic facilities	NIKKEN Consultants, Inc.
Masaharu Matsui	Irrigation	Sanyu Consultants International, Inc.
Shohei Sata	Drainage and sewerage	Nihon Suido Consultants Co., Ltd.
Kinichi Ohno	Economic analysis	NIKKEN Consultants, Inc.

Corresponding to this, the counterpart team shown on the next page was organized by the Ministry of Public Works and Power, the Government of Indonesia.

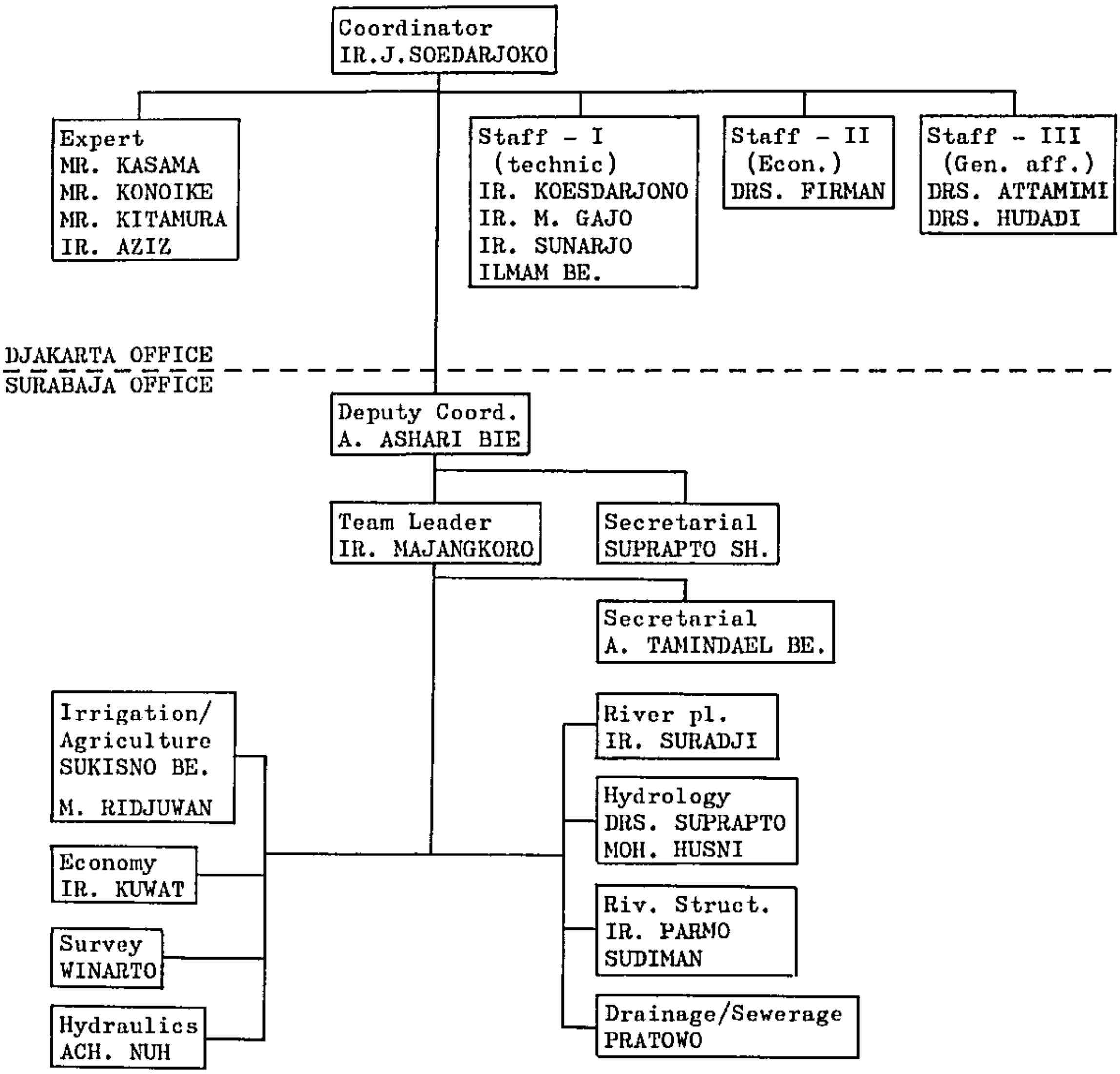
The study team arrived at Djakarta on December 21, 1971 and the first meeting was held in Djakarta on December 23 and 24, 1971 for discussion between the study team and the counterpart team concerning (1) the scope of works in the Terms of Reference, shown in Appendix A, which had been presented beforehand to the Government of Japan and (2) questions, Appendix B, which had been asked to the Directorate of Water Resources Development prior to the study team's leaving Japan. The results of discussions are shown in Memorandum No. 1, Appendix C.

After the above-mentioned discussions in Djakarta, the study team moved to Surabaja, where data collection and surveying were made until March 13, 1972, in the course of which, on February 24, the second meeting was held in Djakarta for further discussions, the results of which are shown in Memorandum No. 2, Appendix C. Concerning some problems which had not been finished in Djakarta, detailed discussions were made in Surabaja on February 26, 1972, the results of which are shown in Memorandum No. 3, Appendix C.

ORGANIZATION CHART OF COUNTERPARTS

FOR

THE FEASIBILITY STUDY OF THE SURABAJA RIVER IMPROVEMENT PROJECT



On March 10 and 11, 1972, the fourth meeting was held in Surabaja for the final discussion on the interim report between the study team and the counterpart.

Since the study period in Indonesia was limitted to three months, some data which are mainly related to the surveying of the Surabaja river system had not been obtained before the study team left Indonesia of March 19, 1972. Those data were sent to the team in Tokyo later on.

The study in Indonesia was principally made pertaining to reconnaisance, data collection, surveying, and discussions on problems. Hence, further studies were made in Japan in connection with analysis of collected data, planning of the project, and justification of feasibility. In the course of the study in Japan, two engineers from the counterpart team, Ir. Usef Gajo and Ir. R.M. Majangkoro, joined the study team for about two months.

After finishing the draft final report, the fifth meeting was held in Surabaja from November 30 up to December 4, 1972 and successively the last meeting was held in Djakarta from December 5 up to 9, 1972 for the discussions on the improvement works to be adopted in the present phase between the study team and the counterparts/officials. On December 11, 1972, Minutes of Meetings shown in Appendix D were made and signed between the team leader Dr. S. Sato and the coordinator Ir. J. Soedarjoko. In accordance with the Minutes, the final report was prepared.

On the other hand, several meetings of the Inspection Committee composed of the Japanese authorities concerned were held in Tokyo for the discussions on the present project and the final report was approved by it.

The report contains two volumes composed of five parts. Volume I is the main report which comprizes Introduction, Summary, Recommendations, Part 1 Master Plan, Part 2 The Surabaja River Improvement Project, Part 3 Water Requirement, and Appendices. Volume II is the report on the technical and economic studies and the list of data used for them.

The present project is considerably complicated as well as large in scale. If the study team had not been strongly supported by the staffs in the Directorate General of Water Resources Development and the Irrigation Service in East Java Province and all the staffs of the counterpart team, the present study could not have been completed. The Japanese study team wishes to express its highest appreciation for the cooperation offered by the authorities concerned, especially its wholehearted gratitude to the staffs of the counterpart team mentioned in the organization chart.

II. SUMMARY

1. Extent of the Project Area.

The Surabaja River is a branch of the Brantas River which has a drainage area of about 12,000 km^2, and runs through Surabaja City, the second largest town in Indonesia. The drainage basin of the Surabaja River stretches over twenty-seven Ketjamatans of East Java Province and practically ends at Gunungsari Dam located nearly on the southern boundary of the city. The area of the basin totals to 604 km^2 gathering the catchment areas of the Marmojo River, 289.7 km^2, the Watudakon River, 99.4 km^2, and other small tributaries, 215 km^2. On the lower part of the river, Surabaja City covers an area of about 290 km^2 which contains a population of about 1,600,000 as of 1971.

The area of the present project covers the above two areas totaling to about 890 km^2, which contain the total population of about 1,900,000, irrigation areas totaling 6,956 ha, and about 2,800 factories including such important ones as Barata Steel Works, Gresik Cement, and Cresik Petro Kimia.

In this project area, there exist (1) important hydraulic facilities such as Mlirip Sluice, Gunungsari Dam, Djagir Dam, Wonokromo Sluice, Gubeng Dam, and Pegirian Gate, (2) important drainage facilities such as pumping stations and Morokrembangan Boezem, and (3) an important structure for prevention of see water, a sea dike extending over a length of 17 km. The locations of these facilities and structures are shown in Fig. 1 of Chapter I, Part 1.

The designation of the Surabaja River system to be delt with in the present project is partially adjusted and shown in Fig. 2 in Chapter I, Part 1.

2. Major Problems in the Project Area.

There are many problems in the present project area concerning removal of troubles and improvement of water intake. They can be arranged as follows:

a. In the rainy season, various quarters of the urban area suffer from habitual inundations due to local heavy rainfall. The rain water fallen to the central part of the city is to be drained by the Mas River partially receiving the aid of drainage pumps, the rain water fallen to the western part is to be drained by the Greges River to the sea making use of the storage action of Morokrembangan Boezem, and the rain water fallen to the eastern part is to be drained to the sea by the several drainage canals crossing the sea dike.

   However, these drainage systems are not working successfully, because (1) the carrying capacities of the main drainage canals including the Mas River is extremely decreased due to silting and other reasons, (2) the storage-capacity of the boezem is decreased owing to silting as well as fish ponds built in the boezem, (3) all the flap gates built on the sea dike at the crossing points of the drainage canals are so deteriorated by sea water that they cannot move in draining, (4) the secondary and tertiary ditches including road-side drains are all devastated, so that the road surfaces form a kind of storage ponds, and (5) the capacities of most of the drainage pumps seem to be reduced due to superannuation.

b. In the farming area of about 6,700 ha in the city, inundations are seen in many places during every rainy season. This is due to the facts that (1) the drainage canals have no sufficient capacities to meet the heavy rainfall fallen to the farm lands and the high tide level at their outlets to the sea and (2), especially in the eastern part of the city, the flap gates installed on the sluices at the crossing points on the sea dike have lost their functions as mentioned above.

c. The Surabaja River system has several important facilities such as Mlirip Sluice, Gunungsari Dam, Djagir Dam, Wonokromo Sluice, Gubeng Dam, and Pegirian Gate, most of which were constructed principally for the purpose of taking water for irrigation and later the purposes of drinking and industrial uses were added. All of them were built so many years ago. For instance, Mlirip sluice was built in 1857 together with Lengkong Dam, Gunungsari Dam was built nearly in 1907 at the same time as the construction of Gubeng Dam, and both Djagir Dam and Wonokromo Sluice were built around 1917. Hence, these important structures have already been so superannuated and become so obsolete that they cannot meet the today's demand for timely operations for not only water use but also flood control.

   Further, the banks of low water channels of the Surabaja and the Wonokromo Rivers are seriously scoured at several places and some portions of the dikes have been cut on the top or partially removed.

d. The sea dike which was constructed to prevent the intrusion of salt water has already been devastated in spite of frequent repair works. Some portions of the dike were nearly broken in the past.

   Furthermore, as mentioned already, all the flap gates of the eight sluices on the dike are extremely deteriorated and have completely lost their functions to prevent salt intrusion and to drain landside excess water. This situation has brought habitual inundations on the farm lands which elevations are the same or lower than the high tide.

e. The lower basin of the Marmojo River suffers from severe inundations during every rainy season. This is due to the facts that (1) the carrying capacity of the main stem of the river does not meet the amount of runoff from the tributaries and (2) the river suffers from high and long-lasting backwater resulting from the habitual diversions of flood water from Gedeg Sluice and Mlirip Sluice throughout the flood season of the Brantas River.

f. Irrigation in the city area is all of gravity system and its major water is taken from the Surabaja River.

   There are two irrigation systems of Gunungsari and Gubeng. Major irrigation facilities are Gunungsari Dam and Gubeng Dam which were constructed around the beginning of this century.

   Both facilities are entirely superannuated and man-power operation of needles is not effective for controlling water surface.

   Especially in large runoff season, the treatment of needles is very dangerous and setting-on and taking-out works of them are not easy. This unfavorable condition results in great fluctuation of irrigation-water intake.

   The related irrigation canals are mostly silted up and both banks of them are collapsed. Some turnouts, checks and water-measuring devices are broken or deteriorated. Thus they may have much losses of irrigation water.

Especially, although the Gungsari canal starting from Gunungsari Dam has been built originally for the purpose of carrying the irrigation water, the present canal always has to receive the whole runoff from Gunungsari Hill. In order to handle the excess water, spillways are installed at three points on the canal. However, owing to the shortage of the capacities of the spillways and the canal itself, frequent inundations occur on the left-side lands and, sometimes, the right bank of the canal is cut by inhabitants to relieve their inundated area, which results in releasing excess water into the urban area of the city and encouraging the inundation.

On the other hand, the Gubeng canal system is passing through the center of the city and much waste materials from the housing area are depositing in the canals, particularly in the upstream section. It is raising a problem whether the water quality of the canal flow is suitable or not for irrigation throughout a year. Separation of the canal functions (irrigation and sewerage) should be taken into consideration for the Gubeng irrigation system.

The basin situated between Gunungsari Hill and Kebraon Hill is drained by the Kedurus River which joins to the Surabaja River the tail water of Gunungsari Dam. In the rainy season, the water level at the confluence often becomes higher than the elevation of the inner land, which brings severe inundations in the paddy fields.

g. Surabaja City is now so energitically expanding that the Master Plant Team of the city has forecasted that the population of the city will reach 4,800,000 after about twenty years and consequently the present farming area will be converted into the urban zone or other related to the urban life.

   Accordingly, the water demand will also be being changed. From this point of view, the forecast of water requirements is requested for each use of irrigation, drinking, industry, and dilution of polluted river water.

3. Countermeasures.

The following countermeasures are proposed against the abovementioned problems.

a. The carrying capacities of drainage channels in the urban area have to be recovered or improved by proper means such as deepening and widening or lowering the normal water levels if possible. However, these works should be done in the sequence from the main canal such as the Mas River to small drains on the road sides.

b. The drainage channels in the farming land also have to be treated similarly to the above.

c. Pumping stations have to be improved in conformity with the design discharges allocated to them.

d. The control function of Morokrembangan Boezem has to be recovered by dredging the siltation in it and the safety of the Boezem has to be secured by replacing the existing Mitre gates to new ones.

e. Gunungsari Dam has to be renewed.

Mlirip Sluice has to be improved by replacing the existing stop-log gate to a modern one driven by electric motor.

Djagir Dam has to be improved by motorizing the existing gates and has to be strengthened on the aprons.

Wonokromo Sluice may be left as it is for the time being.

f. The dikes which have been cut on their tops or partially removed have to be rehabilitated, and the low-water banks severely eroded have to be revetted.

g. It is desirable, if possible, to lower the normal water level upstream of Gubeng Dam by improving it. This will also allow to lower the normal water level upstream of Djagir Dam. These will give good effects to the drainage not only of the urban area but also of the extensive basin between Gunungsari Hill and Kebraon Hill.

However, the implementation of this idea should not be decided untill sufficient studies have been finished, because (1) salt water intrusion into the wells located around the dams and lowering down of water level of the wells are conceivable and (2) some compensation measures are needed for securing the present water intake for drinking, irrigation, and other industrial uses.

Since the present river water to be taken for the said purposes are already polluted by the wastes from the urban area and the degree of contamination will increase year by year with the development of the city, a new source for cleaner water must be seeked somewhere upstream from Gunungsari Dam. Therefore, when we consider of both the improvement of the urban drainage and the security of cleaner water, the idea of lowering the said normal water levels is worthy of future studies.

h. Eight sluices with flap gates on the sea dike, called Tjumpat, Kendjeran, Sukolilo, Larangan, Wonosari, Kalidami, Keputih, and Medokan have to be renewed and a new sluice, say New Tambakwedi, has to be added. Further some portions of the dike have to be strengthened.

i. Gunungsari Canal has to be widened and deepened so as to be capable of accepting the runoff from Gunungsari Hill. In this case a new irrigation canal or conduit is needed, for instance, along a dike of the canal. At the same time, the existing spillways have to be improved or renewed.

j. The Marmojo River has to be improved by widening and deeping within the extent that the works are feasible.

4. Design Flood Discharges.

a. The following fundamental principle has been laid down for handling flood water.

The flood water of the Brantas River shall be drained to the sea through the Porong River, the flood water coming from the Surabaja River's own basin which ends at Djagir Dam shall be drained to the sea through the Wonokromo River, and the rain water fallen to the city area shall be drained to the sea through the major drainage canals including the Mas River and the Greges River.

b. Fifty-year flood has been adopted at the present stage for the improvement of the Surabaja/Wonokromo River. The design discharges are as follows:

   370 m^3/s for the reaches from the river mouth to Krikilan

   350 m^3/s for the reaches from Krikilan to Sidogede

c. Twenty-year flood has been adopted at the present stage for the improvement of the Marmojo River. The design discharge is as follows:

   230 m^3/s for the reaches from Sidogede to Klubuk

   Although the return period of design discharge for this river is smaller than the Surabaja's, the inundation will surely be reduced so much, because the habitual diversion of the Brantas flood into the Marmojo and the Surabaja Rivers are to be stopped after the completion of New Lengkong Dam.

d. Ten-year flood has been adopted at the present stage for the improvement of the Mas River in consideration of the importance this river, though five-year flood is usually used in many towns and cities in the world.

   The design discharge varies from 70 m^3/s at the river mouth to 20 m^3/s at Wonokromo Sluice, as shown in detail in Chapter IV, Part 2.

e. Five-year flood has been adopted at the present stage for the improvement of the drainage canals other than the above, according to the usual way for laying down the design storm in the urban area.

5. Improvement Works at the Present Phase.

Since some of the above-mentioned problems need further studies before planning, the following improvement works which compose the main part of the whole project have been adopted in this report as the plan at the present phase.

a. Improvement works of the Surabaja River composed of construction of New Gunungsari Dam, improvement of Mlirip Sluice by replacing the existing gate to a modernized one and Djagir Dam by motorization of gate operation and repairs of aprons, and repairs of dikes and revetments.

b. Improvement works of the Mas River by excavation of about 210,000 m^3 and revetment of about 8,000 m.

c. Improvement works of Morokrembangan Boezem by dredging of about 250,000 m^3 and renewal of Mitre gates.

d. Improvement works of the sea dike by renewal of eight of nine existing sluices, construction of a new sluice, and repairs of the dike body.

e. Improvement works of the Marmojo River by excavation of about 320,000 m^3

and some embankments and revetments.

f. Appurtenant facilities composed of a concrete-block yard and concrete test apparatus, six recording-rain-gage stations, nine recording-water-gage stations, and four communication stations.

The above-mentioned construction works are scheduled to be carried out in five and a half years including the period for detail design, and the total cost of construction works amounts to Rp 3,473,041,000. Major fundamental principles in executing the works are as follows:

a. Engineers.

The construction works shall be carried out under the supervision of the Indonesian engineers who have good experiences in similar works to the present ones, especially the dam construction and dredging as Gunungsari Dam and Morokrembangan Boezem. The Indonesian engineers shall be assisted by consulting engineers.

b. Organization.

The organization shown in Chapter X, Part 2 is recommendable for the execution of the works.

c. Equipment.

Some special equipment listed on ANNEX No. 4, Appendix D, Volume I shall be provided by DPUT and delivered at the job sites in good working condition, and ordinary equipment required for construction works, such as concrete mixers, etc., shall be prepared by contractors.

d. Gates.

The construction of gates shall be carried out in cooperation with foreign experienced contractor. With regard to the provision of gates, design and manufacturing shall be ordered to the foreign contractor. However, manufacturing of the gate bodies and installation of the gates shall be carried out by local manufacturer under the technical guidance of the said contractor.

6. Economic Justification.

Benefit-cost analysis has been made with regard to the total cost of the above-mentioned improvement works including maintenance and operation and the benefits to be derived from the works. The results are shown below as to two cases of project life, 20 and 50 years.

Project life	20-year		50-year	
Discount rate	3%	6%	3%	6%
Benefit-cost ratio	2.28	1.78	4.58	2.80
Internal rate of return	13.9%		15.8%	

This indicates that the present project is sufficiently feasible from the economic point of view. However, the benefits considered in this analysis were all direct and tangible ones. Therefore, if indirect and/or intangible benefits other than the above are taken into consideration, the present project will be the more feasible.

7. Water Requirements.

The present project area neccessitates not only the water for drinking, irrigation, and industry, but also the water for dilution of polluted river water in the Mas and the Pegirian. For the purpose of estimation of annual water requirements during the twenty years from 1972 up to 1992, the variation of the population of Surabaja City and the acreage of farm land to be irrigated during the above twenty years were first forecasted. They were estimated at 3,980,000 souls and 6,349 ha respectively in the year of 1992. On the basis of these figures, the water requirements in every five years have been estimated as follows:

unit : cu m per second

Year	1972		1977		1982		1987		1992	
water use	Ave.	Max.	Ave.	Max.	Ave.	Max.	Ave.	Max.	Ave.	Max.
Municipal	1.60	1.90	3.00	3.75	4.86	6.08	7.12	8.90	9.81	12.26
Industrial	1.11	1.33	1.33	1.60	1.53	1.84	1.68	2.02	1.82	2.18
River Dilution	0.42	0.53	0.94	1.18	1.71	2.14	2.78	3.48	4.17	5.13
Irrigation	6.29	12.24	6.20	12.07	6.13	11.93	6.05	11.79	5.98	11.65
Total	9.42	16.00	11.47	18.60	14.23	21.99	17.68	26.19	21.78	31.22

III. RECOMMENDATIONS

1. Since the present project has basic importance to the development of Indonesia as well as Surabaja district and is technically and economically feasible, the implementation of the project should be commenced as soon as possible.

2. With regard to the areal improvement of the urban drainage system and the improvement of the irrigation system, further studies of technical and economic feasibility should be made successively after the present study. For this study about one year will be required.

3. On completing the present project, an operation organization for water management and flood control should be established. The organization should be studied at the stage of final design for the present project.

4. Although 50-year flood has been adopted as the design discharge of the Surabaja River, it may be necessary in the future to adopt a discharge of more than 50-year return period, say 100-year flood, in accordance with the development of the municipality of Surabaja and its hinterland. At the time, the whole width between the both dikes of the river will surely play an important role. Therefore, new construction of any houses and/or any facilities should not be permitted in the area between the both dikes.

PART 1

MASTER PLAN

CHAPTER I

PRESENT STATUS AND MAJOR PROBLEMS IN THE SURABAJA RIVER BASIN

1. General.

(1) Socio-economic back ground of the project area.

1) Project area.

Indonesia is located in a tropical zone which lies between two oceans, the Pacific and the Indonesian Oceans. Area of the land is about 2,020,360 km^2, and the whole territory extends from 6^o north latitude to 11^o south latitude and from 95^o to 141^o east longitude. The greatest distance from west to east is 5,110 km and the greatest distance from north to south is 1,888 km.

Java is the fifth largest island in Indonesia and covers an area of 134,044 km^2 including Madura island, and it is divided into three provinces, East Java, Central Java and West Java. Administratively, the present project area is located in East Java province in Java island. East Java Province covers an area of 46,866 km^2 including Madura island and is divided into the following administrative units:

Administrative Units in East Java

Province	Capital	Number of Kabupatens	Number of Kotamadyas	Number of Ketjamatans
East Java	Surabaja	29	8	533

The present project area, as is seen in Fig. 1 of Chapter XXIV, Part 4, stretches over twenty-seven Ketjamatans including those of Kotamadya Surabaja, capital city of the East Java province.

Total area of twenty-seven Ketjamatans is estimated at nearly 87,000 ha which comprize the Surabaja city area of 29,000 ha and its surrounding area of 58,000 ha.

The Surabaja city area of 29,000 ha is roughly composed of such zones as densely inhabited area of 7,000 ha, farm land of 17,000 ha, fish pond of 4,000 ha and other area of 1,000 ha. It is expected, however, that the farm land will rapidly be decreased with the future development of the city.

The surrounding area of 58,000 ha excluding the Surabaja city area is composed of such zones as densely inhabited area of 11,000 ha, farm land of 40,000 ha and other area of 7,000 ha. This area of which farm land amount to nearly 70% of the whole has formed a productive farm zone owing to favourable climatic conditions and abundant supply of irrigation water from the Surabaja river.

2) Climate.

The project area is situated in latitude 7^o S and longitude 113^o E.

According to the records obtained by the Surabaja Meteorological Observatory, monthly mean temperature is approximately 26.9°C, and the maximum and the minimum temperatures appear in October through November and July through August respectively. Monthly mean relative humidity is about 78% and annual mean rainfall depth is about 1,800 mm. In general, the rainy season continues for the five months from November to March; on the contrary, the dry season is for the five months from June to October. The records of monthly mean rainfall depths for the six years from 1960 to 1965 at the above observatory show that the maximum is 329 mm in January and the minimum is 12 mm in September.

3) Population.

The population census in Indonesia in 1970 shows that the total population was 115 million and the increase was roughly 18 million compared with the census in 1961, i.e. the rate of the population increase was approximately 2%. In terms of population, Indonesia is ranked as the fifth largest country in the world after China, India, the Soviet Union and the United States. Especially, Java island is one of the most densely populated areas in the world. There live 557 people per square kilometer in 1970.

According to the Monthly Statistical Bulletin published by Biro Pusat Statistik in Jan. 1972, Java has a population of about 72,286,000 in 1969. In spite of less than 7% of the total land area, Java contains nearly 65% of the population of the whole country.

The population in the project area in 1971 is about 2,030,000 including 1,622,000 of Kotamadya Surabaja. Examination of mean density of population shows that 16 Ketjamatans of Kotamadya Surabaja and 11 Ketjamatans in the project area except Kotamadya Surabaja have respectively 5560 persons per km^2 and 740 persons per km^2. It can be said that the present project area is one of the most densely populated ones in Java island, because mean value of the population per km^2 in the area is by far larger than that of the whole Java mentioned above. Population of each Ketjamatan is shown in Table 1 of Chapter XXIV, Part 4.

According to the statistics of population prepared by Statistic Division of Kotamadya Surabaja, population of Surabaja city for the last four years from 1968 to 1971 are as follows:

Population of Surabaja City

Year	1968	1969	1970	1971
Population	1,285,810	1,409,363	1,518,352	1,622,256

It seen from the above figures that the rate of population increase in Surabaja city is approximately 7% per year. If it is assumed that the annual increase of population for the past four years from 1968 to 1971 continues linearly also in future, the population of Surabaja city is expected to reach roughly 3.75 million by 1990 as described in Chapter XX, Part 4.

4) Agriculture and industry.

In Indonesia, the First Five-Year Plan was started on April 1, 1969 and the basic strategy was to bring the agricultural field into focus, especially the production of food laying stress on rice. Namely, the main objective of this plan was to increase the yield of rice during these five years, so that Indonesia could gradually decrease the import of rice and, ultimately, is expected to stop it completely.

Although this plan has been gradually giving good results since 1969, something is still left to be desired.

In order to achieve the object in the field of agriculture, however, it should be supported by favourable conditions in the fields of infrastructures, i.e. high priority equal to the production of agriculture should be given to the rehabilitation and improvement of such infrastructures as roads, rivers, bridges, railroads, harbours, airports, electric power, etc. Accordingly, it may be said that the Surabaja river improvement project to be planned for the purpose of protection of flooding is also one of the most important thing in order to achieve the object.

Population engaging in agriculture sector in Indonesia is about 70% of the whole. However, most of the food crops have been being consumed in the country without being exported. Table 1 shows production of food crops in Indonesia for the last six years from 1965 to 1970. As seen in the table, the growth of the production of irrigated paddy is the steadiest one.

Productions of staple food crops in East Java and Surabaja city areas are shown in Tables 2 and 3. As obvious from Tables 1, 2 and 3, unit yields per ha of paddy in East Java and Surabaja city areas are by far larger compared with that of the whole Indonesia. This fact shows that the present project area is high by situated in the agricultural sector in Indonesia.

Such paddy production in the present area has been supported by efforts of Bimas and Inmas programs, which respectively covers the acreage of 367,000 ha and 150,000 ha out of total planted area of 1,193,000 ha in East Java province. As a result, the yield of paddy in East Java province in 1970 amounted to approximately 4,388,000 ton in dry stalked paddy exceeding the target yield by about 1,000,000 ton.

On the other hand, harvested area of paddy in the Surabaja city area in 1970 was roughly 10,000 ha for both the rainy and dry season paddy. Out of the harvested area, irrigated area was nearly 6,700 ha as described in detail in this Chapter later on. However, the existing irrigation area will be converted step by step into urban area in proportion to the development of Surabaja city. According to our study as explained in Chapter II, Part 3, it is expected that the existing irrigation acreage in the Surabaja city area will be reduced to nearly 6,000 ha by 1990. The detail breakdown of agricultural production in East Java and Surabaja city are shown in Table 5 through 17 of Chapter XXIV, Part 4.

As mentioned above, although the agricultural production has been chosen as the most important objective of the development in the country, it is also being recognized that the expansion of industrial output in both the short and long terms is an urgent necessity. And the target in the long term is to achieve a balanced growth among the agricultural, industrial and service sectors. Although the efforts of the government and industrial enterprises are some of the most important prerequisites to the future economic progress, it can be said that rehabilitation and improvement works of infrastructure will also be to promote the development of industries, because the construction works may stimulate the industrial production in compliance with demands of the construction materials. Table 4 shows productions of major industries for the last six years from 1965 to 1970.

Table 1 Production of Food Crops in Indonesia

	Irrigated paddy (dry stalked paddy)	Non-irrigated paddy (dry stalked paddy)	Maize (shelled)	Cassave (fresh-roots)	Sweet potatoes (fresh-roots)	Peanuts (shelled)	Soybeans (shelled)
1965							
Area (1,000 ha)	5,875	1,452	2,507	1,754	416	351	583
Production (1,000 ton)	14,968	2,104	2,365	12,643	2,651	244	410
Yield (ton/ha)	2.57	1.45	0.94	7.21	6.37	0.70	0.70
1966							
Area (1,000 ha)	6,011	1,600	3,778	1,513	402	388	605
Production (1,000 ton)	15,517	2,443	3,717	11,232	2,475	263	417
Yield (ton/ha)	2.58	1.53	0.98	7.42	6.16	0.68	0.69
1967							
Area (1,000 ha)	5,995	1,521	2,547	1,524	360	351	589
Production (1,000 ton)	15,303	2,095	2,369	10,747	2,144	241	416
Yield (ton/ha)	2.55	1.38	0.93	7.05	5.96	0.69	0.71
1968							
Area (1,000 ha)	6,307	1,657	3,269	1,526	390	390	676
Production (1,000 ton)	17,195	2,355	3,166	11,356	2,364	287	420
Yield (ton/ha)	2.73	1.42	0.97	7.44	6.06	0.74	0.62
1969							
Area (1,000 ha)	-	-	-	-	-	-	-
Production (1,000 ton)	18,726	2,045	2,271	10,845	1,904	257	416
Yield (ton/ha)	-	-	-	-	-	-	-
1970							
Area (1,000 ha)	-	-	-	-	-	-	-
Production (1,000 ton)	20,917	2,147	2,888	10,451	3,029	293	488
Yield (ton/ha)	-	-	-	-	-	-	-

Source: Monthly statistical bulletin, Jan. 1972, Biro Pusat Statistik, Djakarta.

Table 2 Production of Food Crops in East Java

	Rainy season paddy (dry stalked paddy)	Dry season paddy	Maize (shelled)	Cassava (fresh-roots)	Sweet potatoes (fresh-roots)	Peanuts (shelled)	Soybeans (shelled)
1965							
Area (1,000 ha)	-	-	-	-	-	-	-
Production (1,000 ton)	-	-	-	-	-	-	-
Yield (ton/ha)	-	-	-	-	-	-	-
1966							
Area (1,000 ha)	1,061	74	-	-	-	-	-
Production (1,000 ton)	3,209	122	-	-	-	-	-
Yield (ton/ha)	3.02	1.65	-	-	-	-	-
1967							
Area (1,000 ha)	1,080	71	-	-	-	-	-
Production (1,000 ton)	3,208	96	-	-	-	-	-
Yield (ton/ha)	2.97	1.34	-	-	-	-	-
1968							
Area (1,000 ha)	1,134	76	-	-	-	-	-
Production (1,000 ton)	4,279	126	-	-	-	-	-
Yield (ton/ha)	3.77	1.66	-	-	-	-	-
1969							
Area (1,000 ha)	1,155	68	-	-	-	-	-
Production (1,000 ton)	4,203	109	-	-	-	-	-
Yield (ton/ha)	3.64	1.60	-	-	-	-	-
1970							
Area (1,000 ha)	1,129	64	1,322	450	63	133	391
Production (1,000 ton)	4,278	110	8,750	2,904	331	81	222
Yield (ton/ha)	3.79	1.73	0.66	6.45	5.22	0.61	0.57
1971							
Area (1,000 ha)	1,258		1,180	461	62	135	391
Production (1,000 ton)	5,108		8,326	2,955	314	83	232
Yield (ton/ha)	4.06		0.71	6.41	5.09	0.61	0.59

Source: Dinas Pertanian Rakjat Propinsi Djawa Timur.

Table 3 Production of Food Crops in Surabaja City Area

	Rainy season paddy (dry paddy)	Dry season paddy (dry paddy)	Maize (shelled)	Cassava (fresh-roots)	Sweet potatoes (fresh-roots)	Peanuts (shelled)
1965						
Area (ha)	5,221	4,069	1,045	1,073	115	229
Production (ton)	18,605	13,408	400	4,864	426	49
Yield (ton/ha)	3.56	3.30	0.38	4.51	370	0.21
1966						
Area (ha)	5,295	4,153	1,013	931	96	292
Production (ton)	19,572	14,124	394	4,249	379	58
Yield (ton/ha)	3.70	3.40	0.39	4.56	3.95	0.20
1967						
Area (ha)	4,655	3,072	824	684	110	220
Production (ton)	15,193	11,738	322	3,039	416	48
Yield (ton/ha)	3.26	3.17	0.39	4.44	3.78	0.22
1968						
Area (ha)	4,661	3,741	1,071	1,114	235	261
Production (ton)	16,660	12,326	412	5,183	884	55
Yield (ton/ha)	3.57	3.29	0.38	4.65	3.76	0.21
1969						
Area (ha)	5,581	4,303	1,092	1,349	218	239
Production (ton)	22,565	14,828	423	6,138	825	61
Yield (ton/ha)	4.04	3.45	0.39	4.55	3.78	0.25
1970						
Area (ha)	5,744	4,137	1,023	1,500	316	242
Production (ton)	23,611	14,966	397	6,900	1,312	47
Yield (ton/ha)	4.11	3.62	0.39	4.60	4.15	0.19

Source: Dinas Pertanian Rakjat Kotamadya Surabaja.

Table 4 Industrial Production

Products	Unit	1965	1966	1967	1968	1969	1970
Urea	ton	94,000	93,000	93,000	96,000	84,000	46,000
Cement	1,000 tons	389	339	322	411	534	534
Paper	ton	11,000	10,000	9,000	11,000	16,000	17,000
Textile	million m	456	250	225	317	415	450
Cotton Yarn	1,000 bales	78	46	93	130	160	-
Accumulator	piece	33,722	31,169	21,000	28,600	32,000	17,250[1]
Radio receivers	1,000 sets	42	93	216	392	364	187[1]
Television sets	set	536	1,448	500	1,200	4,500	2,093[1]
Electric bulb	1,000 pieces	7,520	5,958	7,831	5,863	8,212	-
Electric/Telephone wire	1,000 m	1,460	250	210	572	1,000	-
Dry battery	1,000 pieces	4,167	2,554	1,210	4,377	4,500	-
Sewing machines assembling	piece	6,014	10,763	5,500	4,000	14,000	7,703[1]
Car assembling	piece	2,204	2,165	1,186	2,403	5,037	-
Motor cycle assembling	piece	-	-	805	6,247	21,388	12,365[1]
Water pump	piece	391	201	-	600	900	-
Huller	piece	794	539	-	900	2,300	-
Union & water pipe	ton	2,253	3,106	1,257	1,197	1,957	-
Galvanized iron sheet	ton	-	-	-	8,125	8,500	-
Sprayer	piece	-	-	-	5,000	20,000	-
Machines and spare parts for agriculture, plantation, mining, textile, etc.	ton	1,431	1,580	1,114	1,900	2,400	1,500[1]
Frying oil/margarine	ton	27,614	24,328	18,631	23,465	28,076	26,611
Coconut oil	1,000 ton	216	245	221	208	250	263
Soap	1,000 ton	248	207	171	130	134	133
Cigarette (machine)	million pieces	15,988	11,120	12,680	14,760	10,910	10,680
Clove cigarette	million pieces	18,593	18,717	23,165	24,000	18,844	19,237
Match	million boxes	338	266	250	238	263	269
Toothpaste	million tubes	14	13	10	13	16	15

Source: Monthly Statistical Bulletin, Jan. 1972, Biro Pusat Statistik, Djakarta.

Note: 1) Till June.

In the present project area, existing conditions of industrial factories were surveyed by the Japanese experts and their Indonesian counterparts for the purpose of the estimation of water requirement at present and in future, the study of factory properties to be used for estimation of amount of damages caused by inundation, and the study of industrial goods used for construction works.

According to our study as explained in Chapter I, Part 3, in the Surabaja city area, there are manufactories of nearly 2,800, including modern large-scale industries such as Cement Factory and Petro Kemia in Gresik. At present, although area for industrial use in the city is estimated at about 800 ha, it is expected to expand to nearly 3,000 ha by 1990, according to the report by the Team Master Plan Surabaja.

Indonesia could export maize of 250,000 tons in 1970, owing to the increase of rice production. However, rice of about 300,000 tons has been imported every year on the average, because production of major foods has not yet reached the level of self-sufficiency.

Table 5 shows records of import of rice, wheat and milk in the last six years. Further, import and export of the whole goods in Indonesia are shown in Table 6 together with those of Djakarta and Surabaja ports.

Table 5 Record of Imports of Rice, Wheat and Milk of the Whole Indonesia

Year	Rice and Glutinous rice		Wheat flour		Milk and Cream	
	Quantity	Amount (CIF)	Quantity	Amount (CIF)	Quantity	Amount (CIF)
	1,000 ton	Million $	1,000 ton	Million $	1,000 ton	Million $
1965	786.8	132.7	32.4	3.9	2.8	1.7
1966	280.5	58.0	47.3	5.0	5.1	2.6
1967	56.6	14.2	152.7	16.0	11.6	5.4
1968	485.9	96.4	367.4	38.4	12.9	6.3
1969	238.2	45.1	294.1	32.1	20.4	8.2
1970	305.7	49.0	324.7	29.6	24.5	9.2

Table 6 Import and Export

unit: 1,000

	Import	Export		
	1969	1968	1969	1970
Whole Indonesia	3,354.4	750.2	-	-
Tandjung Priok	1,625.8	132.4	173.1	153.3
Surabaja	472.7	397.0	540.7	639.3

5) Prices of commodities.

Indonesian economy in 1966 was under especially unfourable conditions and ultra-inflation got rampant in the country. This is seen from the below index numbers of market prices for 62 commodities in Djakarta, which rose to 650% in 1966 compared with the preceding year.

Rate of Increase of Index Numbers of Market Prices for 62 Commodities in Djakarta, 1966-1970

Year	1966	1967	1968	1969	1970
Increase in %	650	120	65	10	10

However, since 1966, desperate efforts to reestablish and improve the economy have been made by the government with cooperation of the people and also these efforts have been supported by experts in various fields. As a result of these efforts, rate of increase of index number went down to 10% in 1969

Further, market prices of principal food articles which are most closely connected with daily life of the people have also been stabilized as seen in Table 7 as well as those of 62 commodities described above.

In Table 7, market price of rice in Java is estimated at about Rp 40 per kg on the average for the four years from 1968 to 1971. According to Dinas Pertanian Rakjat Kotamadya Surabaja, market price of rice in Surabaja city and its surrounding areas in 1971 is also Rp 40 per kg. But price of dry stalked paddy at farmer level corresponding to market price of rice is Rp 16 or 17 per kg as shown in price list of agricultural products of Table 18 in Chapter XXIV, Part 4.

Table 7 Average market prices of 12 food articles in Java

unit: Rp

Articles	Unit	1965	1966	1967[1]	1968	1969	1970	1971
Rice	kg	583	5,412	15.49	42.36	36.49	42.55	40.28
Maize (shelled)	kg	267	2,660	7.26	19.11	20.31	19.60	20.60
Cassava (roots)	kg	59	1,220	2.84	7.26	6.31	8.08	7.60
Sweet potatoes	kg	72	1,341	2.96	7.40	6.84	8.52	8.60
Peanuts(shelled)	kg	684	8,342	25.66	58.84	73.61	83.81	85.00
Soybeans (mixed)	kg	549	5,792	16.82	38.09	53.18	52.72	58.89
Buffalo (meat)	kg	2,103	24,071	56.67	128.57	175.61	214.99	273.14
Dry salted fish	kg	1,205	15,101	45.50	90.30	115.73	120.44	136.67
Hen's eggs	unit	96	1,212	3.16	6.96	9.42	10.86	11.23
Coconut ripe (unsmelled)	unit	199	1,953	8.16	15.18	18.54	18.55	23.72
Coconut oil	700cc	1,002	9,560	35.21	71.48	85.12	87.38	93.90
Salt briquette	1/2kg	53	894	3.02	4.83	10.24	11.11	9.48

Note: 1) Figures after 1967 are showed by New Rupiah.

Source: Monthly Statistical Bulletin, Jan. 1972, Biro Pusat Statistik, Djakarta.

As described above, consumer price has been stabilized since 1967, and also production of agricultural foods has been remarkably increased. As a result of these facts, the Gross National Product (GNP) of Indonesia has considerably grown up as shown in Table 8. The GNP per person in 1972 is reported to reach nearly US$ 100. There is no doubt that such growth of GNP has been brought by the desperate efforts of the government officials and the people.

Table 8 Gross National Product (GNP) per Person

Uunit: US$

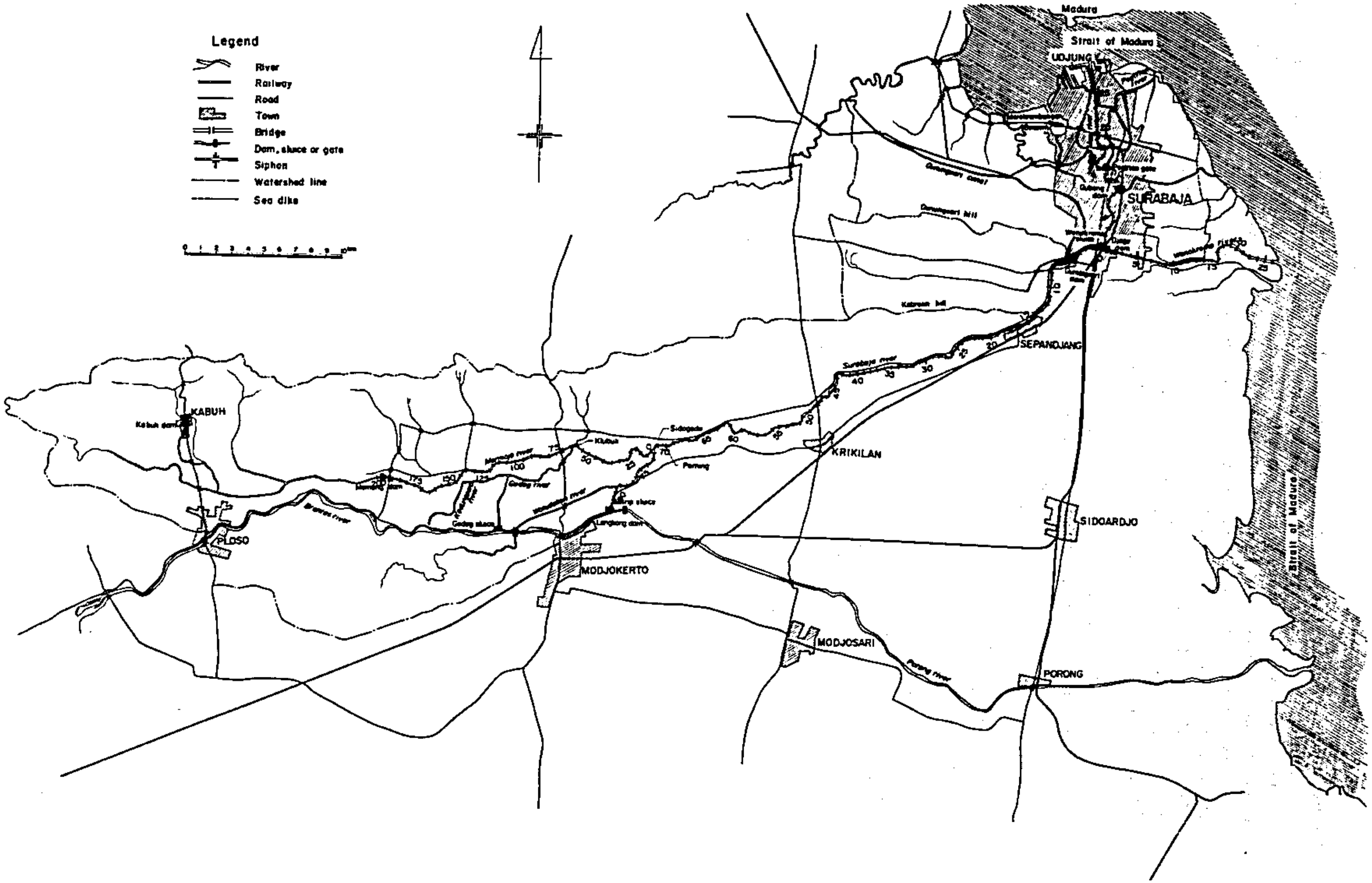
Item	1966	1967	1968	1969	1970
GNP	7.5	19.9	45.2	57.3	68.8

(2) River system in the project area.

The present Surabaja river, originating in the Marmojo-river basin and gathering the water diverted from the Brantas river through Mlirip and Gedeg sluice and the run-off from residual basins, flows through Surabaja city, the second big town in Indonesia and pours into the Strait of Madura, devided into the two rivers, Wonokromo canal and the Mas river. The river has a total catchment area of 604.4 km^2, a total length of 100 km, river-bed slope ranging from 1/300 in the upper basin of the Marmojo to 1/4200 near the river mouth, and has about 1.9 million inhabitants in the drainage basin including Surabaja city.

The whole basin is shown in Fig. 1. In the upstream part of the Marmojo-river basin, we find Kabuh dam, which has a drainage area of 28.2 km^2 and has been serving as an intake for irrigation water. In the middle reaches of the Marmojo river, about 31.9 km downstream from the top of the river, we find Mernung dam, which has a drainage area of 155.1 km^2 including Kabuh-dam's

Fig. I The Surabaja River



one and also has been serving as an intake for irrigation water. Gathering the runoff from the residual basin, the Marmojo river unites, at Klubuk, the Gedeg river which diverges from the Brantas river through Gedeg sluice and unites the Kedungsoro river before joining the Marmojo. Gedeg sluice usually serves as an intake for irrigation water, but, in the rainy season, this sluice has been used to serving as a spillway of the Brantas river. At Klubuk about 14.0 km downstream from Mernung dam, the Marmojo river has a drainage area of 277.1 km^2. Going downwards, the Marmojo river joins the Surabaja river at Sidogede about 6.2 km downstream from Klubuk.

The Surabaja river diverges from the Brantas river through Mlirip sluice at Mlirip and, uniting, at Wonosari, the Watudakon river which collects the water of a drainage area of 99.4 km^2 and crosses the Brantas river with Watudakon syphon, joins with the Marmojo river at Sidogede, where the total drainage area of the Surabaja river amounts to 419.3 km^2. At Perning about 1.1 km downstream from Sidogede, the drainage area of the Surabaja river amounts to 332.3 km^2.

About 32.2 km downstream from Perning, we find Gunungsari dam which has been serving for several tens of years exclusively as an intake for irrigation water. The catchment area of the Surabaja river ends at this point amounting to 604.4 km^2. Going downwards about 2.5 km without any addition of catchment area, at a point called Djagir, the Surabaja river bifurcates into Wonokromo canal or the Wonokromo river and the Mas river. However, the flood flow of the Surabaja river is usually discharged into the Wonokromo river, eventually pouring into the Strait of Madura.

At the bifurcation of the Mas river and the Wonokromo river, there exist Wonokromo sluice and Djagir dam, both of which, dumming up the water of the Surabaja river, serve as facilities for taking the water for drinking and other use.

Going further downwards along the Mas river about 4.3 km downstream from Wonokromo sluice, we find Gubeng dam. This dam has been serving for many years as facilities not only for taking irrigation water but also for keeping ground water higher than in the area downstream of the dam.

At Ngemplak about 2.1 km downstream from Gubeng dam, the Pegirian river diverges from the Mas river. At this biburcation of the Pegirian river, a gate called Pegirian gate is installed together with Ngemplak pumping station. This gate and pump are used for purification of the Pegirian river which, uniting Tambakwedi drainage canal at a point just upstream of Tambakwedi sluice, pours into the Strait of Madura.

On the other hand, there exists a sea dike along the north-eastern coast of the city over a length of about 17 km from Udjung to the Wonokromo river. This dike has been serving for years for prevention of flood and salt intrusion from the sea.

Storm water in the city area downstream from Gunungsari dam is usually drained by several drainage canals including the Surabaja, Mas, and Pegirian river. Of these drainage canals, the Mas river runs through the middle of the city and several pumping stations have been installed along its course, and the Greges river, running through the densely populated area of the city, serves for storm-water drainage and pours into the strait through a regulation pond called Morokrembangan Boezem. Storm water which comes from Gunungsari hill is received temporarily by Gunungsari canal which is usually used for irrigation and then, through this canal or across it, drains into the Surabaja river or the above-mentioned drainage canals.

Surabaja city which has a population of about 1.6 million as of 1971 over its municipal area of about 290 km^2. In the rainy season, the city is always menaced not only by inundation caused by local heavy rainfall, but also by both floods, from the Surabaja river and from the sea, especially by superannuation of flood-control facilities such as sluices, dams and pumps.

Beside this, the lower part of the Marmojo basin suffers from habitual inundation which sometimes reaches over 2,000 ha of rice field.

Moreover, in connection with the above-mentioned, problems of rehabilitation or improvement of urban drainage and sewerage systems as well as irrigation and drainage systems can be raised.

For convenience's sake when we mention the rivers and hydraulic structures in the succeeding chapters, the designation given in Fig. 2 is shown here.

2. Menace of Flood in Surabaja City and Its Hinterland.

The flood flow of the Surabaja river results from the run-off from its own drainage basin including the Marmojo river as the main, as well as inflow of the flood water of the Brantas river through Mlirip and Gedeg sluice. Surabaja city and its hinterland have been manaced by such flood flows and besides Surabaja city has very often been suffered from local inundations caused by local heavy rainfall and aggravation of local drainage. At the same time, they are always menaced by intrusion of sea into low-lying area.

(1) Floodings in the drainage basin of the Marmojo river.

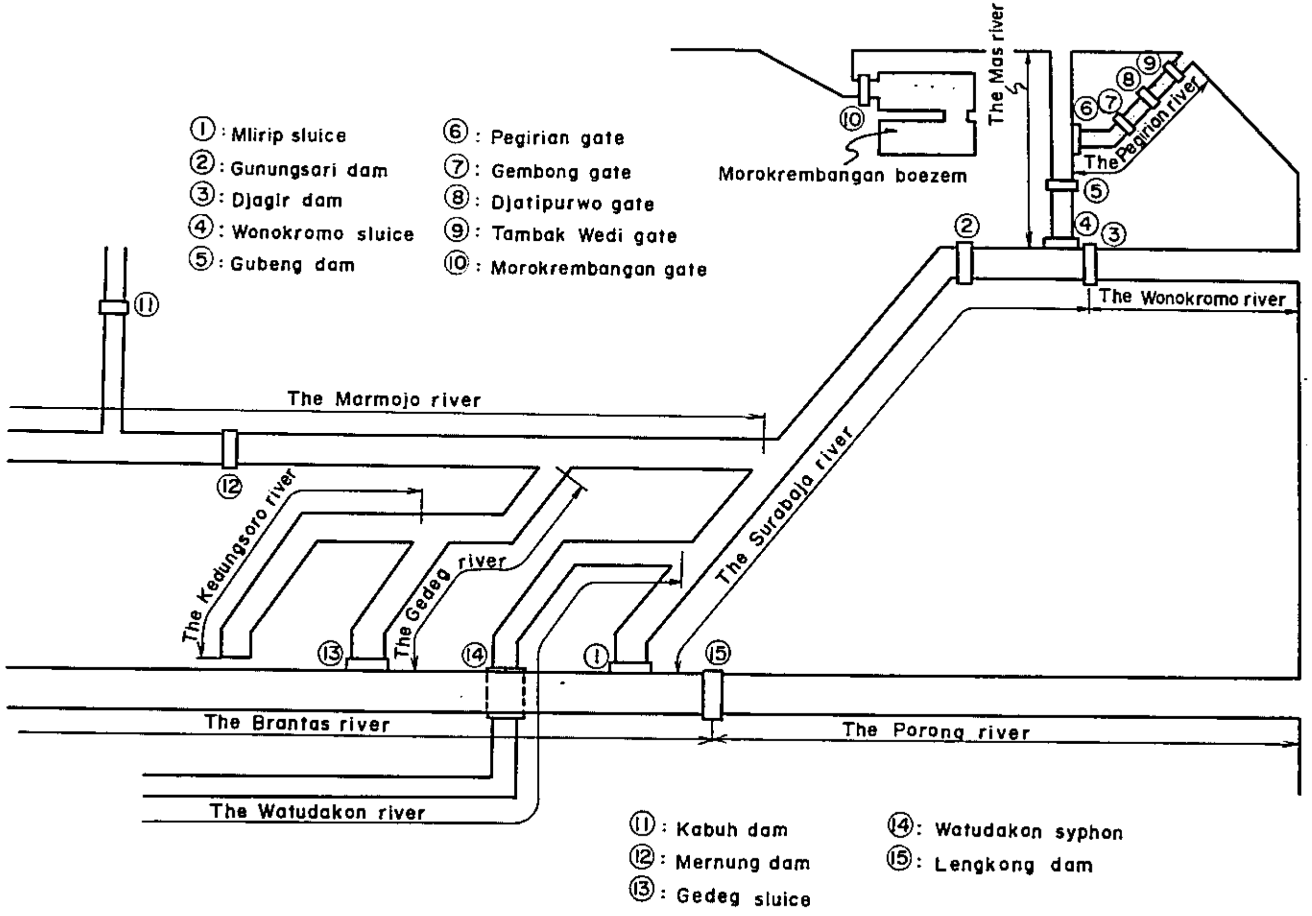
In the rainy season which usually sets in in November and continues upto next May, flood water of the Brantas river is very often poured into the Gedeg river through Gedeg sluice as well as into the Surabaja river through Mlirip sluice, which results in heightening the water level not only in the channels of the Gedeg and the Kedungsoro river, but also at the confluence of the Surabaja and the Marmojo river. Owing to backwater from the confluence, water level on the lower Marmojo is kept raised over a long time, which makes the drainage of the sorrounding area almost impossible and causes serious inundation in the inner basins in a form of impounded rainfall and/or backflow water from the Marmojo river. It is reported that the inundation continues over one or two months in some basins on the occasion of big flood.

(2) Floods of the Surabaja river.

The major structures on the Surabaja river such as dams and sluices have all already been superannuated and the function of their operation has been extremely depressed. Therefore, the operation cannot follow rising of water stage on the occasion of rapidly swelling floods, which has every possibility of destruction of structures or overtopping the river banks. As the case may be, it might be taken that flood flow, once caused by such unfavourable matters, would intrude into the central part of Surabaja city.

Furthermore, the both banks of the Surabaja river are so severely scoured that it frequently causes collapses of banks. Some of them have already reached the main body of the dikes. This has every possibility to

Fig. 2 Designation of Rivers and Hydraulic Structures



lead to collapse of dike.

(3) Flooding in Surabaja City.

Catchment area of Surabaja city may be classified into two zones, agricultural area and urban area. Each zone has different patern of its flood damage and cause of flooding.

Floods have caused physical damage to buildings and their contents, bridges, roads, railways, etc. in the urban area and crop losses in agricultural area. Furthermore, other losses such as cost of flood fighting and evacuation from flooding area, and loss of income due to interruption of business have also been caused by the floodings. Present and past conditions of the floodings and some considerations on the system are discussed in the following paragragraphs.

1) Urban area.

On the occasion of local heavy rainfall, frequent floodings occur in several places of the urban area of Surabaja city owing to the aggravation of conditions of drainage channels and pumps. Especially, the facilities for urban drainage are considered to be insufficient for the runoff from Gunungsari hill which is surrounded by Gunungsari canal on the urban side. Such runoff has every possibility to overtop the canal resulting in pouring into the urban area, which might cause terrible inundation in the urban area.

The drainage system in the city area, as mentioned above, has inadequate sections to run off all the storm water to the discharge points causing inundations of streets, walks and yards, and flooding of other low-lying structures, together with attendant inconvenience, traffic disruption and damage to property. These conditions are further worsen due to flatness of ground surface slope which makes it difficult to design the channels to give faster velocity of flow, and consequently their capacity is reduced.

The inundations have been concentrated in densely populated areas where depressed water was sometimes detained over few days causing a substantial amount of damage. This damage was heavy especially in the areas near Djl.Kedungdoro, R.A. Kartini and Djl. Indrapura. Inundations in these areas have been created mainly due to insufficient cross sections of existing channels which are reportedly to have been silted up gradually by settled substances over the last few decades. Channel beds have so far elevated by more than 50 cm from they used to be. Naturally, most tributary area of these channels have become impossible or difficult to discharge the rain water by gravity to these channels during heavy rainfalls.

These conditions caused deep inundations of more than one metre at Djl. Kedungdoro and gave the heavy damage to the property of this area. Other areas, west to the Mas river near Djl. Raja Darmo and Djl. Diponegoro, are also a frequently-flooded area and have suffered from inundations during the past few decades. In these areas, the inundations were caused by not only rainfall over the area but also by water overflowed from Gunungsari canal at several points between Wonosari Kidul and Kembang Kuning, where protective dike are provided insufficiently in its height to sustain the high water elevations during the heavy rainfalls.

In order to relieve these areas from the frequent floodings, the city authority has been devoting her emphasis to improvement programme by dredging the river beds, banking and paving dikes and replacing old pumps to new ones. However, the recent rapid urbanization in the areas has made this improvement programme difficult to catch up with its tempo and complaint from the inhabitants in the areas are flushing to the city.

2) Farming area.

The drainage system of the agricultural land has not been rearranged and completed yet, so that inundation in agricultural land is seen in some places during every rainy season.

All the drainage canals and outlet structures have not sufficient capacity to meet runoff discharge at the site, and the deterioration of the canals has been progressing. The water of some drainage canals is also used for the living of the people in the downstream area by checking up the flow. Heavy rainfall, high tide and the poor drainage system are the causes of inundation in the area and give a lot of losses in crop production. Countermeasures against inundation should be studied in connection with the improvement of irrigation systems.

3. Devastation of Urban Drainage and Sewerage Systems.

(1) Drainage system.

1) General.

Surabaja city area has thirteen major drainage areas which generally collect and carry rainfall runoff water across the city area for final discharge to the sea. The thirteen drainage canals are listed in Table 1 and illustrated on Fig. 3.

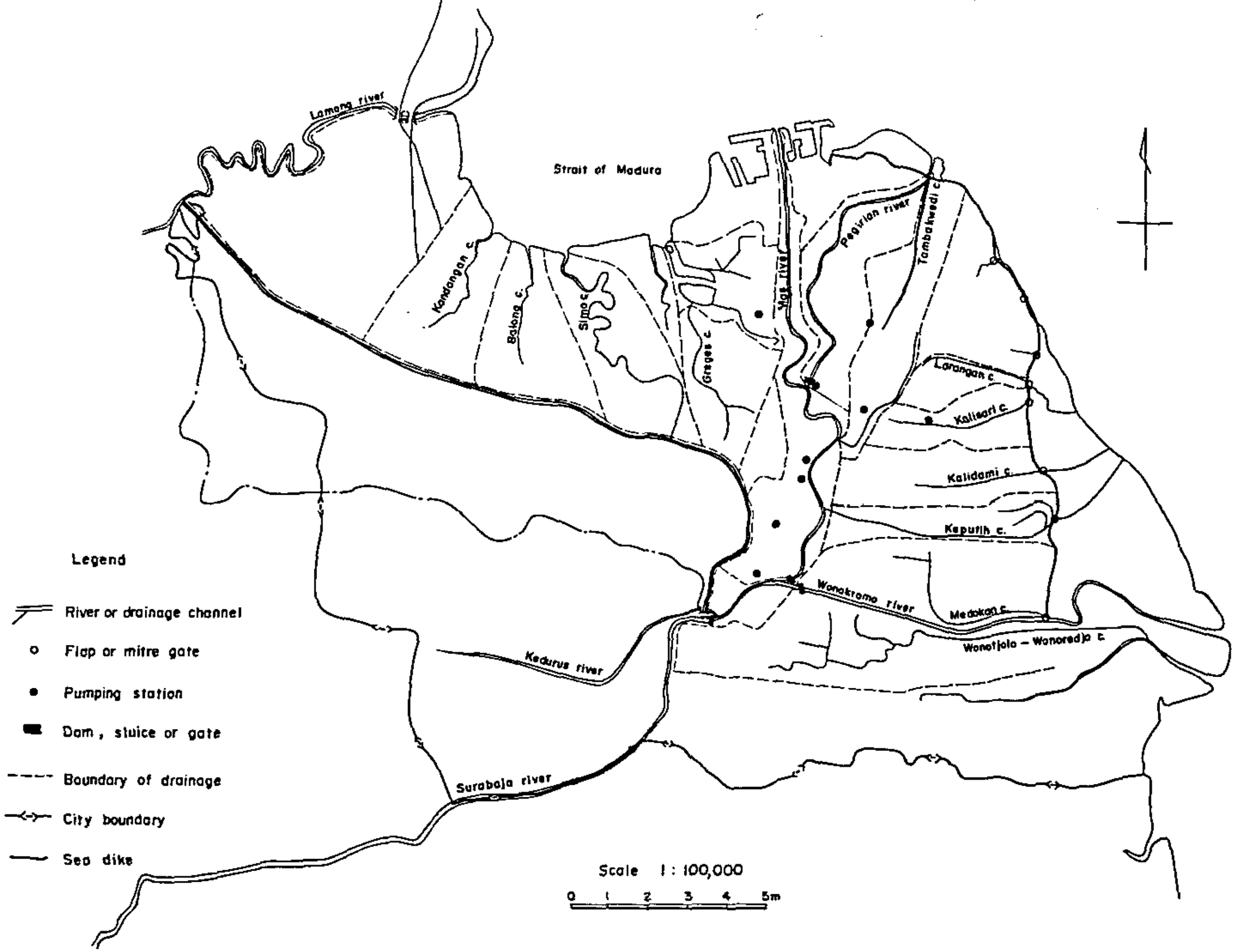
Table 1 Major Drainage Canals in the City

Name of canal	Drainage area (km^2)
Kandangan	6.6
Balong	6.4
Simo	7.0
Greges	15.8
Mas	11.9
Pegirian	9.6
Tambakwedi	9.8
Larangan	2.8
Kalisari	4.7
Kalidami	7.5
Keputih	5.5
Medokan	11.5
Wonotjolo-Wonoredjo	15.6

All the drainage systems, except the Pegirian, the Greges and the Mas river, are flowing mostly through undeveloped areas and are being used for both irrigation and drainage purposes.

In urban area, storm water from streets and households is usually collected by small ditches or conduits and discharged to main drains. In some areas, however, flooding has frequently been observed during rainy

Fig. 3 Drainage Systems in Surabaja City



seasons. This is mainly due to the flatness of ground surface and low ground elevation. When water surface of trunk channels rises, discharge of the rain water from depressed area has become impossible without the aid of pumping facilities.

In order to relief these low-lying areas from the frequent floodings, nine drainage pumping stations namely; Ngemplak, Gunungsari, Darmo, Kupang, Pesapen, Kalikepiting, Simolawang, Keputran, and Darmohusodo, as marked on Fig. 3, have been constructed and operated since 1910s. Their function is to lift the storm water from the tributary areas either directly or through outfalls to main drains. Details of the pumping stations will be mentioned later. Each pumping station is provided with pump house, pumps, manually-operated bar screens and other necessary appurtenances. Pumps are operated by crew, who are on double shift, when water surface elevation in pump well has been raised and reached to the point, over which level runoff water in low-lying areas in tributary may be depressed. It is observed, however, that even though the pumping capacity has been considerably increased recently, their strengthened capacity was not fully demonstrated. This is probably because most branch and lateral drains are insufficient to carry the rainfall runoff together with low-runoff coefficient of tributary thus reducing the total rainfall runoff to the pumping station. As discussed in a succeeding section, pumping stations have been constructed since 1910s and some of the pumping facilities are considerably superannuated and rehabilitation and replacement would be necessary.

The present major drainage systems in urban area are as follows:

2) The Pegirian river.

This river serves as the main drainage system for the north-eastern part of the city which lies between the railway and water-shed line which exist along the roads of Djl. Raja Hang Tuah, K.H. Mas Mansjur, Pengampon Klm, and Grogal Kalimir, connecting to the diversion from the Mas river near Ngemplak Bridge.

Starting at the diversion, the Pegirian river flows to the north along Djl. Undaan, Pengampon, Bungran, Njamplungan etc., toward Djl. Wonosari Lor and then turns its direction to the east until it reaches to the confluence with the Tambakwedi canal.

It has number of small branches and laterals within the tributary, but most of them have insufficient cross sectional areas to discharge the amount of runoff during rain storms.

The tributary area covers the most densely populated districts in the city, with an averaged population density of some 200 persons per hectare ranging from 100 to as high as 1400 persons per ha (after the Team Master Plan Surabaja) and the total population served by this system is assumed to be 190,000.

At present, no modern sewerage system is provided in the area, and the most wastes and refuses discharged from the area finally inflow to the Pegirian river. This condition produces septicity and emanating strong odours especially during low-flows, creating undesirable esthetic and public health problems.

The municipality of Surabaja has engaged in the improvement programme of the river since several years ago and some measures have been carried out in order to relief the prevailing deteriorated conditions. Among the measures are the construction of Ngemplak pumping station and dredging of silt in the river bed in 1971 and 1972.

Purpose of the construction of pumping station is to pump a reasonable amount of water of the Mas river to the Pegirian river aiming to reduction of the strong odours from the Pegirian, and if possible, to give the river water enough capacity of self-purification. This station may be operated from time to time when the flow rate of the river has become low and odours are offensive. After the completion of these works, the river condition is said to have been improved to some extent.

3) The Greges river.

This river is serving for western part of the city area. The tributary area covers Dupak, Sawahan and some part of Sumakuwagean. This river starts at Kembangkuning near the Gunungsari canal and flowing to the westernly direction. Then at Simo Sido Muljo, near old river bed which used to connect with the Simo river, it changes the direction to the north, finally inflowing to Morokrembangan Boezem.

In the tributary area inundations have often been caused and gave considerable damage to inhabitants' property and public facilities. This damage was especially heavy in the area near Djl. Kedungdoro, R.A. Kartini and Djl. Indrapura, causing deep inundation at some places of more than one metre in depth, leaving the area depressed over a few days. This condition is mainly due to insufficient cross sectional area of channels together with flatness of bed slope.

One of the most important channels of this area flows from Djl. Indrapura to westward crossing under the railway at Kemajoran finally emptying to the Greges river. Upper reach of this channel has almost been silted up since the last decade, especially in a conduit which crosses the railway, and all the water used to flow through this channel finds its way to northward discharging finally to another crossing channel near Parangklitik. This condition may give unexpected quantity of wastes to this point thus causing frequent floodings in this area.

4) The Mas river.

This river serves the most important districts covering the central part of the city ranging from residential, commercial, industrial and institutional areas of nearly 1,380 ha. This river starts from the bifurcation of the Surabaja river and finally discharges its water to the sea.

Water level of this river is influenced by the tide and makes it difficult to discharge storm runoff from low-lying area in tributary, and three pumping stations, Darmo, Kupang and Keputran have been constructed and operated since 1910s. Capacity of these pumping stations, as stated in a succeeding section, has been considerably reduced due to defacement of pump impeller, and recently some rehabilitation works were executed adding several new pumps.

Most floodings in the tributary occured in the area which is surrounded by Djl. Raja Darmo, Kembang kuning, Padmosusastro, Aditiawarman and Djl. R.A. Kartini, to where water of the Gunungsari canal occasionally overflow when runoff exceeds its capacity. During the rain storm, Darmo pumping station is operated to lift the runoff, however, the lifted water sometimes spill out to the ground from an outfall of the station, because of reduced cross sectional area of the outfall channel at some places, thus causing the recurrence of the water.

(2) Existing waste water disposal system.

There is neither modern sanitary sewerage system nor drainage system in the city. At present the storm water, mixed with the domestic sewage and industrial wastes, is discharged to drains and other available water ways either directly or, in the case of toilet wastes, after passing through septic tanks or pump wells. Only a small portion of urban persons have water-borne sanitation facilities. Excreta disposal within the city area mostly depend upon the existing drains and channels.

The existing sanitary sewerage system in the city area may be classified into two different types namely; so-called "separate system" and "cesspool system". The former is provided in Tambak Sari and Tidar areas. It has collection pipes into which waste from houses is discharged through house connections. The waste is finally conducted to a pumping pit locating at the end of the system and is discharged directly to the nearby waterway. The latter is a combination of reaching cesspool and seepage pit. This system relies its disposal of the sewage on the absorptive capacity of the soil which is of controlling importance. Deposited sludge or scum is therefore, removed at intervals between two and four years.

In many densely populated and industrialized areas in the city, there is a rapid deterioration of the water quality and becoming less agreeable year by year. There is a very persistent public demand for pollution control in the city area. It is quite clear that to bring such control into effect requires completely modern sewerage system. However, under the circumstances, the discussions on the water quality control in the rivers, therefore, may have to be concentrated only on the temporary relief measures in this report.

4. Devastation of Sea Dike and Flap Gates.

From the northern part to the eastern part in Surabaja city, a continuous sea dike is built over a distance of about 17 km. The body of dike and its flap gates have been so deteriorated that almost all of the gate leaves cannot move due to corrosion by the sea and devastated body of dike has been frequently eroded by high tides which always accompany waves. Such critical condition has every possibility of collapse of dike which might inevitably cause the flood of sea in the low-lying area of Surabaja city.

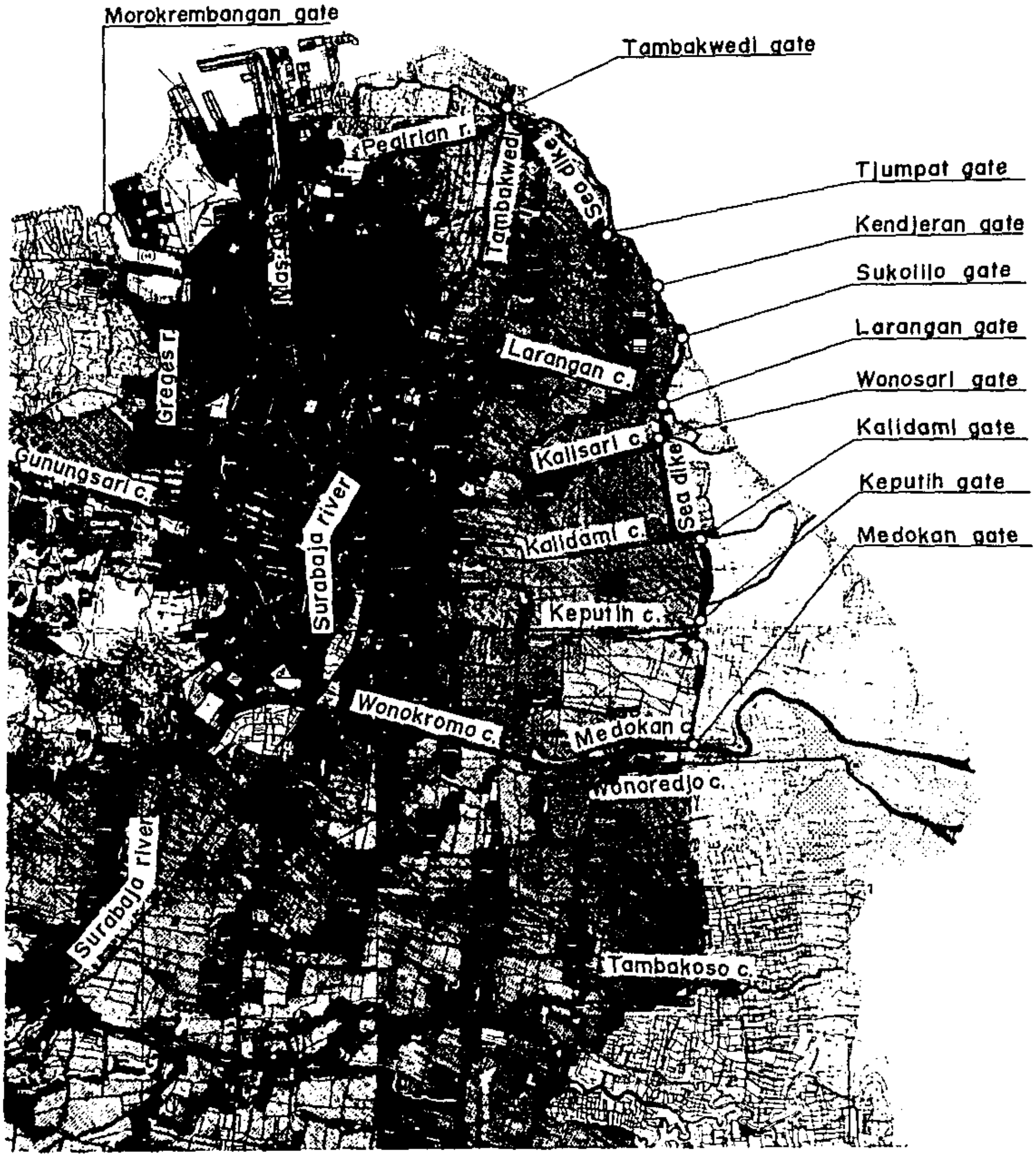
5. Irrigation Systems.

(1) General.

Irrigation in the city area is all of gravity system and its major water are taken from the Surabaja river.

There are two irrigation systems of Gunungsari and Gubeng. Major irrigation facilities are Gunungsari dam and Gubeng dam which were constructed at the beginning of this century with the purpose of regulation of water surface for irrigation and navigation.

Fig.4 Sea Dike and Flap Gates



Both facilities are truly overaged and man-power operation of needles are not effective for controlling of water surface.

Especially in large runoff season, the treatment of needles are very dangerous and setting-on and taking-out works of them are not easy. This unfavorable condition results in great fluctuation of irrigation-water intake to be supplied. The operation of the dam should be improved to a mechanized modern one.

In addition to the above, there is one pump irrigation system in the upstream area. The name is Grompol pump system and this system is used for a supplemental irrigation to the dry season crop only. One of two pumps is broken at present. Farmer is waiting for its repairs.

The related irrigation canals are mostly silted up and side slopes of them are eroded. Some turnouts, checks and water-measuring devices are broken or deteriorated. Also they may have much losses of irrigation water.

Operation and maintenance of the irrigation facilities in the city area is under the jurisdiction of Wonokromo section office of East Java Irrigation Service and the Crompol system belongs to the control of Sidoardjo Section Office of East Java Irrigation Service.

In order to increase agricultural production, a rehabilitation and improvement work of irrigation system should be programmed and carried out.

Even if the city planning has such prospect as the whole irrigation area will be completely changed into urban area by 1990, the present condition of irrigation system should be rehabilitated because the farmer who is actually living on the land will use the system.

According to our investigation, it was found that decrease of irrigation area is only 102 hectares in 8 years, therefore such prospect as by the city planning team is very doubtful about its realization.

We would like to point out the following important items to be considered as the foundation for the development of the city;

a. Dispersion of misgiving about the flood and inundation by strengthening of sea dike and rearrangement of drainage systems.

b. Establishment of necessary water supply systems for irrigation, living, drinking and factory use.

c. Enlargement of electric supply system.

d. Construction of city-road net and

e. Upgrowth of standard living level.

Therefore, the city planning should be studied carefully in cooperation with the experts in respective field of speciality. From this point of view, a study group of agriculture sector should be added to the city planning team.

(2) Irrigation area.

Irrigation area (6,956 ha) under the command of the Surabaja river (downstream of Mlirip sluice) is grouped into three systems as follows;

a. Grompol pump irrigation area:
Area is 227 ha and pump irrigation isn't for rainy season crop but for dry season crop.

b. Gunungsari dam gravity irrigation system:
Right bank irrigation area of the river is totaled to 2,089 ha and there are four intakes, and irrigation is for rainy season paddy as well as dry season crop. The irrigation block is as below:

Irrigation Block

No.	Name	Hectare	Field Elevation (m)
W-1	Simowau	387	4.00 - 1.5
W-2	Kebonagung	1,511	4.00 - 0.0
W-3	Djambangan	62	4.00 - 3.5
W-4	Karah	129	4.00 - 1.0
		Total : 2,089	

Left bank irrigation area of the river is totaled to 1,723 ha and five intakes are provided at upstream of the dam. The purpose of irrigation is for rainy season paddy as well as dry season crop. The irrigation block is as below:

No.	Name	Hectare	Field Elevation (m)
W-5	Powowijung	430	4.00 - 2.5
W-6	Gunungsari	1,293	2.00 - 0.5
		Total : 1,723	

Total command area of the dam is 3,812 ha.

c. Gubeng dam gravity irrigation system:
Irrigation area is totaled to 2,917 ha in the right bank area only of the river, and irrigation is for rainy season paddy and for dry season crop. There are two intakes and irrigation block is as below:

Irrigation Block

No.	Name	Hectare	Field Elevation (m)
W-7	Kalibokor	1,109	1.00 - 0.0
W-8	Djeblokan	1,808	0.50 - 0.0
		Total : 2,917	

The location of the irrigation block has been illustrated in the attached figure and yearly change of the irrigation area for the last eight years is tabulated as below:

Irrigation Area of the Surabaja River Project

(unit: hectare)

Irrigation Block	*1964*	*1965*	*1966*	*1967*	*1968*	*1969*	*1970*	*1971*
Wonokromo Area								
W-1 Simowau	404	404	387	387	387	387	387	387
W-2 Kebonagung	1,520	1,520	1,511	1,511	1,511	1,511	1,511	1,511
W-3 Djambangan	62	62	62	62	62	62	62	62
W-4 Karah	129	129	129	129	129	129	129	129
W-5 Rowowijung	430	430	430	430	430	430	430	430
W-6 Gunungsari	1,319	1,319	1,319	1,319	1,319	1,319	1,293	1,293
W-7 Kalibokor	1,143	1,143	1,143	1,129	1,129	1,129	1,109	1,109
W-8 Djeblokan	1,824	1,824	1,824	1,824	1,808	1,808	1,808	1,808
Sub-total :	6,831	6,831	6,805	6,791	6,775	6,775	6,729	6,729
Sidoardjo Area								
S-1 Grompol	227	227	227	227	227	227	227	227
Total :	7,058	7,058	7,032	7,018	7,002	7,002	6,956	6,956
Decreased Area	-	0	26	14	16	0	46	0

Above figures are as of January in each year.

The general map of irrigation area is given in Fig. 5 and the diagram of irrigation system is given in Fig. 6.

(3) Irrigation facilities.

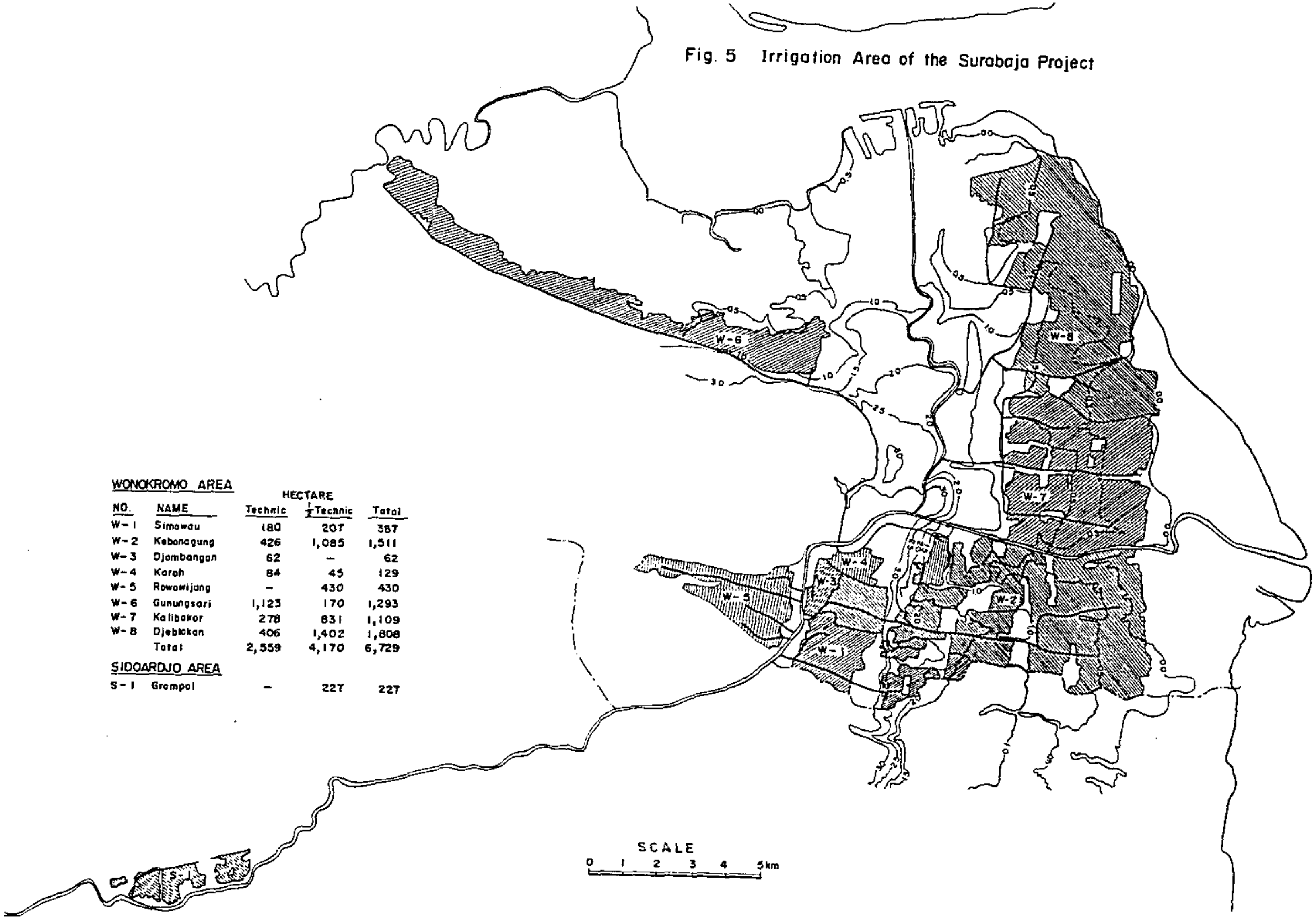
The Gunungsari dam and the Gubeng dam which are principal structures of the irrigation area, have been utilized for long since the beginning of the 20th century. The operation system of the structures is all by manpower and very difficult to retain the necessary water level for irrigation, especially in rainy season against the change of flow. It takes too much time to set and take out needle bars. These unfavorable conditions have resulted in the difficulties of effective water supply and shortage of irrigation water on the area concerned.

It seems that the two diversion dams are overaged and a modernized operation is requested.

Existing irrigation canals are not wellmaintained, a lot of sediment and dust in irrigation canals interrupt the flow of irrigation water. Repair works on deteriorated canal, turnout and check structure are essencially required. Water measuring divices should be provided at the diversion sites. It is judged that all the irrigation canal system should be checked and rehabilitated.

Especially, the Gunungsari canal is meandering along the foot of the hill and all the water from hill side slope is being drained into the canal, after that, excess water is discharging to the main drainage canals (K. Simo, K. Balong, and K. Kandangan) through the canal spillways provided at the point of intersection.

Fig. 5 Irrigation Area of the Surabaja Project



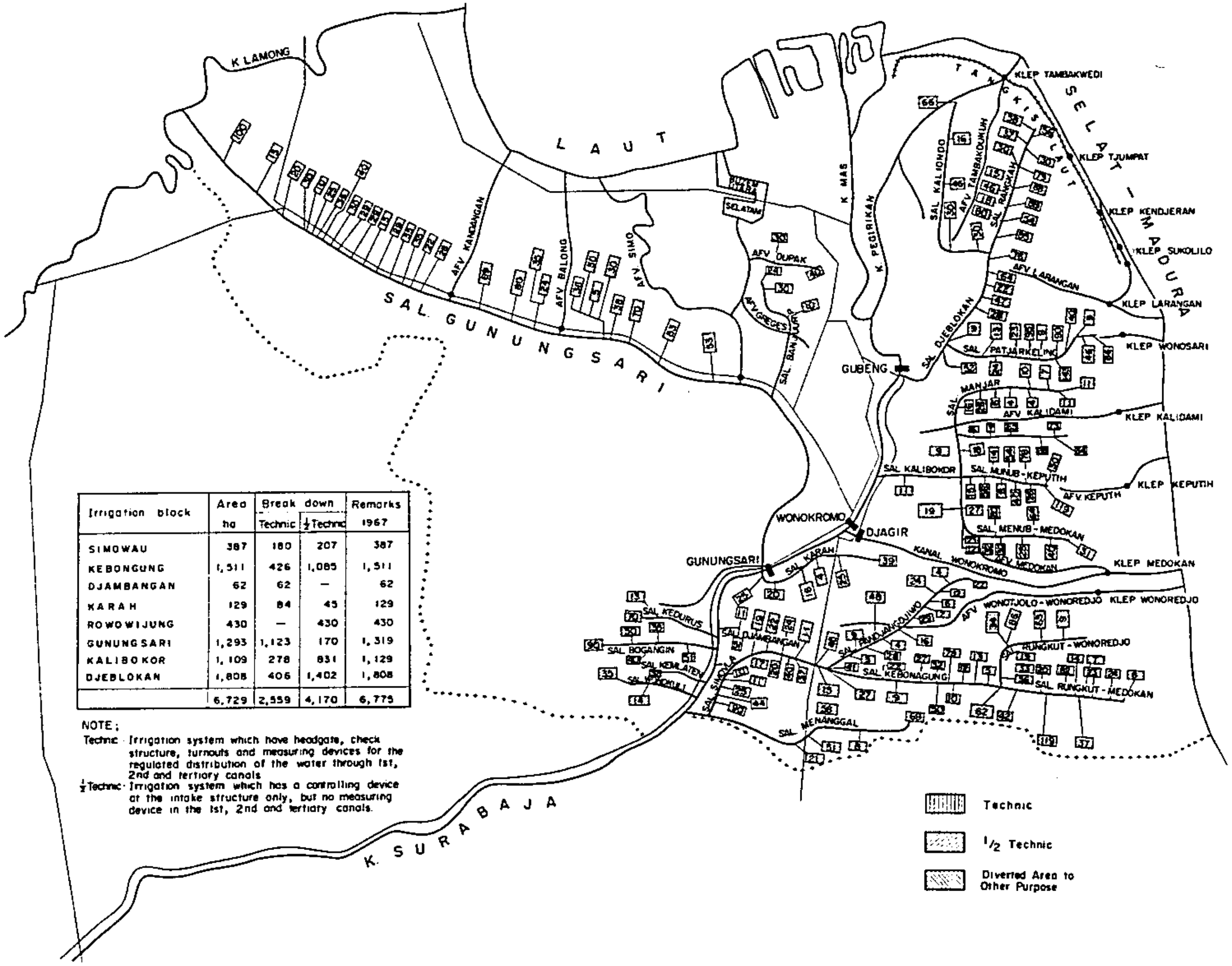
WONOKROMO AREA

NO.	NAME	HECTARE Technic	½ Technic	Total
W-1	Simowau	180	207	387
W-2	Kebonagung	426	1,085	1,511
W-3	Djombangan	62	–	62
W-4	Karah	84	45	129
W-5	Rowowijung	–	430	430
W-6	Gunungsari	1,123	170	1,293
W-7	Kalibokor	278	831	1,109
W-8	Djebiokan	406	1,402	1,808
	Total	2,559	4,170	6,729

SIDOARDJO AREA

NO.	NAME	Technic	½ Technic	Total
S-1	Grompol	–	227	227

Fig. 6 Irrigation System in the Surabaja City Area



Irrigation block	Area ha	Break down Technic	Break down ½ Technic	Remarks 1967
SIMOWAU	387	180	207	387
KEBONGUNG	1,511	426	1,085	1,511
DJAMBANGAN	62	62	—	62
KARAH	129	84	45	129
ROWOWIJUNG	430	—	430	430
GUNUNGSARI	1,293	1,123	170	1,319
KALIBOKOR	1,109	278	831	1,129
DJEBLOKAN	1,808	406	1,402	1,808
	6,729	2,559	4,170	6,775

NOTE;
Technic : Irrigation system which have headgate, check structure, turnouts and measuring devices for the regulated distribution of the water through 1st, 2nd and tertiary canals.
½ Technic : Irrigation system which has a controlling device at the intake structure only, but no measuring device in the 1st, 2nd and tertiary canals.

Capacity of the canal and the spillways is not sufficient to the runoff water from the developing area on the hill side slope, therefore the separation of the purpose (irrigation and drainage) is the most important subject for the Gunungsari canal system.

On the other hand, the Gubeng canal system is passing through the center of the city and much waste materials from the housing area are depositing in the canal, particularly in the upstream section. It will be a problem whether the water quality of the canal flow is suitable or not for irrigation through out a year.

Separation of the canal functions (irrigation and sewerage) should be taken into consideration for the Gubeng irrigation system.

Dimensions of major irrigation facilities are summarized as follows:

a. S-1 : Grompol Irrigation Block

Grompol pumping station

Type of pump	: Centrifugal pump
Diameter of pump	: 20 cm
Lifting water requirement	: 47 lit/sec, 2 units
Total head	: 5 m
Prime mover	: Diesel engine, 2 units 21 Hp, 275 rpm

b. W-1 : Simowau Irrigation Block

1) Intake : Stop-log and chain rolling up system, manpower operation

Structure	: Concrete		
Width of inlet	: 1.65 m and 1.70 m		
Width of pier	: 1.50 m		
Height of wall	: 3.30 m		
Elevation (SHVP)	:	Upstream (m)	Downstream (m)
Concrete bed		3.684	3.254
Water surface		5.150	4.987

2) Canal

Name	Length (m)	Width Top (m)	Width Bottom (m)	Height (m)	Structure
Sal. Simowau	1,400	6.10	5.10	0.51	Earth
Sal. Menanggal	4,800				
Upstream		5.10	4.20	0.34	Earth
Downstream		3.70	2.75	0.25	Earth

c. W-2 : Kebonagung Irrigation Block

1) Intake : Stop-log and chain rolling up system, manpower operation

Structure	: Concrete
Width of inlet	: 3.60 m and 3.70 m
Width of pier	: 1.20 m
Height of wall	: 3.10 m

Elevation (SHVP) :	Upstream (m)	Downstream (m)
Concrete bed	4.037	4.080
Water surface	5.207	4.560

2) Canal

Name	Length (m)	Width Top (m)	Width Bottom (m)	Height (m)	Structure
Sal. Kebonagung	9,000				
Upstream		7.70	5.70	0.78	Earth
Middlestream		7.15	5.00	0.81	Earth
Downstream		5.75	3.20	0.32	Earth
Sal. Pandjangdjiwo	3,600	4.95	4.00	0.44	Earth
Sal. Rungkut-Medokan	2,500				
Sal. Rungkut-Wonoredjo	2,300				

d. W-3 : Djambangan Irrigation Block

1) Intake : Slide gate

Structure : Concrete
Width of inlet : 1.20 m
Height of wall : 3.20 m

Elevation (SHVP) :	Upstream (m)	Downstream (m)
Concrete bed	3.631	3.421
Water surface	5.093	4.702

2) Canal

Name	Length (m)	Width Top (m)	Width Bottom (m)	Height (m)	Structure
Sal. Djambangan	440				
Upstream		4.20	2.50	0.62	Earth
Downstream		4.50	3.10	0.65	Earth

e. W-4 : Karah Irrigation Block

1) Intake : Stop-log and chain rolling up system, manpower operation

Structure : Concrete
Width of inlet : 2.70 m
Height of wall : 2.20 m

Elevation (SHVP) :	Upstream (m)
Concrete bed	3.380
Water surface	4.630

2) Canal

Name	Length (m)	Width Top (m)	Width Bottom (m)	Height (m)	Structure
Sal. Karah	3,300				
Upstream		6.10	5.00	0.52	Earth
Middlestream		4.85	4.00	0.21	Earth
Downstream		2.00	0.60	0.45	Earth

f. W-5 : Rowowijung Irrigation Block

1) Intake

Name	Structure	Inlet Wall Width (m)	Inlet Wall Height (m)	Elevation Bed Ups. (m)	Elevation Bed Dos. (m)	Elevation Water Surface Ups. (m)	Elevation Water Surface Dos. (m)
Wonokuli	Flashboard	0.75	2.25	3.465	3.465	5.285	4.975
Kemlaten	Flashboard	0.70	1.70	3.846	4.160	5.226	4.938
Bogangin	Flashboard	0.77	2.00	3.715	4.235	4.550	4.580
Kedurus	Flashboard	0.70	1.30	3.590	2.452	4.780	4.742

2) Canal

Name	Length (m)	Width Top (m)	Width Bottom (m)	Height (m)	Structure
Sal. Wonokuli	100				
Upstream		3.30	3.20	0.37	Earth
Downstream		2.20	2.10	0.20	Earth
Sal. Kemlaten	50				
Upstream		4.50	1.95	0.27	Earth
Downstream		2.80	2.40	0.35	Earth
Sal. Bogangin	200				
Upstream		3.95	3.40	0.44	Earth
Downstream		2.45	2.30	0.37	Earth
Sal. Kedurus	50				
Upstream		4.50	2.70	0.21	Earth
Downstream		2.20	1.70	0.18	Earth

g. W-6 : Gunungsari Irrigation Block

1) Intake : Slide gate, manpower operation

Structure : Concrete, sysphon in succession of 32 m long

Width of inlet : 2.00 m

Height of wall : 4.55 m

Elevation (SHVP)	Inlet (m)	Outlet (m)
Concrete bed	0.20	-0.20
Water surface	4.910	4.852

2) Canal

Name	Length (m)	Width Top (m)	Width Bottom (m)	Height (m)	Structure
Sal. Gunungsari	21,133				
Upstream		8.00	5.50	1.30	Earth
Middlestream		6.50	3.10	1.70	Earth
Downstream		6.00	1.50	1.80	Earth

h. W-7 : Kalibokor Irrigation Block

1) Intake : Stop-log and chain rolling up system, manpower operation

Structure : Concrete

Width of inlet : 3.25 and 3.25 m

Width of pier	: 1.00 m		
Height of wall	: 2.90 m		
Elevation (SHVP)	:	Upstream (m)	Downstream (m)
Concrete bed		0.888	0.444
Water surface		2.228	2.040

2) Canal

Name	Length (m)	Width Top (m)	Width Bottom (m)	Height (m)	Structure
Sal. Kalibokor	1,800	9.50	5.40	0.90	Earth
Sal. Menur-Keputih	2,500				
Upstream		5.60	4.60	0.40	Earth
Downstream		5.40	5.10	0.23	Earth
Sal. Manjai	3,000	7.00	5.20	0.39	Earth
Sal. Menur-Medokan	5,200	7.35	5.50	0.34	Earth

i. W-8 : Djeblokan Irrigation Block

1) Intake : Stop-log and chain rolling up system, manpower operation

Structure	: Concrete	
Width of inlet	: 2.50 and 2.50 m	
Width of pier	: 1.00 m	
Height of wall	: 2.00 m	
Elevation (SHVP)	:	Upstream (m)
Concrete bed		0.658
Water surface		1.960

2) Canal

Name	Length (m)	Width Top (m)	Width Bottom (m)	Height (m)	Structure
Sal. Djeblokan	3,700				
Upstream		9.80	8.50	0.50	Earth
Middlestream		9.00	5.10	0.38	Earth
Downstream		8.10	5.70	0.37	Earth
Sal. Patjarkeling	3,500				
Upstream		6.20	5.00	0.32	Earth
Middlestream		5.50	4.00	0.19	Earth
Downstream		5.50	4.50	0.32	Earth
Sal. Rangkah	3,800	4.95	3.40	0.12	Earth
Sal. Kaliondo	5,000	4.70	3.50	0.18	Earth

(4) Irrigation water.

Actual water distribution records have been examined through the Daftar Pertanaman for the last 7 years at Wonokromo section office.

The figures in the records (Daftar Pertanaman) are not of the result of the direct measuring but of the estimate by the officials of the irrigation service. Since some measuring devices have been broken and deteriorated, canal capacity is not clear yet. So the distributed water is estimated by the experienced officer.

The amount of water intake is not met with the water requirements of the crops in the command area but influenced by flow conditions of the Surabaja river.

The matters mentioned above should be noted to read and consider the records. It can be pointed out from the maximum figure in the records that difficulty of water intake has been increased year by year.

Because it was 12.8 m^3/sec. in 1965 but 6.8 m^3/sec. in 1971. According to the record in 1964 the following figures have been found for the irrigation area in Surabaja city:

Maximum : Middle 10 days of February	; 12.8 m^3/sec.
Average of growing season of rainy season paddy (6 months December to May)	; 9.8 m^3/sec.
Minimum : Middle 10 days of September	; 1.7 m^3/sec.
Average of growing season of dry season crops (6 months June to November)	; 3.9 m^3/sec.

(5) Cropping.

Most of the irrigation area (about 95%) is cultivated paddy in the rainy season, but cultivation of dry season paddy is regulated because of the insufficient discharge of the Surabaja river. Regulated dry season paddy is about 25% and non regulated dry season paddy which is not assured of irrigation water supply, is about 25% to the whole irrigation area.

Maize, peanuts, pepper and green leaves (such as sawi) are cropped on the land after harvested paddy. The area of them is only about 1% to the whole irrigation area throughout a year.

(6) Farm drainage.

All the farm drainage have been done by the natural gravity system. Main drainage canals of the right bank area of the Surabaja river are flowing down to the sea from the west toward the east along the topographic slope.

In the left bank area and about a half of Djeblokan area, main drainage canals are running to the sea from the south toward the north. In order to prevent reverse flow of sea water to the upstream owing to high tide, flap gates are mostly provided at the outlet portion of the drainage canal. However, waste materials, floating materials and sediment materials are concentrating at the gate position and disturbing the natural opening or closing of the gate.

In addition to these, as the steel hinges of the gate are mostly rusted by sea wind, it may not be expected for the gate to open or close smoothly. Most of drainage canals are earth structure and the flow capacity of the canal system has been decreased by deposited sand and other waste material.

Dimensions of major drainage facilities have been summarized as follows;

Dimension of Drainage Facilities (Outlet Structure)

Name	Canal Length (km)	Catchment Area (ha)	Drainage Capacity (cu m/sec)	Gate of Outlet Width (m)	Height (m)	Number
Right bank area of the Surabaja river						
Wonotjolo-Wonoredjo	8	1,560	4	2.00	2.00	5
Medokan	4	1,150	3	2.40	1.50	2
Keputih	2.25	550	2	1.75	1.50	1
Kalidami	4.3	750	2	2.00	1.50	1
Kalisari	4	470	-	3.00	1.50	1
Larangan	3.25	280	1.5	1.50	1.50	1
Tambakwedi	4	980	4	5.00	3.15	2
Left bank area of the Surabaja river						
Kedurus	2.50	400	5	-	-	-
Greges	10.0	1,580	10	5.00	3.50	3
Simo	5	700	8.4	2.00	2.00	2
Balong	6	640	7.2	2.20	2.00	2
Kandangan	6	660	8	2.20	2.00	2

It seems that improvement of poor drainage and excavation of canal are both required and dissolution of inundation on farm land will contribute to increase of agricultural production.

(7) Inundation in agricultural land.

Inundation area in Surabaja city can be grouped into the following four areas.

1) Low elevation area below 1.0 m along the drainage canals which are pouring into the sea.

Generally, the land surface slope is about 1/4,000 to 1/5,000. It can be considered that major causes of inundation are:

a. occurence of heavy spot rainfall in the rainy season.
b. insufficient flow capacity of the drainage canals.
c. difficulty of drainage during the high tide time in one day.
d. unfavorable condition of flap gates.
e. insufficient capacity of outlet structures.

According to the answers from farmers interviewed, max. 25 cm depth and one week inundation is occuring one or two times per year, but they experienced max. 60 cm inundation in the past. These areas are under the jurisdiction of Ketjamatan Rungkut, Sukolilo and Tandes.

2) Left bank area along Gunungsari canal.

As described in the paragraph of irrigation facilities, Gunungsari canal is collecting the hill side drainage water and discharging to drainage canals through the three canal spillways.

The design capacity of the canal may be less than 3.5 m^3/sec for irrigation. In comparing with capacity, the run off from hill side area is enormously large. The catchment area is about 2.5 km^2 per km of canal length on the average.

By the reason mentioned above, the runoff water from hill side area can not flow down smoothly but inundates along the canal.

Especially, in the 5 km of upstream portion, the drainage problem of the hill side is becoming seriously because many houses have been constructed illegally by the people at the hollow land on the hill side. They are used to break the right bank of Gunungsari canal in order to prevent from the inundation of their houses, as the result the city center is suffering the damage of inundation.

These drainage problems should be solved, for example, by a solid crossing method in connection with the city drainage system.

These areas are under the jurisdiction of Ketjamatan Wonokromo, Sawahan, and Tandes.

3) The basin between Gunungsari hill (north hill) and Kebraon hill (south hill) along the Afv. Kedurus.

The area is situated in the south west of Surabaja city. The main drainage canal is Afv. Kedurus and its total length is about 20 km.

The name of the canal is also called K. Menganti, K. Djuwet, K. Koendang and Afv. Kedurus from the upstream to the downstream. Afv. Kedurus joins the Surabaja river just below Gunungsari dam.

In the flood season, the water elevation of the confluence to the Surabaja river becomes higher than inland elevation, the back water affects to the paddy field in the Towowijung area.

Sometimes inundation on the low land reaches 1 m in depth and continues 2 or 3 days. The inundation of the area has a close connection with the operation of Djagir sluice. This area is under the jurisdiction of Ketjamatan Karangpilang.

4) Low land area along Kali Lamong.

The inundation area is terminal area of the Gunungsari irrigation block. It seems that the cause of inundation is the overflow from the Kali Lamong by the lack of proper embankment. The area is under the jurisdiction of Ketjamatan Tandes.

6. Obsoleteness and Superannuation of Dams and Sluices.

(1) General.

For the purpose of development of the area downstream of Mlirip bifurcation of the Brantas river, several dams and sluices were built since the construction of Lengkong dam and Mlirip sluice in 1857 on the Porong and the Surabaja river.

According to the survey made in 1970 by the Fact Finding Mission and our present survey, the dams and sluices located downstream of Mlirip bifurcation are presumed to be built in the years as shown in the following table.

Dams and Sluices

Located Downstream of Mlirip Bifurcation

Name	River	Type	Constructed
Lengkong dam	Porong	stoplog	1857
Mlirip sluice	Surabaja	stoplog, lock	1857
Gunungsari dam	Surabaja	needle, stoplog, locks	1907
Wonokromo dam	Surabaja	stoplog, lock	1917
Djagir dam	Wonokromo canal	roller gate	1917
Gubeng dam	Surabaja	needle, stoplog, locks	±1907
Pegirian gate	Pegirian		±1930

As easily seen in this table, more than fifty years have elapsed since the construction of major dams and sluices and the structure itself remains unchanged since their original construction though some repairs were made occasionally.

On all dams and sluices except Djagir dam, stoplogs and wooden needles are installed as they were, while only Djagir dam has comparatively modern lift gates for the convenience of regulation of flood. However, these gates are also operated by man power.

For instance, as for Gunungsari dam which must pass flood flow, it takes more than 8 hours to open one gate, because about 5 minutes are needed to remove one needle by man power and one gate has 100 needles. And in case of Djagir dam, it takes about 5 hours to open one gate, because about one minute is needed to lift the gate leaf one centimeter by man power and the full-open depth is 3 m.

These dams are, therefore, not only very inconvenient for operation but also very dangerous for flood control.

The locks which have been installed on dams and sluices are also devastated and frequently used as a flushing channel for sediment in stead of locking because ship traffic is extremely decreased nowadays.

All the dams and sluices including locks are so obsolete and superannuated that some of them are even dangerous for flood control. In improving the whole Surabaja river, these dams and sluices also should be improved by adjusting them so as to harmonize with the river improvement plan.

(2) Mlirip sluice.

Stop-log weir	8.5 m	one gate	elevation of bed-sill	17.00 m
Lock	5.0 m x 30 m	one	"	"

As already described, this sluice was built in 1857 at the same time as the construction of Lengkong Dam, aiming at the water supply for irrigation and other use in Surabaja district as well as damming up the Brantas water to facilitate the water intake from Lengkong Dam.

From the view point of structure, the body of the sluice is still strong enough to be bearable with further use. However, it seems to be necessary at least to repair the grooves of stop logs and the apron downstream of the dam.

The discharge passing through this sluice is regulated by stop logs operated by man-power hoist. This will have to be improved by motorization.

(3) Gunungsari dam.

Needle weir	Movable 10 m	5 gates	El. bed-sill	1.5 m
	Fixed 10 m	4 gates	"	1.5 m
Stop-log weir	5 m	2 gates	"	1.5 m
Lock	5.5 m x 30 m	two	"	0 m

Gunungsari dam was built for the purpose of obtaining irrigation water around 1907 at the same time as the construction of Gubeng dam which exists on the Mas river.

The dam consists of nine needle gates, 10 m each in width, on the right side, two stop log gates, 5 m each in width, in the middle, and two locks, 5.5 m each in width, on the left side. Out of the nine needle gates, four on the right hand have been permanently closed by reinforced concrete needles, providing a land for houses on the right bank closely upstream of the dam.

The dam body made of brick and covered with mortar firmly maintains its form in spite of lapse of 60 years. However, the gates are all made of wood and operated by man-power hoist. As for the needles, they have no equipment to operate. All of these seem to be very difficult for operation because of superannuation.

The locks are under repairs as of March of 1972, through which a few boats occasionally pass the dam transporting sand, bamboo, or pottery which are reportedly unloaded and sold at Wonokromo.

During floods, the needles and stop logs are usually removed according to the communication which informs the discharge diverted to the Surabaja river through Mlirip sluice. However, the operation of this dam is as inconvenient as it takes 5 minutes to remove one needle and that it takes 3 hours to collect a needle once dropped in the water.

The dam is usually operated in this way. Two groups which consist of eight persons each and work in shifts of 12 hours open the five needle gates by every one-sixth of the width of each gate according to the increase of flood, that is, by every 16 out of 100 needles on one gate every run of opening the whole gates.

Since it takes 5 minutes to remove one needle of 10 cm square, it will need about 8 hours to open one gate perfectly, about 40 hours for five gates, and about 50 hours to open all gates including two stop-log gates. Further such removal work by man power is very dangerous.

Teak wood is used for stop logs and needles. The life of one stop log is reported to be as long as about 5 years and one stop log costs about Rp. 50,000. However, needles are so subject to wearing that the life is usually as along as about 2 years, hence an expenditure of Rp 300,000 per span is unavoidable every two years as one needle costs about Rp 3,000.

The intake on the left bank closely upstream of the dam is located just off the bank, so that the inlet is subject to silting. Therefore, this inlet, if possible, should be moved to the front where no dead water may occur.

In the rainy season, some of the gates are always opened to spill flood water, which will cause large velocity due to small opening and large head difference.

The inspection bridge, under repairs as of March of 1972, is about 1.3 m in width, which is unfavourable to motor traffic.

(4) Wonokromo sluice and Djagir dam.

Wonokromo sluice which has two stop log gates, 5 m in each width, and two locks, 5.5 m in each width and Djagir dam which has three Stoney gates, 10 m in each width are located at the bifurcation of the Surabaja river and Wonokromo canal and serve for flood control as well as regulation of water requirement. Wonokromo sluice serves only to discharge required water for downstream area, while excess flood water is all discharged to Wonokromo canal through Djagir dam. Both of these structures were built around 1917 and time of more than a half century is gone by. Djagir dam is the only one which has steel gate leaves on this river, neverthercless they are still operated by man-power hoists.

1) Wonokromo sluice.

Stop-log weir	5.0 m	2 gates
Lock	5.5 m x 40 m	two

Since the flood water of the Surabaja river is discharged through Djagir dam, the overflow discharge of this sluice is very little even in the rainy season. Hence, the river water downstream of this sluice is heavily contaminated.

The body of the sluice seems to be still strong enough to further use. The lock is devastated and left closed.

2) Djagir dam.

Stoney gate	10 m	three gates	El. bed-sill	-2.00 m

Truss type is adopted for the beam of this gate, on the top of which a flush board of sluice type is installed for the purpose of flushing trash.

The hoist which is driven by man power is installed in the middle of the span and can transmit its power through a shaft to both sides of the gate leaf. Wire and chain are used doubly.

In operation of the gates, seven persons are usually engaged. They consist of 2 groups which have each three persons and work in shifts of 12 hours. Lifting speed of a gate leaf is 1 cm/min, therefore it takes 3 hours to fully open one gate, which necessitates about 15 hours to open the whole gates. Accordingly it is, by all means, necessary to enable these gates to move safely and timely by motorization of operation, because no failure is permitted for such important gates which should pass the flood safely into the floodway.

In 1970, the steel structures and woodenplates of the gate leaves as well as inspection bridges of reinforced concrete were improved. Both the body and the sheds are made of concrete and the portals are also used as stairways.

Since this dam always has to keep the water upstream of the dam as high as 3.2 m SHVP for the purpose of taking water for drinking, the velocity of flow under the bottom of the gate reaches very high especially in the rainy season. This is greatly unfavorable to the condition of the downstream channel to cause severe damages on the apron or revetments when such underflow goes on for a long period.

(5) Gubeng dam.

Needle weir	Movable	10.5 m	one gate	El. bed-sill	-0.75 m
	Fixed	10.5 m	four gates	"	-0.75 m
Stop-log weir		5.16 m	two gates	"	-2.8 m
Lock	5.5 m x 49.3 m		two		-3.0 m

This dam was built around 1907 almost at the same time as Gunungsari dam and has served for the supply of irrigation water to the farm land located in the eastern part of Surabaja city. Its structure is also similar to that of Gunungsari dam. After that, Wonokromo canal was opened in order to discharge the flood water of the Surabaja river into this canal, so that the original width of the dam has become unnecessary, consequently four needle gates on the left hand were closed with reinforced concrete needles. Namely, the existing dam consists of five needle gates, 10.5 m each in width, on the left side, two stop log gates, 5.16 m in each width, in the middle and two locks, 5.5 m in each width, on the right side. Out of five needle gates, four on the left hand are permanently closed with reinforced concrete needles. The two stop-log gates are still utilized for regulation of discharge. One lock on the right hand of the two is used for sand-flash channel for Djeblokan intake located closely upstream of the dam. Along the left bank up-and downstream of the dam, cultivated land with houses stretches to the low water channel. The gates are operated principally for the purpose of holding the water level upstream of the dam. This causes a trouble against urban drainage.

The body of the dam still maintains its form of structure, but the gates and the hoists are severely superannuated and very incovenient to operate

(6) Morokrembangan gate.

Mitre gate 5.0 m three gates El. bed-sill -3.00 m

This is a very important gate located at the outlet of Morokrembangan Boezem which facilitates the drainage of the Greges-river area by making use of tidal range.

The gate is composed of three Mitre gates of 5 m in span and 4 m in depth. The body is still strong and no deformation, such as settlement and inclination, is seen. Because of heavy weight, however, the hinges of the steel gates are easily broken and moreover, as expected, the tidal part including hinges are severely corroded by sea water. It is reported, therefore, that those gates were very often taken off from the body by power of sea waves and, once they were taken off, it was very difficult to fix them as they were before. This always threatens the surrounding area with flooding from the sea as well as from the Greges river.

7. Obsoleteness and Superannuation of Drainage Pumps.

(1) General.

Since Surabaja city has developed on a low-lying land, the natural drainage due to gravity is difficult in some part of the urban area. To overcome this difficulty, pumping stations have been built since more than fifty years ago. Though partial improvement has been made, most of them are still utilized in the original form. Therefore, those pumping stations will have to be examined not only from the point of capacity but also from their obsoleteness and superannuation. The present state of existing pumps will be mentioned in the following sections.

(2) Gunungsari pumping station.

Drainage area: about 75 ha
Pumps: two units
Pump No.1: Horizontal-two-way-centrifugal pump
Diameter: D = 600 mm
Discharge: $Q = 42\ m^3/min = 0.7\ m^3/s$
Motor: Electric motor, 29.5 kw (40 hp), 965 rpm
Pump No.2: Same
Nominal total capacity: $1.4\ m^3/s$
Estimated existing capacity: about $0.5\ m^3/s$

These pumps have been installed to drain landside water to the Surabaja river. This station was built before the second war and the existing pumps are said to be diverted from military use. In the inlet channel are used two Hume pipes of 800 mm in diameter, the capacity of which is doubtful. Drainage capacity of these pumps is extremely reduced probably owing to the development of gap between the runner and the guide vane.

It is reported that this pumping station should be removed in the future, because this station always drains waste water to the Surabaja river where an intake for drinking water is located downstream not far from the outlet of the station.

(3) Darmo pumping station.

Drainage area: about 150 ha
Pumps: 4 units
Pump No.1: Horizontal-one-way-centrifugal pump
Diameter: D = 400 mm
Discharge: Q = 0.26 m^3/s
Motor: Electric motor
Pump No.2: Vertical-axial-flow pump
Diameter: D = 600 mm
Discharge: Q = 1.1 m^3/s
Motor: Electric motor
Pump No.3: Vertical-axial-flow pump
Diameter: D = 900 mm
Discharge: Q = 1.30 m^3/s
Motor: Electric motor
Pump No.4: Vertical-axial-flow pump (Ingersoll-Rand)
Diameter: D = 900 mm
Discharge: Q = 1.47 m^3/s
Motor: Electric motor, 145 kw, 417 rpm, H = 10 ft.
Nominal total capacity: 4.13 m^3/s
Estimated existing capacity: 1.5 + 1.47 = 2.97 m^3/s

This pumping station has been playing an important role in this residential district. Since pumps No.1 to No.3 were installed before the war, their drainage capacity has been extremely reduced. The reduction of No.1 and No.3's capacity has resulted from the facts that the angles of runners were reduced when they were remade in a workshop. As for No.2, considerable gap has been developed between the runner and the guide vane owing to using it without any repairs. In 1969, a pump of Ingersoll-Rand was added. It seems that this station was frequently rearranged for the purpose of strengthening the capacity. This has caused the insufficient intervals of pumps, which is likely to cause interference of flow during the suction. When the drainage capacity will be increased in the future, it may be necessary to examine the capacity of the existing suction well. Pump No.4 is well installed utilizing the outdoor space and at present regarded as the main force.

(4) Kupang pumping station.

Drainage area: about 210 ha
Pumps: 6 units
Pump No.1: Horizontal-two-way-centrifugal pump
Diameter: D = 400 mm
Discharge: Q = 0.67 m^3/s
Motor: Electric motor, 35 hp, 1000 rpm
Pump No.2 No.4: Same as above
Pump No.5: Vertical-axial-flow pump (Ingersoll-Rand)
Diameter: D = 900 mm
Discharge: Q = 1.47 m^3/s
Motor: Electric motor, 145 kw, 417 rpm, H = 10 ft
Pump No.6: Same as above

Nominal total capacity: 2.67 + 2.94 = 5.61 m^3/s
Estimated existing capacity: 1.0 + 2.94 = 3.94 m^3/s

The drainage area is considerably large and, in the rainy season, the pumps are obliged to run almost every day. Since the pumps No.1 to No.4 were installed before the war and have been used for a long time, their capacities are extremely reduced. Pumps No.5 and No.6 are of Ingersoll-Rand type and installed well utilizing a space on the right side of the old pump house, but it is a demerit that the length of the delivery pipes is very long. It is reported that the fuses of No.6 are frequently burnt out, this is also said to result from large resistance due to the length of the pipe. This pumping station is playing an important role, but its capacity is still insufficient.

(5) Keputran pumping station.

Drainage area:	about 15 ha
Pump:	1 unit
	Horizontal-one-way-centrifugal pump
	Diameter: D = 250 mm
	Discharge: Q = 0.12 m^3/s (nominal)
	Motor: Diesel engine, 30 hp, 2000 rpm

The drainage area is small, mainly Djl. Urip Sumohardjo. The pump is of Indra type made in 1965. This type is also used at Kalikepiting and Pesapen. The engine is a Samova-Perkins of England and it is reported that exchange of parts is difficult at the occasion of trouble. It seems to be desirable to install electric motor in the future. The drainage canal is snugly put in order and, in the dry season, natural drainage due to gravity is possible.

(6) Simolawang pumping station.

Drainage area:	about 90 ha
Pumps:	5 units
Pump No.1:	Horizontal-one-way-centrifugal pump
	Diameter: D = 500 mm
	Discharge: Q = 0.2 m^3/s
	Motor: Electric motor
Pump No.2:	Horizontal-two-way-centrifugal pump
	Diameter: D = 680 mm
	Discharge: Q = 0.67 m^3/s
	Motor: Electric motor
Pump No.3:	Same as above
Pump No.4:	Horizontal-two-way-centrifugal pump
	Diameter: D = 850 mm
	Discharge: Q = 1.13 m^3/s
	Motor: Electric motor
Pump No.5:	Vertical-axial-flow pump (Ingersoll-Rand)
	Diameter: D = 900 mm
	Discharge: Q = 1.47 m^3/s
	Motor: Electric motor, 145 kw, 417 rpm, H = 10 ft

Nominal total capacity: 2.67 + 1.47 = 4.14 m^3/s
Estimated existing capacity: 1.0 + 1.47 = 2.47 m^3/s

The station is built making well use of limitted land and water is being collected from two directions. A new pump of Ingersoll-Rand type has been added to the old four pumps which capacities are already extremely reduced though they have large diameters. This is due to the fact that the runners are already worn out and the gap between the runner and the guide vane is enlarged as three times as it was. The revolution of motors are also reduced.

(7) Pesapen pumping station.

Drainage area: 74 ha
Pumps: 3 units
Pump No.1: Horizontal-one-way-centrifugal pump
Diameter: $D = 400$ mm
Discharge: $Q = 0.25\ m^3/s$
Motor: Electric motor
Pump No.2: Horizontal-one-way-centrifugal pump (Indra type)
Diameter: $D = 250$ mm
Discharge: $Q = 0.12\ m^3/s$
Motor: Diesel engine
Pump No.3: Same as above
Nominal total capacity: $0.49\ m^3/s$
Estimated existing capacity: $0.12 + 0.24 = 0.36\ m^3/s$

The present station seems to have been constructed by large rearrangement of the old one and is collecting water from three directions. Under the pump house, we find a canal which enables natural drainage. When the outer stage is higher than the inner one, a gate is closed so that pump drainage is possible. Therefore, the delivery pipes are exposed on the outside of the gate. Two pumps of Indra type were recently added to the old ones.

(8) Darmohusodo pumping station.

Drainage area: about 15 ha
Pump: 1 unit
Horizontal-one-way-centrifugal pump
Diameter: $D = 250$ mm
Discharge: $Q = 0.12\ m^3/s$
Motor: Diesel engine

Drainage area is very small. The pump sucks canal water directly from earth canal and drains into the air. It is desirable from the viewpoint of maintenance to install trush screen and pump house. The pump is of Indra type and an old engine is used.

(9) Kalikepiting pumping station.

Drainage area: about 15 ha
Pump: 1 unit
Horizontal-one-way-centrifugal pump
Diameter: $D = 250$ mm
Discharge: $Q = 0.12\ m^3/s$
Motor: Diesel engine

Drainage area is small, but the suction well is put in order constructed with concrete. The screen is made of bamboo, which brings about a head difference of from 5 to 10 cm. At the time of construction, the engine (Samova-Perkins type) diverted from Kupang used to be out of order.

(10) Ngemplak pumping station.

Drainage area: Null
Pump: 1 unit
Vertical-axial-flow pump (Ingersoll-Rand)
Diameter: D = 900 mm
Discharge: $Q = 1.47\ m^3/s$
Motor: Electric motor, 145 kw, 417 rpm, H = 10 ft

This station was constructed in 1971 in order to such the fresh water of the Mas river into the Pegirian river for the purpose of dilution of extremely contaminated Pegirian water. The pump is not operated yet as of February of 1971.

8. Ground Water.

(1) General.

Water level and salinity of water of fifty-one wells were measured by the Japanese Study Team and the Indonesian Counterpart during the period from December 1971 to March 1972. The aim of the measurement is to obtain data for studying the influence of damming up of Gubeng and Djagir dam upon ground water table and the salinity.

(2) Selection of wells for measurement.

In the area shown in Fig. 7, we first located suitable sites for well measurement at the rate of one point per km^2 on a map of 1/20,000, next at the field, specified proper wells for measurement around the sites selected on the map. Furthermore, we selected principal bridges and sluices as the sites for measurement of water stage of the river for the purpose of making longitudinal profiles of river-water surface.

Five wells, L_{10}, L_{11}, L_{14}, L_{15}, L_{18} were chosen up and down-stream of Gubeng dam for the purpose of studying the damming up effect of the dam upon protection of ground water from intrusion of salinity.

(3) Result and consideration.

1) Result of measurement.

The following papes show the result of measurement of ground water table, longitudinal profile of water level of the Mas river at the time of measurement of wells, relation between tide level and ground water table, and vertical distribution of salinity concentration in well water.

2) Consideration.

i) Ground water table.

a. River-water level dammed up by Gubeng and Djagir dam seems to be serving to sustain the ground water around river channel.

b. On the left side of the Mas river, a trough of ground water table is seen and runs toward the Greges river. According to an old map in the 9th century, this trough seems to be a trace of the old Surabaja river which was used as a fairway for ships.

c. On the right side of the Mas river, Kalibokor and Djeblokan canals seem to be serving to sustain the ground water around the canals.

ii) Salinity concentration.

a. Salinity concentration of water in wells (L_{11} and L_{14}) which are located very near the Mas river upstream of Gubeng dam is lower by about 200 ppm compared with those in wells which are located downstream of Gubeng dam (L_{15} and L_{18}) and distant from the river channel (L_{10}).

b. In wells which are located downstream of Gubeng dam and in an area where ground water table is lower, salinity concentration is found to be higher especially near the bottom of well. For instance, L_{15} and L_{18}.

c. The above-mentioned two facts seem to suggest us that higher level of ground water serves to reduce salinity concentration of ground water, although it is not affirmed yet without further measurement including the one in the dry season.

Fig. 7 Contor Map of Ground Water Table

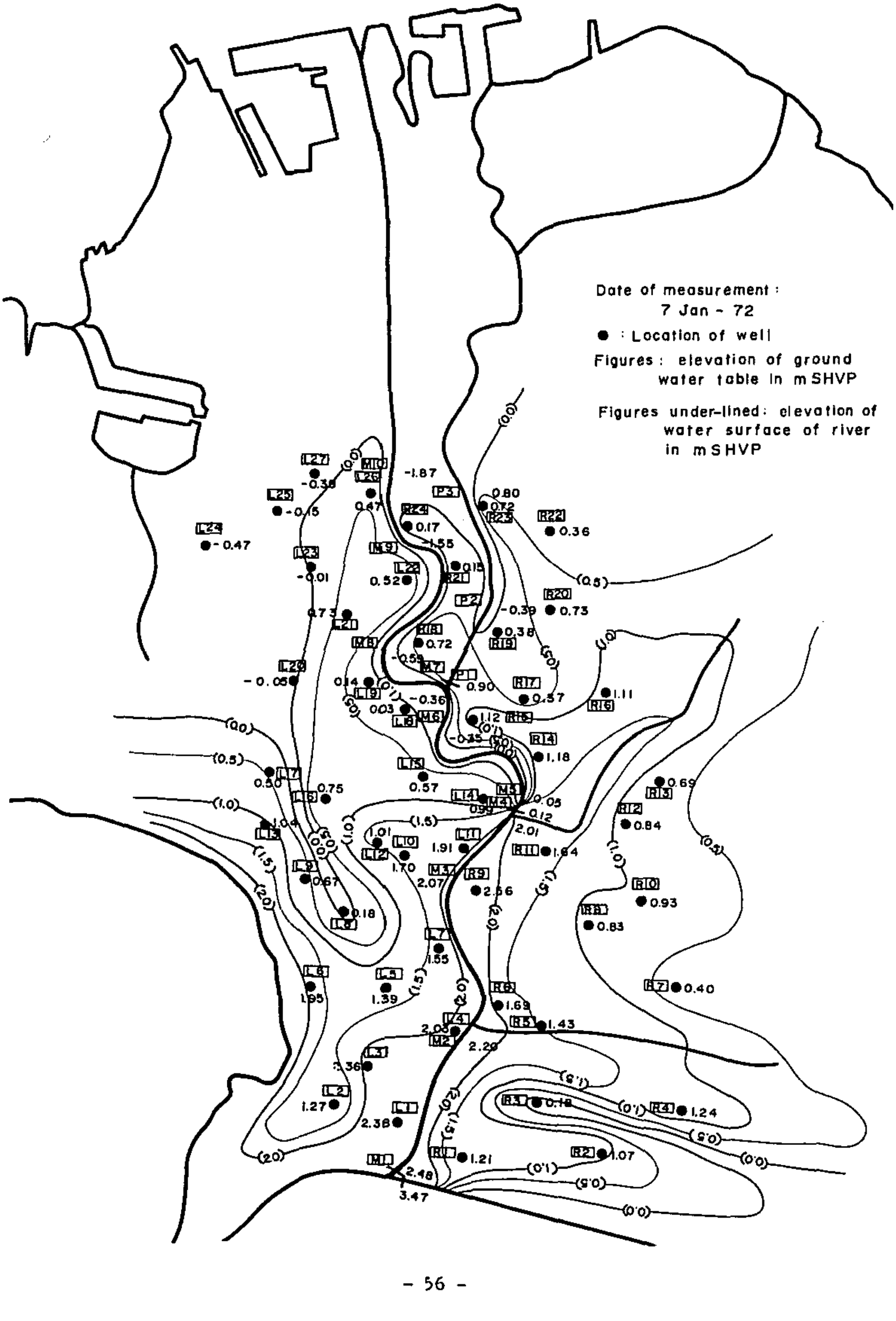


Fig. 8 Contor Map of Ground Water Table

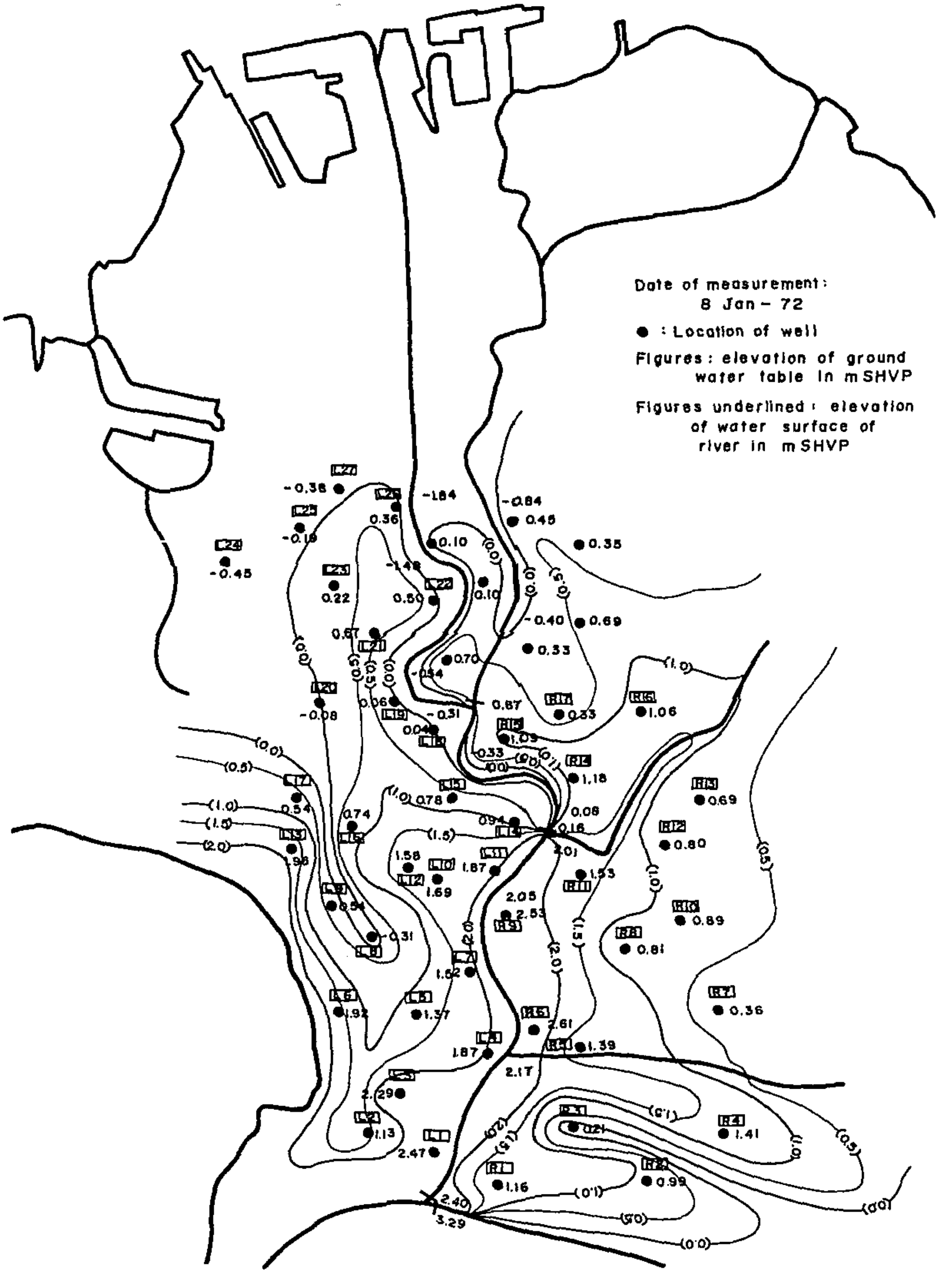


Fig. 9 Old Map of Surabaja in 9th Century

Fig.10 Profile of Water Surface of the Mas River

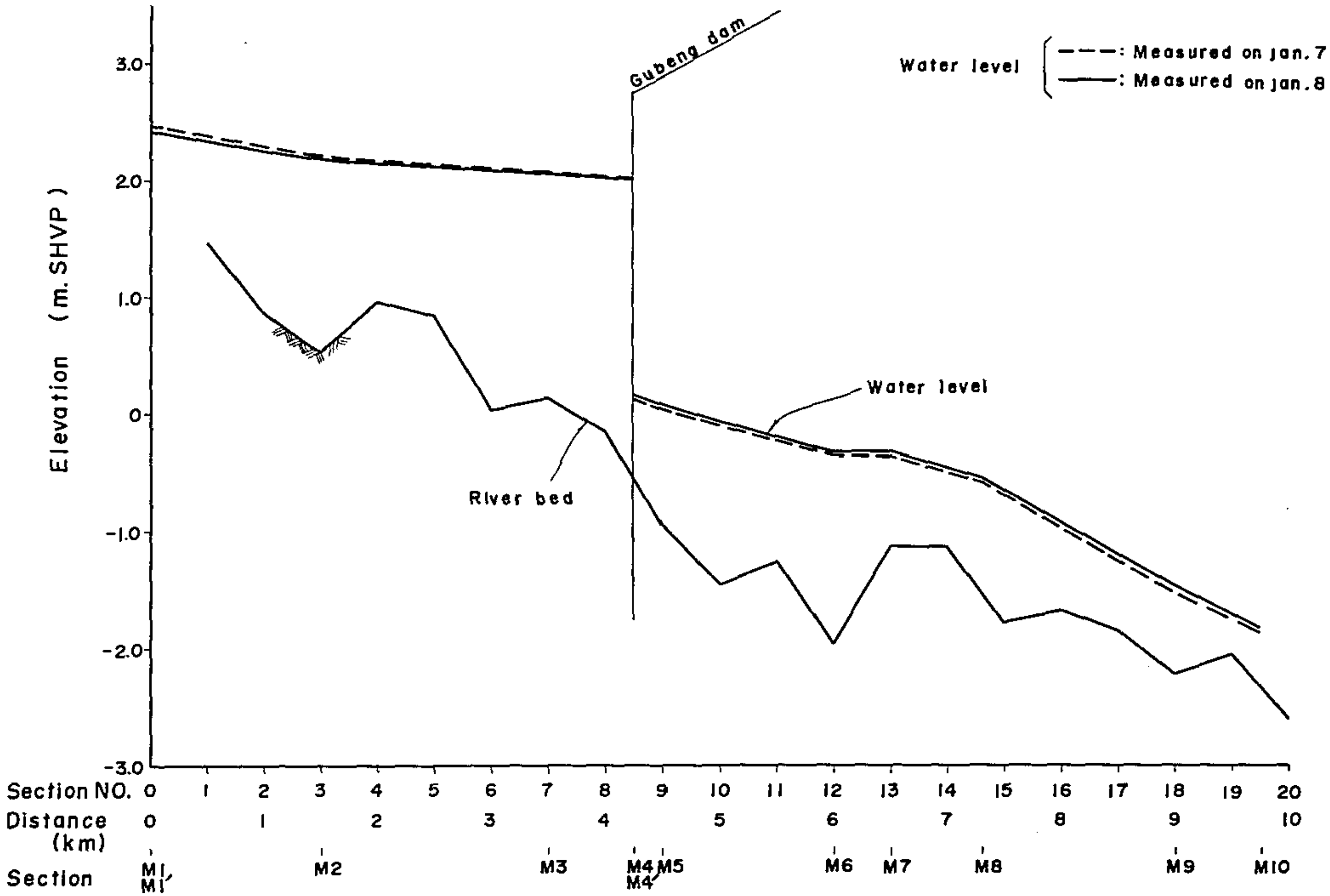


Fig. 11 Hourly Measurement of Ground-Water Table Measured on Feb. 18. 1972

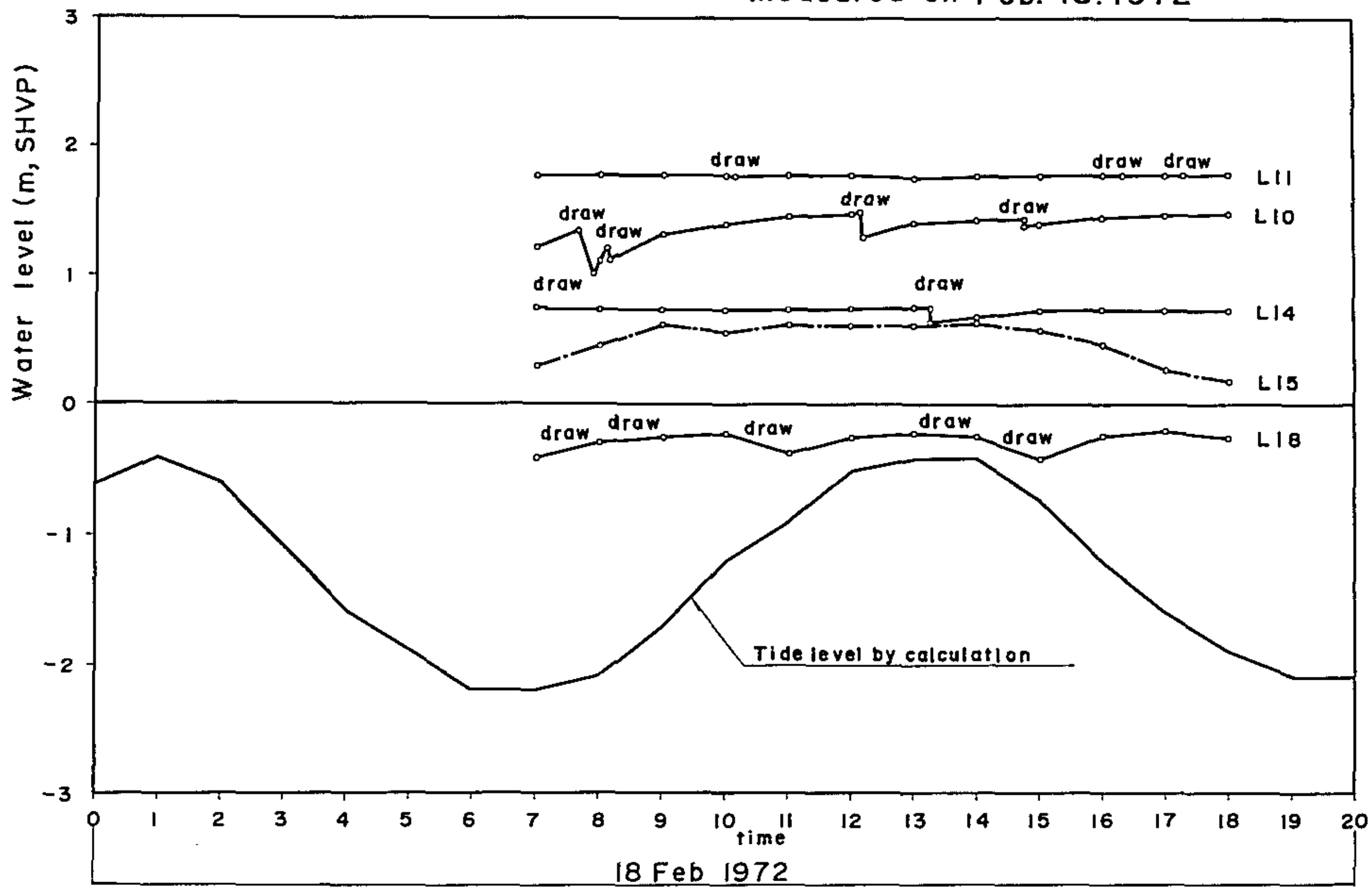
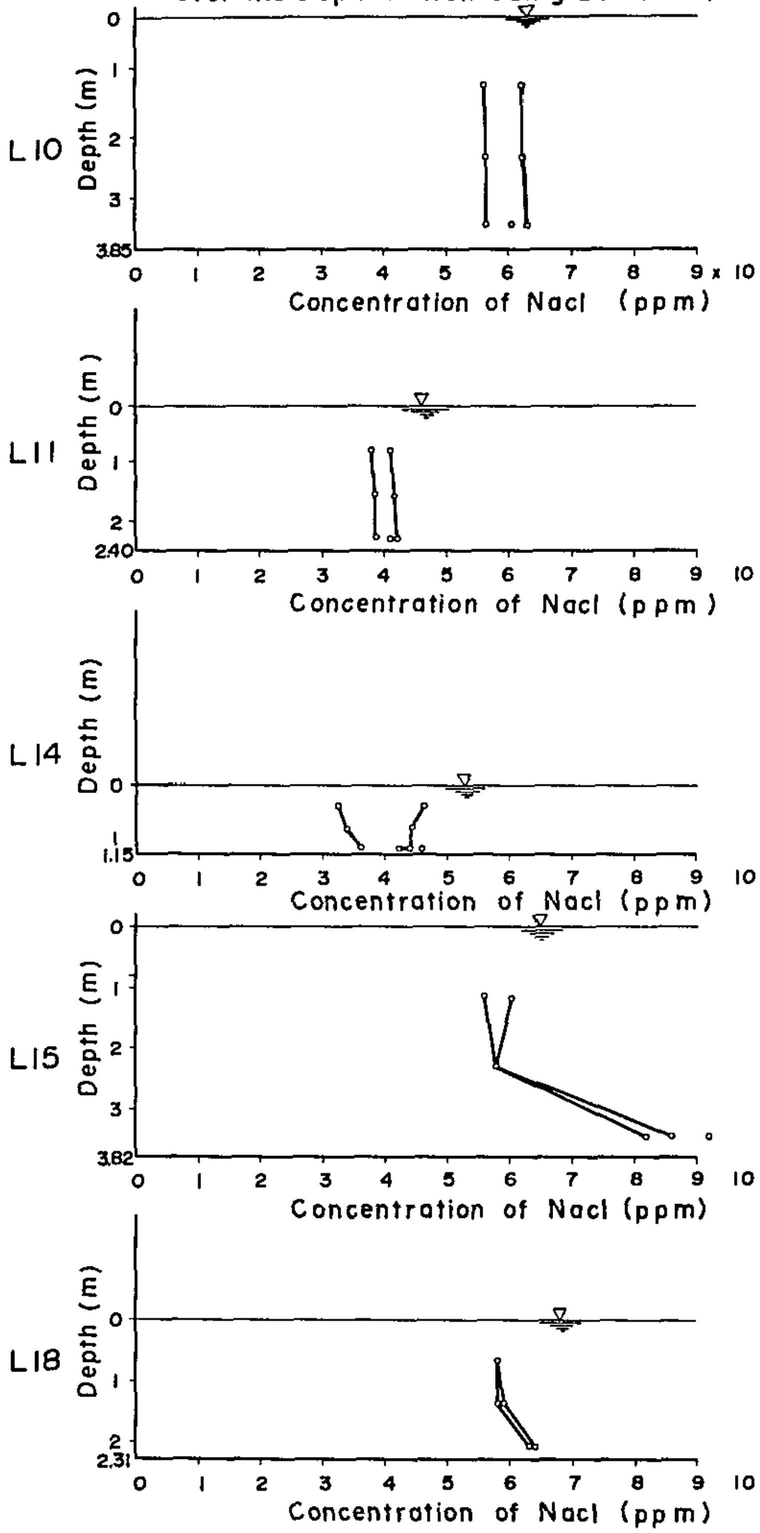


Fig. 12 Nacl Concentration (ppm) of Well Water Measured over the Depth of Well Using Electrical Conductivity Meter



CHAPTER II

MASTER PLAN

1. Improvement of Major Rivers.

(1) Basic policy of flood control.

Any flood runoff from the extensive area of the Brantas basin should not be passed through Surabaja city, the second largest town in Indonesia, but it should be led to the sea as safely and quickly as possible. The Brantas river basin survey team also has recently decided not to divert the Brantas flood into the Surabaja river through both Gedeg sluice and Mlirip sluice. This view is quite in accord with ours. The Brantas flood should be poured into the sea through the Porong river.

However, the Surabaja river still has its own basin amounting to about 600 km^2. Though the Marmojo river was originally a tributary of the Surabaja river, it should be taken as the main stem after this. The flood runoff from this basin also should not be passed through Surabaja city for safety and future development of the city. This flood flow should be poured into the sea through Wonokromo canal. In this meaning, we shall often call this river the Surabaja/Wonokromo river.

The drainage basin of the Surabaja river ends at Gunungsari dam. However, Surabaja city still has its own basin amounting to 285 km^2. Rain-water which has fallen to this basin should be drained to the sea using small rivers including the Mas river. This is, so to speak, the urban drainage.

On the north-eastern coast of Surabaja city, we have a line of sea dike which is to protect the low-lying area against the intrusion of sea water. Across this dike extending over a length of 17 km from Udjung to Wonokromo canal, some urban drainage canals including the Pegirian river pour into the sea. In this regard, the function of the sea dike and the sluices on it should be maintained soundly from the viewpoint of drainage of landside as well as prevention of sea-water intrusion.

After all, the problems boil down to keep away any flood from the Surabaja area with the view of controling flood.

Since 50-year flood has been adopted for design of the Porong river, more than 50-year flood also should be adopted for design of the Surabaja river. In this project, however, 50-year flood has been adopted for the Surabaja river and 20-year flood has been adopted for the Marmojo river. After the study mentioned in Part 4, we have decided the design-discharge distribution on the Marmojo and the Surabaja/Wonokromo river as shown in Fig. 1 and 2.

If Surabaja city will be protected against the 50-year menance of the river, the city should be also protected against the 50-year menace of the sea. In this project, therefore, the 50-year force of the sea has been adopted for the design of sea dike. However, for the design of sluices on the sea dike, the 5-year flood has been adopted. But, considering that the reimprovement may be very difficult in the future, the 10-year flood has been adopted for the design of the Mas river which streches from Wonokromo sluice to Udjung.

Fig. 1 Design Discharge of the Marmojo River
(20-year flood)

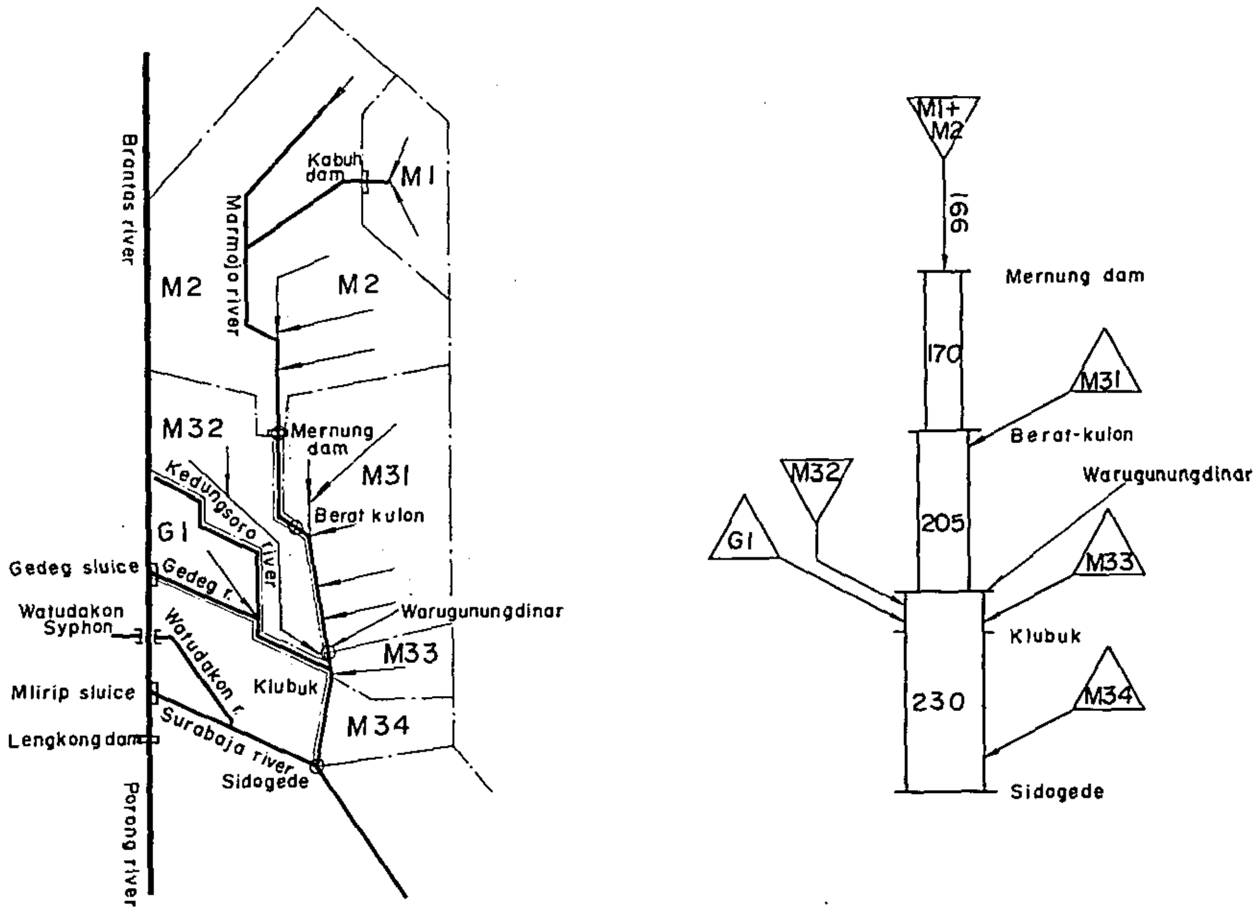
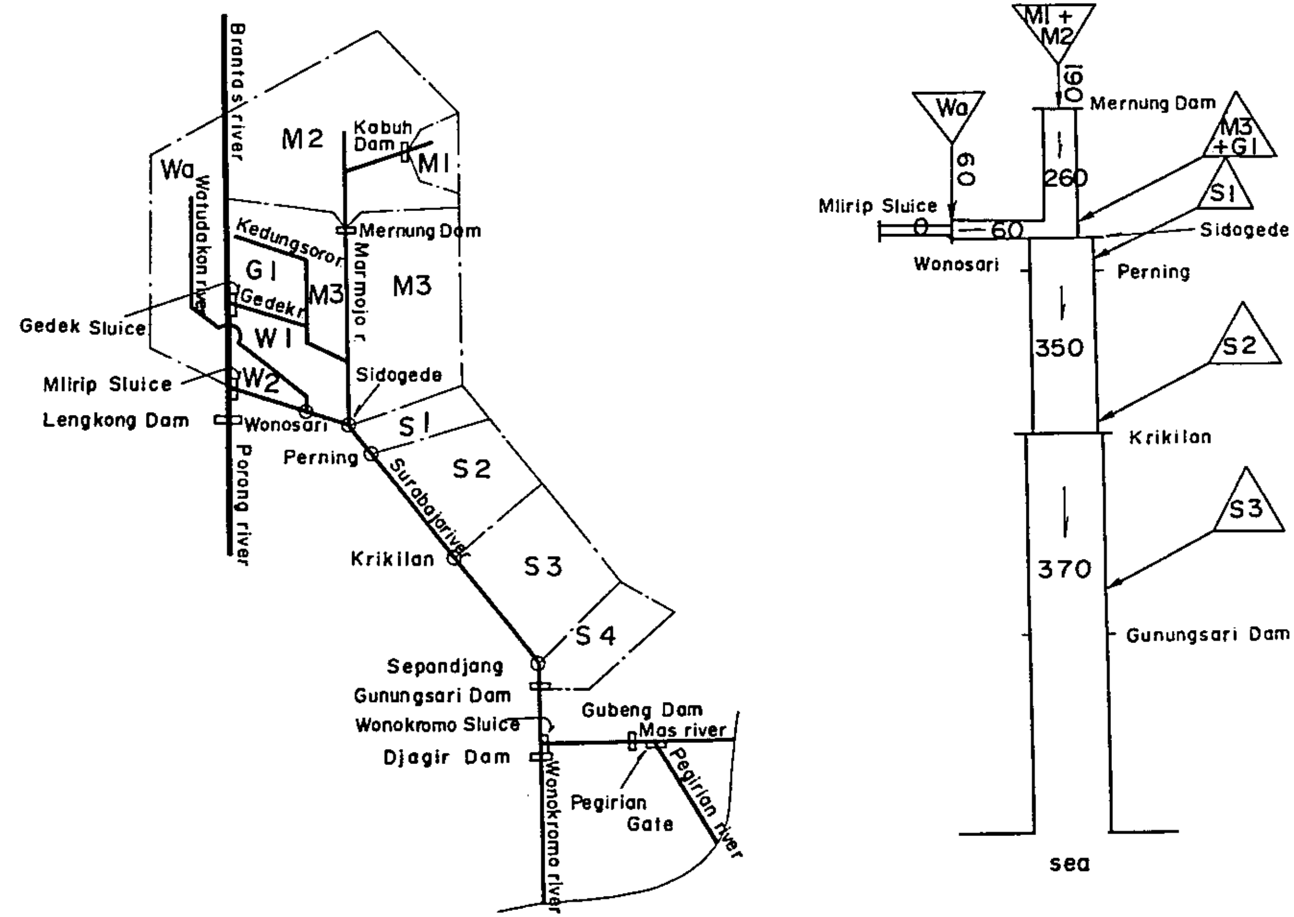


Fig.2 Design Discharge



(2) The Marmojo river.

Although the Marmojo river is a small one, the catchment area of which is only 320 km^2, it suffers from habitual inundations which often extend over 2,000 ha and sometimes continues more than one month. It is obvious, however, that the mere stopping the diversion of the Brantas flood to the Gedeg river and the Surabaja river will reduce the damage caused by inundation or, at least, will decrease the duration of inundation, because not only the flood discharge of the Marmojo river will be decreased, but also the water stages at the confluence of the Marmojo and the Gedeg and that of the Marmojo and the Surabaja will be lowered.

However, the Marmojo river still has its own runoff, though the duration may be much more shortened than the present. Therefore, we have to make a plan for prevention of inundation in accordance with the scale of the new Marmojo and the economic feasibility. The detail of the plan will be explained in Chapter II, Part 2.

(3) The Surabaja/Wonokromo river.

As mentioned later, the present carrying capacity of the Surabaja/Wonokromo river just meets the design discharge almost on the whole length of the river. However, some local improvement works of embankment and revetment are necessary, because some portions of dikes are destroyed and some portions of river banks are so severely scoured that it reaches the body of dike.

Mlirip sluice should be improved by motorizing so that the gates may be operated keeping pace with New Lengkong dam. The body of the sluice may be left as it is, except that only a local portion is improved for the new gate. The lock shall be left for ships as it is, except a local improvement for the new gates.

Gunungsari dam shall be renewed, because it is required to operate the gates as safely and quickly as possible in accordance with new flood waves, since the duration of flood will be shortened than the present one on account of stopage of diversion of the Brantas flood. The lock of this dam shall be abolished, because the transportation by ship is expected to be converted into the transportation by truck at this dam.

Wonokromo sluice may be left as it is, because this sluice will have only to pass more than 10 m^3/s of river water for irrigation, municipal and other use and the lock will not be used any longer.

Djagir dam is still needed together with Wonokromo sluice for the purpose of taking drinking water and possibly prevention of salt intrusion into the ground water. Since this dam will have to be operated at the same pace as Gunungsari dam in order to pass the Surabaja flood, it shall be improved by motorizing the gate operation, while the dam body may be left as it is, because the body is still bearable for further use and has the capacity to meet the design discharge. However, the apron and the adjacent revetments are already worn out. Therefore, considerable repairs are needed for them.

The improvement works mentioned above will be explained later in detail.

(4) The Mas river.

Though the flood water of the Surabaja river is always discharged into the Wonokromo river, the Mas river still has its own catchment area amounting to 13.8 km^2. In this meaning, this is an isolated river which makes one of main drainage canals in the urban area.

Unfortunately, the width of this river has been narrowed by inhabitants, especially by illegal occupants at some reaches. This makes the carrying capacity extremely small principally on the upper reaches. Therefore, this river channel shall be enlarged to meet the design discharge as mentioned later and shall be prepared for the further improvement of smaller drains expected in the following stage.

On the middle reaches of the river, there exists Gubeng dam which has been serving to supply irrigation water to Kalibokor and Djeblokan irrigation system and possibly to prevent the salt intrusion into the ground water. In the present stage, therefore, this dam is needed and shall be left as it is.

On the other hand, the present drainage is in a very difficult status owing to the high stage which is sustained by the dam. Therefore, if it is allowed to lower the water stage by means of lowering or removing the dam, it will bring several important merits, such as (1) to facilitate the natural drainage of urban area which surrounds the Mas river and (2) to allow to lower the water stage upstream from Wonokromo sluice, which will facilitate the drainage not only of the wider area of town, but also of the Kedurus basin which has about 2,870 ha of paddy field.

However, this idea has also several demerits, that is, (1) it may allow salt to intrude into the ground water, which may give trouble to well-water use, (2) it will necessitate compensation works to the intakes for drinking, irrigation and other water, and (3) it may bring some trouble to the people living on both banks of the river. However, if more precise survey will be made for these problems, it is sure that some proper ways for solution will be found.

2. Improvement of Urban Drainage Systems.

(1) Major drainage canals in Surabaja city.

1) Major drainage basins.

The Surabaja city area shall be drained by major thirteen drainage canals as mentioned in Section 3, Charpter I, Part 1. Four of them, Kandangan, Balong, Simo canals, and the Greges river, collecting rain water fallen to the western area of the Mas river, pour into the western coast of the city. The other eight canals, Pegirian, Tambakwedi, Larangan, Kalisari, Kalidami, Keputih, Medokan, and Wonotjolo-Wonoredjo, collecting rain water fallen to the eastern area of the Mas, pour into the eastern coast of the city.

The situation and the drainage area of each canal have already been shown in the above-mentioned Section.

2) Proposed plan for improvement.

i. Mas river.

The idea of improvement of this river has already been mentioned in Article (4) of the previous section and the improvement works in the present project will be mentioned in detail in Chapter IV, Part 2.

This river has three pumping stations, called Darmo, Kupang, and Keputran. The capacities of these pumps will have to be examined carefully at the time when an areal survey of drainage systems is made on the next occasion, and these pumping stations will be improved according to necessity. However, in case Gubeng dam and Djagir dam be allowed to be lowered or removed, these or some of these stations may be abolished because the natural drainage will surely be facilitated. This is also the next problem.

If it is needed to improve such stations, pumps of insufficient capacity or lacking smoothness in operation should be given priority and, at the same time, a consistent plan should be made taking the suction capacity of inflow conduit or pump well, location of equipment and their intervals into consideration. In those pumping stations which have insufficient inflow conduits for suction capacity of pumps, water level of pump well will rapidly be let fall with start of operation, and in those which have insufficient intervals of pumps, air will be inhaled into the pump owing to strong turbulent flow. This often brings about unfavourable results such as shortening life of equipment, etc. In improving pumping stations, therefore, the optimum plan should be chosen from both aspects of construction cost and maintenance cost by a new study of capacity of conduit and type and number of pumps for improvement.

ii. The Greges river and Morokrembangan Boezem.

The Greges river, as mentioned previously, has a catchment area almost as large as the Mas river and is serving for the drainage of almost the whole urban area on the left side of the Mas river.

Although the present catchment area stretches up to Darmo, this upstream basin should be comprized in the catchment area of the Mas river at the time when the drainage of the latter area have been improved. The catchment area at that time will be 13.9 km^2, while the present one is 15.8 km^2. In improving the drainage of the basin, an areal survey will be needed covering its whole area.

Morokrembangan Boezem which lies at the lower end of the Greges river shall be used for the time being, because its regulation effect to the drainage of the Greges basin is remarkably large and wide surface area of the lake itself has another significance from the water pollution control point of view. On the surface of the boezem, the active growth of green plants are observed. This may be due to the enough oxigen disolved in the water which encourages the plants activity, and even though a considerable amount of contaminated water is inflowing to the boezem through the Greges river, the water quality of the lake is being kept relatively clean.

In order to secure the required storage capacity, the deposits at the mouths of the rivers pouring into the boezem shall be dredged. At that time, it may be profitable to reclaim some proper land near the boezem.

The Mitre gates which are installed at the mouth of the boezem have been so devastated that they have to be improved. Even on this occasion, the gate leaves and a part of piers will have only to be improved, because the gate body is expected to be bearable for further use. At the same time, it is important to take into account such arrangement of surroundings as improvement of inspection bridges and lighting around the movable part in order to facilitate operation of gates.

Although the boezem has been decided to be still used in the present project, it should be abolished in the future, because it will probably hinder the development of the city as it is obvious that the sediment and refuses which will continually be poured by the rivers will settle in the boezem year by year and the natural purification capacity of the boezem will be reduced year after year, and eventually it will change into disagreeable facilities. In the future, it may be profitable to reclaim the whole basin and make use of it as an urban land at the time when the utility value of land has been raised.

iii. The Pegirian river.

As already known, requirement for the drainage and sewerage systems is the elimination of the water pollution. The primary reason for the contamination of the river is that more polluted water is now discharged as a result of excessive concentration of population and industries, relatively short length and small flow in comparision with pollution load of effluent have contributed to the low self-purification capacity of the water way, and the sewer system which is indispensable for the treatment of effluent from households and small scale factories are poorly developed.

Although it is obvious that finally the complete modern sewerage system will be provided to the whole city area, it may elapse considerable time until the implementation of the sewerage project. It may, therefore, be necessary to establish the staged improvement programmes to meet the demand. The programmes may be as follows:

(i) First stage programmes.

a. Eliminate the sewage odour in the river by dilution with fresh water. As may be discussed later, the dilution water requirement for the Pegirian river is at present, 0.42 cu m/sec, and it may be pumped up by the Ngemplak pumping station. The pumping capacity of the station is 88 cu m/min or 1.45 cu m/sec which may be able to satisfy the dilution water requirement by the year 1992. After the complete modern sanitary sewerage system has been completed in the future, dilution water for the river may not be necessitated.

b. Remove the deposits on the channel bed and widen the cross-sectional area so that the increased flow capacity may be obtained.

(ii) Second stage programmes.

When the amount of the sewage has increased and the pollutants have been increased, the following second step programmes should be implemented.

a. The sewerage system of Surabaja city should be of the separate system, and the Pegirian river will finally be converted into the storm-water channel shutting off all the sewage flow from the tributary. To intercept the sewage, the intercepter should be laid along the Pegirian river and lead it to the final disposal place where the sewage odour may not affect any nuisances to neibouring inhabitants. By doing so, the prevailing offensive odour in the central part of the city area may be eliminated or at least be decreased.

b. As the sewage volume has further been increased and the urban area has also been expanded, the untreated raw sewage will affect adversely to the disposal place. At this stage, some measures to purify the sewage would have to be considered. Where possible, the oxidation ponds should be constructed for the treatment of the sewage.

iv. Gunungsari hill.

Gunungsari hill is formed of relatively low mountain stretch ranging between 20 to 30 metres high above the mean sea level. It covers an area of nearly 85 sq km in the southwestern part of the city. Having located near high class residential area and convenient place, Gunungsari hill has been developed and used as recreational and residential areas for years. Recently a marked tendency to accelerate the pace of development of the hill has been seen. For example, it has been encroached year by year from the foot by houses by which rich green is being diminished accordingly.

The hill used to be covered by trees and considered to be effective as a water conservation area where considerable part of the rainfáll could have been retained by vegitations and decreasing the runoff to the downhill canal. It is observed, however, that the recent encroachment at the hill side by houses have increased the runoff coefficient in this area. This may be a main reason of the recent frequent floodings at some places along the canal from which considerable amount of water overflows to the neighbouring areas.

Gunungsari canal has been built at the foot of the hill since several decades ago to deliver the irrigation water to farm lands in the western part of the city. Cross sectional area of the canal was, therefore, provided with a capacity of passing the water required for the irrigation purpose only and has not enough capacity to handle the increased amount rainfall water.

In order to improve these deteriorated conditions, attempt have been made by the city authority by widening the cross-sectional area of the canal or constructing bañk at such frequently flooded area as Darmo and Kembang.

What is most urgently required for the canal are: to provide with enough capacity to enable to handle the rainfall runoffs without causing any damage in the tributary, and, if possible, to cut off the excess amount of water at suitable points and divert the water into bypass channels which may discharge the rain water finally to the sea through the shortest routes. Three diversion gates are located at Simowagean, Balongsari, and Kandangan from which the excess water may be diverted during the storm. Taking these facts into consíderation, the canal improvement may include the following:

a. Catchment basin of the canal is to be rearranged. The principle idea is that the runoff from Gunungsari hill area should be collected by the canal and transported to proper points from which the water should be diverted to the Simo and other rivers through which the runoff from the hill will finally be emptied to the sea.

b. Provide the wider section which is to be decided on the basis of the new design criteria. The cross section may be trapezoidal with sub section at the bottom by which the velocity of flow even during the dry seasons may be kept at certain level thus preventing the undesired sludge deposits at the bottom. This sub channel may be used to carry the irrigation water during the dry seasons. If circumstances permit, an isolated irrigation channel or pipe may be constructed along with the canal.

v. Other drainage canals.

The drainage canals of Kandangan, Balong, and Simo not only have their own catchment basins in the downward area from Gunungsari canal, but also, as mentioned above, are to take care of the runoff from Gunungsari hill. All or some of them should be provided with properly-designed diversion facilities at the crossings with Gunungsari canal which, in such a case, will serve as a kind of collecting canal for the runoff from the hill.

Since those drainage canals have many bends especially on their lower reaches, the carrying status should be improved by short-cutting so as to empty the floods as quickly and safely as possible to the sea.

The drainage canals of Tambakwedi, Larangan, Kalisari, Kalidami, Keputih, Medokan, and Wonoredjo originate in the urban area in the right side of the Surabaja and the Mas river, and, collecting the drained water in the farmland as well as in the urban area, pour into the Strait of Madura on the north-eastern coast.

Since the carrying capacities of these canals are all aggravated by settlement of silt and other refuses, they should be rehabilitated by normlizing the water area, and, since the habitual inundation on the landside of the sea dike is encouraged by the devastation of flap gates on the dike, the gates should be rebuilt so as to pass the excess landside water without causing any damages.

It is reported that a large boezem which may collect almost all of the drainage canals is planned. But this idea may not be accepted drainage canals should be poured into the sea as isolated mutually as shown in the previous figure.

In the south-western area of Surabaja city, there exists a drainage canal named Kedurus which is serving for the drainage of the rice field of 2,870 ha. In order to improve the drainage, it is greatly desired to lower the water level just downstream of Gunungsari dam by lowering or removal of Djagir dam and Wonokromo sluice as mentioned in Section 1 of this chapter.

(2) General principles for planning of urban drainage.

1) Basic considerations for planning.

In formulating an overall drainage programme, careful consideration is given not only to physiographic and other physical conditions but to public health problems prevailing at Surabaja. The more important ones are the following:

a. The very large areas involved, 29,200 ha for 1990 and 5,300 ha for 1970,

b. The flatness of the region, which means that the hydraulic gradients of the drainage channels would generally been between 0.0002 and 0.0005,

c. The presence of many swampy areas and shallow depressions, and the small portion of paved or roofed areas; all of which point to low run-off coefficient,

d. The low ground elevation; by which many drains are influenced by the tide and is considered to have given the optimum breeding condition to salt-marsh mosquitoes in some area,

e. The use of drained water for bathing at some places, and

f. The disposal of household refuse to drains.

2) Short-term programme.

To satisfy the reasonable drainage requirements of the area, with a minimum expenditure of money, the programme of improvement must respect the priorities and stay within the guidelines listed below:

a. Clean out and remove all obstructions now cluttering the existing drainage channels such as the Pegirian and the Mas rivers,

b. Enlarge existing drainage channels, particularly the sections which restrict flow,

c. Decrease the roughness of main channels through the cleaning obstacles to smooth flow,

d. Pave as much unlined drains as possible to eliminate the mosquito breeding places,

e. Divert enough quantity of water from the Surabaja river to the Pegirian river in order to prevent the sludge accumulation, odour and mosquito production by giving enough velocity of flow to sweep them away,

f. Plan the improvements to deal with only the most seriously affected areas at first. This would afford time for observation and studies leading to the selection of the most economical solution,

g. Provide, where possible, the retardation basins to reduce the peak runoffs of the drains, and

h. Limit the initial capital requirement.

On the basis of these techno-economic studies, the adopted gravity drains and channels capacity may be limited to extent that is below ultimate condition. The capacity will be increased gradually in accordance with the necessity. The construction schedule should be such as to provide service as rapidly as possible to the greatest possible number of inhabitants, thereby maximizing the benefit and minimizing the per capita cost. At the same time, early alleviation of the most critical health and water pollution problems is an urgent requirement. The construction schedule should take into account these factors and also the need to limit as far as possible the initial capital investment which in any case, will impose a considerable strain on a financial resources available in the drainage planning area. Since it is considered necessary to begin work in some area immediately because of the rapid growth in the area and since the greatest financial difficulties and uncertainties will be at beginning, the first stage programme will be critical. The initial phase of the urban drainage programme may include design and rehabilitation works of the existing systems over a relatively short-term at such a rate that the impact of the cost will not be unreasonable but that it will be possible eventually to keep pace with and catch up on the population growth rate.

3) Outline of the system.

The overall system may consist of main trunk channels, pumping stations and out-falls. Storm water from households and streets or parks will inflow to gutters or to storm water inlets which are to be connected to branch or lateral drains, or directly to trunk channels. This system may be planned to be constructed in several stages up to a few decade later, providing trunk channels as well as the rehabilitation of the existing system according to demand.

District boundaries are decided by watersheds of natural drainage districts, placing the boundary lines at or near the ridges. The layout of the present drainage channels can stay the same for each of thirteen natural drainage systems. Only the size of the channels may change. The tributary areas and trunk channels of the drainage systems may be found in Fig. 3 in Chapter I.

4) Quantity of storm water.

In computation of storm water quantities, the rational method may be used. The computed quantity of storm water is a function of the area to be drained, the rainfall intensity and the runoff coefficient.

Experience has shown the rational method to yield satisfactory results if properly applied, and has been widely used in many cities. Quantity of storm water may be presented in the equation is;

$$Q = \frac{1}{360} CIA$$

where Q is a peak flow, in cu m per second, C is a runoff coefficient, I is an average rainfall intensity, mm per hr, and A is a tributary area, in ha.

5) Rainfall frequency for design.

The average frequency of rainfall occurrence to be used for storm sewer design may be decided as 3, 5 or 10 year depending upon the importance of the area to be protected. The protection afforded by the sewer system should be consistent with the amount of damage prevented. Although the study of the feasibility of the system in deciding intensity most economical rainfall for design of the system was not conducted, judgment supported by records performance in other similar city is the basis of selecting the design frequency. The following range of frequencies may be reasonably accepted:

i) For storm sewers in relatively low population density area such as residential, building and official and force area, 3 to 5 year frequency.

ii) For high-value districts such as commercial and shopping areas, 10 to 20 more, depending on economic justification.

6) Rainfall in Surabaja city.

Character of rainfalls of the Tropics is different from those in the Temperate zones. The Tropical rainfalls are usually caused by the so-called "black clouds" which develop from towering mass of clouds and releave rain water at relatively strong intensity in a short duration covering area of nearly five to six kilometres in diameter. The clouds usually follow the direction of wind.

Observation of specific characteristics of the rainfalls in Java island was recently conducted by Tadashi Tanimoto, Colombo Plan Expert who submitted to the Institute of Hydraulic Engineering, Ministry of Public works and Power, the "Rivised and Enlarged Edition of the Hourly Rainfall Analysis in Java". According to his observation, the most rainfalls were concentrated in the afternoon especially between 15:00 and 20:00, having strong rainfall intensity at the beginning with a relatively short duration, and the rainfalls in Surabaja area have a character of mountain distribution, that is to have much rainfalls in the evening but less in the daytime.

He also reviewed the hourly rainfall accumulation curve in Java proposed by Dr. Boerema, and concluded that the proposed curve may give a little higher rainfall intensity as compared with the result of his studies. In the report, however, no such data indicating the intensities in short duration which are required in formulating rainfall-intensity-duration equation are given.

Necessary rainfall data for the drainage system in Surabaja then have been collected and analized for each rainfall between 1962 and 1972 by means of Logarithmic Normal Distribution Method, and the rainfall-intensity-duration equations for 5, 10, 20, 25, 50, 100 and 200 year return periods are computed as follows:

5 yr return period; $I = \frac{6,680}{t + 41}$

10 " " $I = \frac{8,800}{t + 50}$

20 " " $I = \frac{11,000}{t + 57}$

25 " " $I = \frac{11,700}{t + 59}$

50 yr return period; $I = \frac{13,800}{t + 63}$

100 " " $I = \frac{15,700}{t + 66}$

200 " " $I = \frac{17,600}{t + 68}$

It should be noted, however, that the accuracy and reliability of the result depend upon the number of the data to be treated. Observed number of only 10 might not be sufficiently enough to support very accurate result. Therefore, result of these examples may be subject to review and modification when more data are available in the future.

7) Time of concentration.

An estimate of the time of concentration to the point under consideration is made so that the average rainfall rate may be determined. For urban storm sewers, the time of concentration consists of inlet time plus time of flow in the sewer from the most remote inlet to the point under consideration. Time of flow in the sewer or channel is estimated closely from the hydraulic properties of the conduit. Inlet time is the overland flow for runoff to reach established surface drainage channel such as street gutters and ditches and travels through them to the point of inlet. It will vary with surface slope, nature of surface cover and length of path of surface flow, as well as with variables influenced by antecedent rainfall intensity and duration such as infiltration capacity and depression storage.

An equation that represent the inlet time for urban sewer was first proposed by Horton and was later improved and formulated by W.S. Kerby in the form:

$$t = \frac{2}{3} \times 3.28 \times l \times \left(\frac{n}{\sqrt{s}} \right)^{0.467}$$

where t: inlet time, in minute,
l: distance from the most remote point to the point of inlet,
s: mean surface slope, and
n: coefficient roughness similar to the runoff coefficient, as given in Table 1.

Let s be 3 per mil and n be 0.1 in our case, the inlet time may be;

t = 8.3 say 8 minutes

Table 1 Values of the n in Kerby's Equation

Character of surface	Coefficient roughness
Smooth pavement	0.02
Bare, packed soil, free of stone	0.10
Poor grass cover	0.20
Moderately rough bare surface	0.20
Average grass cover	0.40
Forest (deciduous tree)	0.60
Dense grass cover	0.80
Forest (deciduous tree, with deep dead leaves)	0.80
Forest (needle-leaved tree)	0.80

8) Runoff coefficient.

Runoff coefficient, C is the variable of the rational method least susceptible to precise determination. Its use in the formula implies a fixed ratio for given drainage area, whereas, in reality, the coefficient accounts for abstractions or losses between rainfall and runoff which may vary for a given drainage area as influenced by differing climatological and seasonal conditions.

In practice, use of overall coefficients found by experience to produce acceptable results. The range of the runoff coefficients, classified with respect to the general character of tributary area, given in Table 2, may be reasonably adopted to the design of the drainage system in Surabaja.

Table 2 Runoff Coefficients for Different Zones in Surabaja City Area.

Zoned areas	Runoff coefficients
Business area	
Shopping area	0.8
High density area	0.8
Harbour area	0.6
Industrial area	0.5
Building & office areas	0.5
Middle density area	0.5
Low dènsity area	0.4
Military area	0.3
Recreational area	0.2
Plantation area	0.1
Forest & hunting area	0.1
Unimproved area	0.2
Free land	0.2

The city planning of Surabaja has been accomplished by the Team Master Plan Surabaja, in which the city area in 1990 is expected to expand as wide as 29,178 ha, while the city area as of 1970 is 5,275.7 ha. The city areas are classified into 15 different zones as given in Table 3.

Table 3 Planned Zonings of Surabaja City Area in 1970 and 1990.

Zonings	1970	1990
Industrial area	211.3 (ha)	2,968.25 (ha)
Building and office area	298.3	849.25
High density "	999	3,176.75
Middle density "	40.6	1,873.5
Low density "	505.6	3,320
Shopping "	74.8	571.6
Harbour "	-	-
Military "	-	2,109
Recreational "	64.9	819
Plantation "	-	1,328
Forest for hunting	-	2,650
Canal and river	213.9	-
Unimproved area	-	3,833.75
Free land	2,233	5,678.9
Other (air port, grave)	634.3	3,833.75
Total area	5,275.7 ha	29,178 ha

From the basic design runoff coefficients given in Table 2 and the ratio of each zoning to the city area given in Table 3, we may obtain averaged design runoff coefficients for both in 1970 and 1990 conditions for the whole urban area.

Table 4 Averaged Runoff Coefficients for the Entire City Area in 1990.

Zoning	Per cent of zone	Runoff coeff.	Averaged Coeff.
Industrial area	10.2	0.5	0.051
Building and office	2.9	0.5	0.015
High density	10.9	0.8	0.087
Middle density	6.4	0.5	0.032
Low density	11.4	0.4	0.046
Shopping	2.0	0.8	0.016
Harbour	-	-	-
Military	7.2	0.3	0.022
Recreational	2.8	0.2	0.006
Plantation	4.6	0.1	0.005
Forest for hunting	9.1	0.1	0.009
Canal and river	-	-	-
Unimproved	13.1	0.2	0.026
Free land	19.4	0.2	0.039
Total	100.0%		0.354 (0.400)

Table 5 Averaged Runoff Coefficient of the City Area as of 1970

Zoning	Per cent of zone	Runoff coeff.	Averaged Coeff.
Industrial area	4.2	0.5	0.021
Building and office	5.9	0.5	0.030
High density	19.7	0.8	0.158
Middle density	0.8	0.5	0.004
Low density	10.0	0.4	0.040
Shopping	1.5	0.8	0.012
Harbour	-	-	-
Military	-	-	-
Recreational	1.3	0.2	0.003
Plantation	-	-	-
Forest for hunting	-	-	-
Canal and river	-	-	-
Unimproved	12.5	0.2	0.025
Free land	44.1	0.2	0.088
Total	100.0 %		0.381 (0.400)

9) Design of channels.

In the drainage system, trapezium cross section or alternate designs for precast and cast-in-place sections are to be used for large scale sewers and rectangular section may be used for medium sizes. Circular section may be used only for small sizes because of its high construction cost and difficulty in obtaining wide variety of its sizes.

Channels in the most part of the city areas may have to be lined by either brick or concrete block, preferably by brick or stone from the economical viewpoint. Another advantage of using brick masonry is its corrosion resistance and is to be recognized for drainage systems in which considerable amount of sewage may be poured for the time being.

Basic design data for the drainage channels may be as follows:

i) Minimum velocity of flow in the channel of not less than 60 cm/sec.

ii) Maximum velocity of flow of not more than 250 cm/sec.

iii) Increasing velocity of flow gradually from the upstream to down stream.

iv) Minimum earth covering of pipes of not less than 90 cm/sec. (where not possible to lay pipe deep suitable protection measure such as concrete covering may be provided)

v) Computation of flows in channels by the Manning's equation, with roughness coefficient of 0.015 for concrete pipe and 0.025 for open channel lined.

10) Design of pumping stations.

Location, arrangement, type of equipment and structure, and external appearance are all basic considerations in the design of pumping station. Stations will be located in areas where water may be impounded without creating an undue amount of flood damage if the inflow exceed the station capacity.

Each pumping station will be provided with necessary equipment such as bar screen, pump well, pumps and accessories etc. These equipment may be so arranged as to give space for the future expansion of the station capacity when needed.

Station capacity must be adequate to meet the maximum rate of flow which may be decided in the sewer design. Because the tributary areas are not fully developed, initial station capacity will be such extent as to meet the requirements for a reasonable time.

3. Improvement of Sewerage Systems.

(1) General considerations.

The protection of water resources against pollution is basic to the development of sound national economy. For both public health and conservation purposes, it is essential that pollution be controlled. As the population increases, it has become recognized that indivisual devices such as privies, septic tanks and such practices as the disposal of night-soil by burial contribute significantly to the pollution of soil and water.

In many densely inhabited and industrialized areas in the city, there is a rapid deterioration of the water quality and is becoming less agreeable every year. There is a very persistent public demand for pollution control in the city area. It is quite clear that to bring such control into effect requires a completely modern sewerage system.

The drainage and sewerage construction and operation programme is a major undertaking requiring great effort and considerable expenditure. However, the benefits are also great. Although there have been no such data available to clarify an interrelation between the morbidity and the expansion of the drainage and sewerage systems, it is evident that if the systems were provided and well managed, many advantages will be gained. Since most of the benefits although real, are unquantifiable, it is not possible at this stage to compute a benefit-cost ratio, and this problem may be studied in the next stage of the project.

Benefits of the system will include:

a. the convenience and prestige value of a basic modern urban infrastructure component which is lacking at present,

b. elimination of the present strong sewage odours emanating from drains and channels and accumulations of sludge and scum therein, deriving from cesspool effluents or lavatories beside the drains, and direct discharge of sullage water,

c. reduction of the mosquito breeding places by eliminating the impounding reservoirs or swamps, and by lowering the salt density of water in tidal rivers,

d. an expected improvement in health, with consequent reduced discomfort and distress, by reduction of gastro-enteritic disease, which is considered at present to shorten the life of residents of the city,

e. elimination of the odours and other nuisances due to the exposure of excreta on the surface of the drains at present having vault latrines and elimination of inconvenience of vault latrines,

f. control and eventual substantial reduction of the present very serious river and estuary water pollution, which now causes extreme visual and odour nuisances in certain areas under low-flow conditions,

g. for economic benefits expected is a reduction in the cost of treatment of the part of Malaria and gastro-enteritic diseases.

(2) Choice of sewer system.

The decision as to whether combined or separate sewers to be constructed for the project should first be accomplished. For Surabaja city, separate sewerage system will reasonably be adopted because advantages may overweigh disadvantages by the following reasons:

a. As the first step, construction and/or rehabilitation of the drainage channels and pumping stations may have to begin in important areas of the city and then gradually extend out into the surrounding areas. Construction of the sewerage system is expected to be further delayed until funds become available. It is therefore, economical to plan the separate system so as to avoid undesired excessive amount of preinvestment by constructing the combined system in which the enlarged sewer capacities are to be shared for the future quantities of the sanitary sewage.

b. Since it is not feasible to construct treatment facilities adequate to care for entire flow of combined system, it is necessary to separate or by-pass a major part of peak flows, and for that outfall facilities are usually designed with capacities only slightly in excess of the dry weather flow from the combined sewers. Hence when it rains, overflows of mixed and storm water at each discharge point with heavy bacterial comtamination of receiving water. In addition, Surabaja has high rainfall precipitations with strong intensity in relatively short duration concentrating mostly in the afternoon between 3:00 and 7:00 p.m., during which time peak sewage flow is likely to occur simultaneously, and if the combined system were provided the contamination of the receiving water ways may further be intensified.

c. Human waste water contains nitrogen and phosophate which may affect the eutrophication of receiving water and will cause the damage to agricultural plants,

d. Because of the flatness of ground surface of the tributary areas, several booster pumping stations may be required to lift the sewage, regardless combined or separate. Under the condition, the separate. Under the condition, the separate system may relieve the stations from lifting of unnecessary amount of increased water and save the expenditure for both construction and operation.

e. The combined system may require replacement of certain portion of existing open drainage channels by closed conduits or pipes, by which inhabitants near the water ways would probably have to renounce their traditional and important customs of taking baths and washing in the water, while the separate system could give better quality of water to the water ways leaving the most existing water ways in shape of open channel as they are at present.

f. Since the combined sewers are normally required to be deeper than storm-water sewers, excavation difficulties such as quick sand, muck etc., encountered in the construction of the large combined sewer may be more costly and make separate systems more desirable.

g. Surabaja has net works of natural drainage system and there are quite a few portions of the drain which are considered to become usable at low expenses if some rehabilitation works are carried out. Shortage of the construction fund is expected to continue for the time being and the rehabilitation works could warrant the efficient usage of the limited fund. These works of course, should accord with design criteria for overall plan so that these facilities will function later as a part of the entire system.

(3) Tributary area of system.

The sewerage planning area may include whole city area expected to expand by the year of 1990, the design year of the system. The area may further be divided into several sewerage districts. The district boundaries are determined by watersheds of natural drainage districts or by such structures as railways, canals and high ways etc., which may interfere with runoffs or sewer construction to across them.

(4) Population estimate.

Total population of Surabaja city area in 1970 was 1,518,352 and is expected to increase as high as 3.75 million by 1990, as studied in Part 4. In accordance with the city master planning, the whole city area is divided into thirteen zonings each having different population density ranging from less than 100 to as high as 1,400 persons per hectare. Design population for each district may be calculated by multiplying the population density by the area for each zone comprised in the district.

(5) Domestic sewage.

Daily average, daily maximum and peak per capita flow of sanitary sewage are taken from the water supply rates, neglecting the losses due to leakage and sprinkling to lawn. Therefore, the quantities of domestic sewage to be used in the designs of the sewerage system are:

Daily average per capita flow;	200 litre
Daily maximum per capita flow;	250 "
Peak per capita flow;	400 "

(6) Industrial wastes.

As already discussed in another chapter, water demand for industrial purposes in 1992 is predicted at 50 cu m per ha of industrial zone daily. Therefore, the total industrial wastes in the district may be computed by multiplying this unit by the area of industrial zones comprised in the district.

(7) Infiltrations.

Infiltrations such as leakage of water from the surrounding ground into the sewers and from other openings are undesirable. However, because of poorly made joints of sewers or broken manholes, reasonable rate of infiltration is to be considered in design.

Units used in expressing infiltration rate include, cu m per day per hectare of sewered area, per unit length of pipe, per cm diameter per km of pipe, or some fixed percentage to the total discharge of sewers. Each expression has a characteristic itself and could be selected depending on the case. However, at this stage there is no such reliable data available to calculate the exact amount of infiltration and perhaps the last one may be used in the design of sewers. As this figure, between 10 and 20 per cent is most widely used and for this city 20 per cent of the daily average sewage flow may be reasonably accepted.

(8) Design flows.

Design flows will be decided by adding domestic sewage, industrial wastes and unavoidable infiltration. The total sewage discharge from the system in 1992, as discussed Part 4, may be as follows:

Daily average sewage flow; 1,132,000 cu m/day

(9) Design of sewer capacity.

A factor of safety is usually provided in the design of sanitary sewers such that small sewers up to 30 to 40 cm in diameter will not flow more than one-half full at the peak design flow. Larger sewers may be designed to flow from more than half-full at the peak design flow up to about seven-tenths of the diameter for sewers 80 cm and larger in size.

Another factor to decide depth of flow is local climate. Having been high in temperature around year, temperature of the sewage in conduits will probably be raised accordingly. Under the condition decomposition of organic materials in the sewage liable to take place in the sewers and sulfide build up will be caused. Sulfide build up is a main cause of erosion of sewers and may harm sewer crew, and some clearance should be provided above water surface so as to ventilate the undesired gasses in the sewers.

(10) Selection of sewer materials.

In the sewer construction, circular cross section may be used for all sewer sizes from 250 mm to as large as 2,000 mm in diameter, because of its excellent hydraulic properties, and the construction cost of precast concrete pipe is usually considered less than that of rectangular cross section sewers cast-in-place.

Sewers are most commonly made of concrete. Other materials such as asbestos cement pipe and brick masonry conduits are also being widely used for sewer construction without trouble. As far as they have ample strength against stresses and corrosion resistance against the sewage gasses, the materials for the sewer can be selected from either the brick masonry, concrete pipe or any other materials.

(11) Design data for sewers.

Principle of sewer design are:

a. Minimum velocity of flow of not less than 60 cm/sec and maximum velocity of not more than 250 cm/sec.

b. velocity of flow in the sewer increasing gradually from upstream to down stream,

c. minimum earth covering of sewer of not less than 100 cm (where not possible, suitable protective measures should be provided).

d. roughness coefficient of 0.013 for centrifuged concrete pipe and 0.015 for other concrete conduits, and

e. maximum depth of pipe laying of less than 7 m.

(12) Manholes.

Manholes are to be provided at junctions of sewers, at changes in alignment of grade except in curved sewers, and at places where the future connection is expected. Maximum spacings of manholes are determined depending upon the sizes of sewers connected to manholes.

(13) House connections.

House connection pipes may be provided between households and public sewers through which sewage is discharged from each house. The pipes may be 150 mm of more in diameter. House inlet or inspection chamber may also be provided where necessary.

(14) Sewage treatment facilities.

Sewerage system concentrates potential nuisances and dangers at the terminus of collecting system and the load imposed upon the transporting water must be unloaded prior to its disposal into receiving water. Although the complete sewage treatment may be needed at the critical time of the year, the plant facilities and devices and its treating capability will be increased gradually in accordance with the increase of the sewage amount.

Sewage treatment may be able to design only after the degree of receiving water quality is decided. At present, time is not mature to discuss in such details as on what process is to be selected or how many number of the plant will be necessary in the system. Before proceeding further into details of treatment designs, an engineering study of the possible means of water pollution control and a social and economic analysis of the most desirable means are required and only after these are established such works may be able to start. In this report, therefore, efforts are given in finding low-cost treatment process which will meet the local requirement from both economical and technical view points.

In the separate sewerage system, both construction and maintenance costs of ordinary sewage treatment facilities may exceed that of sewers, and the need for low-cost treatment devices, whose operational requirements are within the capabilities of the rapidly growing urban communities and industrial complexes, is therefore increasingly apparent.

As a means of low-cost waste water treatment facilities, in many countries where enough sunlight and high temperature is expected, stabilization ponds have been widely used to treat sewage discharged from communities and successively operated until now. The stabilization ponds, also called sewage lagoons or oxidation ponds, are recognized to have two advantages for the cities in which financial problems are prevailing; i) their construction requires principally land and labour, and ii) they are generally cheaper to install than conventional treatment plants. The cost of constructing adequate treatment plants often discourage their installation in cities with limited budgets. However, treatment is possible by means of stabilization ponds which are usually less expensive than more conventional method. Stabilization ponds have undergone sufficient study and development to be classified as one of the major type of waste water treatment system.

An observation at Morokrembangan Boezem clarifies that so far as the pollutants loading are kept at a suitable balance with bacteria, the receiving body of water has capability of self-purification and the water will not be contaminated. In other word, part of the lake is acting as a sort of stabilization ponds thus preventing contamination of the effluent of the lake. Despite the inflow of considerable amount of contaminated water through rivers, the water quality of the lake has been kept relatively good so far. This may be because natural purification is put to good use in shallow ponds. Bacterial decomposition of organic wastes matters releases carbon deoxide to the water, and growth of algae develops under the favorable climatic conditions. Beside utilizing carbon dioxide, this microvegetation consumes ammonia, phosphorus, and other plant-growth substances that are presented in the waste water. These phenomena are clearly observed in the water of the lake.

However, disadvantage is also accompanied with the stabilization ponds, i.e., requirement of wide land and odour problems during operation. If we assume that allowable BOD loading on the ponds surface as 50 kg/ha/day and daily per capita BOD loading as 30 g/day, approximately 1,600 persons may be able to be served by one hectare of pond surface area. By this design criteria, for example, nearly 3,000 ha or about 10 per cent of the whole city area will be required if we depend all the sewage treatment by the stabilization ponds at the year of 1990. It is obvious that land acquisition of such a wide area is not possible, and therefore, utilization of stabilization ponds will be limited only in the districts in which enough space is obtainable without any trouble derived.

By the reasons discussed above, utilization of the stabilization ponds for treating sewage should be considered only as a transitional step to a completely modern treatment plant until the time when quantity and contamination of sewage have increased and fund for construction of treatment plants has become available. Careful considerations are to be paid on justification in selecting the system from the economical and pollution prevention standpoints, and staged programmes are to be prepared in order to reach the optimum solution.

4. Dilution of River Water

(1) Odour elimination.

As discussed in a previous section, rivers in the city area, especially the Pegirian river, have been suffering from heavy water contamination which is emanating offensive odours to wide areas in central part of the city. Although this condition will not be able to improve unless the modern sewerage system is constructed, some suitable temporary measures should be taken without delay.

Among the effective measures to reduce the septic odours of water are; i) dilution of waste water by fresh water, ii) aeration of the waste water, and iii) oxidization of the water by chlorine. Of which, the dilution and flushing may be most economically and reliably applied to the river system in Surabaja city if enough amount of dilution water is available.

The dilution of waste water usually requires relatively large amount of fresh water. In some cases as much as twenty to fourty times to the waste water were necessary to obtain the satisfactory result. Even if this volume of the dilution water is available, other difficulties such as necessity of wide land for enlarged sections and increased construction cost may be accompanied with. It is, therefore, necessary to conduct an experiment to find out the minimum dilution ratio of the fresh water to the waste water, to weaken the intensive odours to bearable level.

In experiment, samples are collected from the Pegirian river and are mixed in plastic cylinders with fresh water which is taken from the Surabaja river at Djagir dam. Four samples are prepared at different dilution ratios such as 1 to 2, 1 to 3, 1 to 5 and 1 to 10, and after being swirled strongly for a few seconds, selected five persons are given samples and smelled them without knowing dilution ratio of samples. The samples are given to the persons in the increasing order from the thinner one, and this test had been continued until any of persons sences the odour. The result appears to indicate that the minimum dilutions ratio is one waste water to five or more fresh water.

(2) Self-cleansing velocity.

Another advantage by flushing of river water is to increase the oxygen disolved by the increased surface area and aeration capacity of water to encourage aerobic microorganisms activities in water, thus preventing growth of scavanging organisms. At the same time, increased velocity of flow in the water may give enough self-cleansing velocity to the water courses to sweep away or prevent accumulation of sludge deposits which is usually the main course to create an anaerobic conditions.

The self-cleansing velocity in terms of removal of sewage solids is a matter of experience and judgment. As a conservative rule of thumb the sanitary engineering profession is rather well agreed that a minimum value of 45 cm per second is a self-cleansing velocity where the channel will normally be flushed out occasionally by increased water.

(3) The Pegirian river.

The Pegirian river, starting from a junction to the Mas river near Ngemplak bridge and flows to the north passing through the heart of the city area finally emptying its water to the sea, has been the main cause of unsanitary conditions in the part of the city. The nuisances and hazards by the river is mainly due to odours that have been produced by anaerobic decomposition of accumulated sludge onto the river bed. The odour of fresh domestic waste water may be described as musty, which is not especially offensive, however, when the water is old and begins to putrify, the odour becomes very objectionable. This condition creates undesirable esthetic and public health problems.

It is obvious that the most effective means to prevent such public nuisances is to shut off all the waste water from rivers. It is expected, however, that the drainage and sewerage constructions will be delayed to the latter stage of the programme, and until that time some temporary relief measures must be considered.

The effective measure to reduce septicity of water is, as mentioned in the previous section, a dilution of waste water by fresh water which is probably most economically and reliably applied to the rivers in this city when enough quantity of flushing water is available.

On the basis of the above considerations, a set of experiment was conducted to determine the least acceptable dilution ratio to diminish the odours. The result indicates that the minimum dilution ratio of the Pegirian river water to the fresh river water is around one to five or more, and the minimum fresh river water required for dilution and flushing the Pegirian at present may be 14,800 cu m daily or 0.17 cu m per second as are discussed in Part 4.

5. Improvement of Sea Dike and Sluices.

Only a part of about 3.6 km of the sea dike which has a total length of about 17 km is exposed directly to the sea. Of this length of 3.6 km, two portions which total to about 1.16 km are being attacked severely by sea waves and, notwithstanding, their crest elevations are lower as much as about 0.5 m compared with the other part. These two portions are so devastated that the dike body has often been eroded by overtopping waves, therefore they should be improved for securing the safety of protected land.

On the other hand, since almost all of the flap gates on the dike cannot move due to corrosion by the sea, they should be renewed by modernized ones in order not only to prevent the intrusion of sea water, but also to facilitate the drainage of landside excess water.

6. Improvement of Irrigation Systems.

(1) General.

As explained in Section 1, Chapter I, Part 1, the production of staple crops in East Java Province which comprizes the project area has been being increased.

Although population of East Java Province was 26,991,782 in 1970 and annual increase was 660,995, food supply to the population seems to have been sufficient in view of the increase of production of crops.

Food supply per capita in 1970 was estimated at 150.21 kg of carbohydrate food in rice equivallent. It can be said that this figure is sufficient because the national target in 1970 was 149.50 kg.

This sufficient food supply was supported by efforts of Bimas and Inmas program, which respectively covers the acreage of 367,066 ha and 150,000 ha out of 1,192,531 ha of total planted area in East Java Province.

Total paddy production of the Province amount to 4,387,911 ton of dry stalked paddy of which the share of Bimas and Inmas was about 19.5 percent. It means that increment over target yield was 962,674 ton of dry stalked paddy.

Owing to the increase of rice production, 250,000 ton of export of maize was achieved in 1970.

Under the situation that the agriculture of East Java Province is going well as above, it is considered that the irrigation area of Surabaja city plays an important role in the agricultural production both in the past and in the future.

At present, in about 95 % of the irrigation area, paddy is planted in the rainy season, but in dry season only 25 % of the area is irrigated and, in another 25 %, paddy is planted without any water supply guaranteed by the Government. Second crop such as maize, peanuts, pepper and green vegetables are produced on the land of about 1 % to the whole irrigation area throughout a year.

With the development of economy of Surabaja city, migration and concentration of population to the city area will be made continually in future. Then it is natural that diversification of agriculture will be promoted and employment opportunity of farmers in the city organization will be increased.

As for farmer's income, other portion of income than agriculture will become large, then aspect of agriculture of the Surabaja city area will change into an urban agricultural pattern. Crops which will be consumed in the city such as vegetables, fruits and other crops, and flowering grasses and ornamental plants will be willingly cultivated by farmers as cash crops which will promise good returns. In the surrounding zone of the city area cattle raising might be developed.

Thus, existing irrigation area in Surabaja city will be changed step by step into urban area for residence, factory and office building, from high elevation zone toward low elevation zone and from near to the city center toward far. Consequently, the center of agriculture will be obliged to move from high elevation area toward low elevation area where terminal irrigation and drainage system are in poor condition.

Such natural tendency will bring some problems on the irrigation canal system and its command area. Conversion of land use is now going on from near to the intake sites of each irrigation system. Some amount of irrigation water for such developed land may be converted to municipal water supplies. In the downstream, however, farming area is still remained, main canal system can not be eliminated and necessary amount of irrigation water should be transported to terminal farming plot.

Sewage, refuse and deposit from such developed land along the upper portion of main canal will so easily pollute the irrigation water that it will become unsuitable for irrigation year after year. To prevent this, some countermeasure should be taken into consideration of city development.

Tendency of decrease of acreage of the irrigation area has been studied carefully as described in chapter II, Part 3. According to the study, it is estimated that the irrigation area of Surabaja city after 20 years will be between 5,700 and 6,400 hectares.

Therefore, rehabilitation works of irrigation systems are still important and should be carried out earnestly to retain or increase agricultural production of the project area.

Especially water distribution among beneficiaries of water use will become more severe in accordance with development of the city. So that repair, rearrangement or establishment of water measurement divices and protection works against water losses may be required in future. Concerning these matters, the following measures should be taken into consideration:

a. Rehabilitation or improvement of intake structure for each irrigation block (including water measurement divices),

b. Rehabilitation of carrying capacity of main, secondary and tertiary irrigation canals,

c. Rearrangement of checks and turnouts in each irrigation block in accordance with conversion of land use,

d. Land rearrangement of farming field especially in the surrounding area of city,

e. Improvement of farm-drainage system in connection with city drainage.

(2) Gunungsari canal.

As described in the present status of irrigation facilities, the

Gunungsari canal is located at the foot of Gunungsari hill in totalling 21 km long approximately and irrigates 1,293 ha of paddy field. All drained water from the hillside is pouring into the canal and discharged to three main drainage canals through canal spillways constructed at crossing points of them.

From the points of effective supply of irrigation water and timely drainage of heavy rainfall, the function of the canal should be divided in accordance with irrigation and drainage.

So it is desirable to examine the following idea by further investigation and study.

a. Between intake point and Simo drainage canal, irrigation canal will be made of pipe line or box culvert and its route will be made as the same as existing right bank center. And existing canal bed and left bank will be enlarged so as to meet the drainage of runoff from hillside.

b. All drainage canals on the hillside shall not be joined with irrigation canal but have a structure of two-level crossing for smooth drainage of the area.

(3) Gubeng irrigation system.

Kalibokor irrigation block and Djeblokan irrigation block belong to the Gubeng irrigation system. The Gubeng dam was constructed about 70 years ago so that the dam might contribute to regulate water surface and divert necessary amount of water to the service area.

Irrigation canals, however, have been quite deteriorated and sediment has been accumulated on the canal bed. Since the main canals of the irrigation block pass through the city center, they are extremely polluted by the waste from the city area. Conversion of land use from irrigation area to city area is remarkable in the two irrigation blocks (especially in higher elevation land).

From the point of desirable urban drainage, Gubeng dam might be abolished. But irrigation area is still remained in the lower elevation land, therefore irrigation water supply can not be stopped. One of the countermeasures against the above, the following idea of new-canal construction may be proposed for the future study;

a. The Gubeng irrigation system will be abolished and incorporated with the Gunungsari irrigation system with a new right-bank main canal.

b. Kebonagung intake will be used for the future intake of new right-bank main canal, and Surabaja-river water shall be supplied to the Gubeng irrigation area.

c. New right-bank-irrigation main canal will be located along Kebonagung canal, Pandjangdjiwo canal and via Pandjangdjiwo syphon, Wonoredjo syphon, Wonokromo syphon, Medokan syphon, Klampis diversion, Manjar Sabrangan diversion, Saluran syphon, Kalikepiting diversion, Kali Djudan diversion, Djudan syphon, Rangkah syphon, Bulak diversion and ending at Pogot diversion. The total length will be about 17 km and the command area will be 3,925 ha out of 4,488 ha which is the total of existing Djeblokan, Kalibokor and Kebonagung irrigation area. The difference of the two areas will be converted from farming field to urban area.

d. According to a very rough calculation, the loss of head between intake and terminal point is 4.5 m, then the optimum water level in front of Kebonagung intake will be 5.00 m (S.H.V.P.). There will be no difficulty in retaining the water surface at Kebonagung intake site judging from actual control of water surface at Gunungsari dam.

e. The canal will be able to contribute to living and industrial water supply as well as irrigation in future.

f. Total construction cost is very roughly estimated at 1,200 million Rupiah equivalent to 737 US$/ha (1 US$ = 415 Rp). So annual cost will be US$ 29, 34, 40, 47 per ha for interest of 3%, 4%, 5%, 6%, respectively. If annual benefit of agriculture sector can be expected to be more than 50 US$/ha as in case of the Brantas Delta Irrigation Rehabilitation Project, the project will have an economic soundness. -When related benefits such as living and industrial water supplies will be added to the above, the project will be more profitable.

For reference, rough route map of new right-bank main canal and profile are shown in Fig. 3 and 4.

Fig. 3 Route Map of New Right Bank Main Irrigation Canal

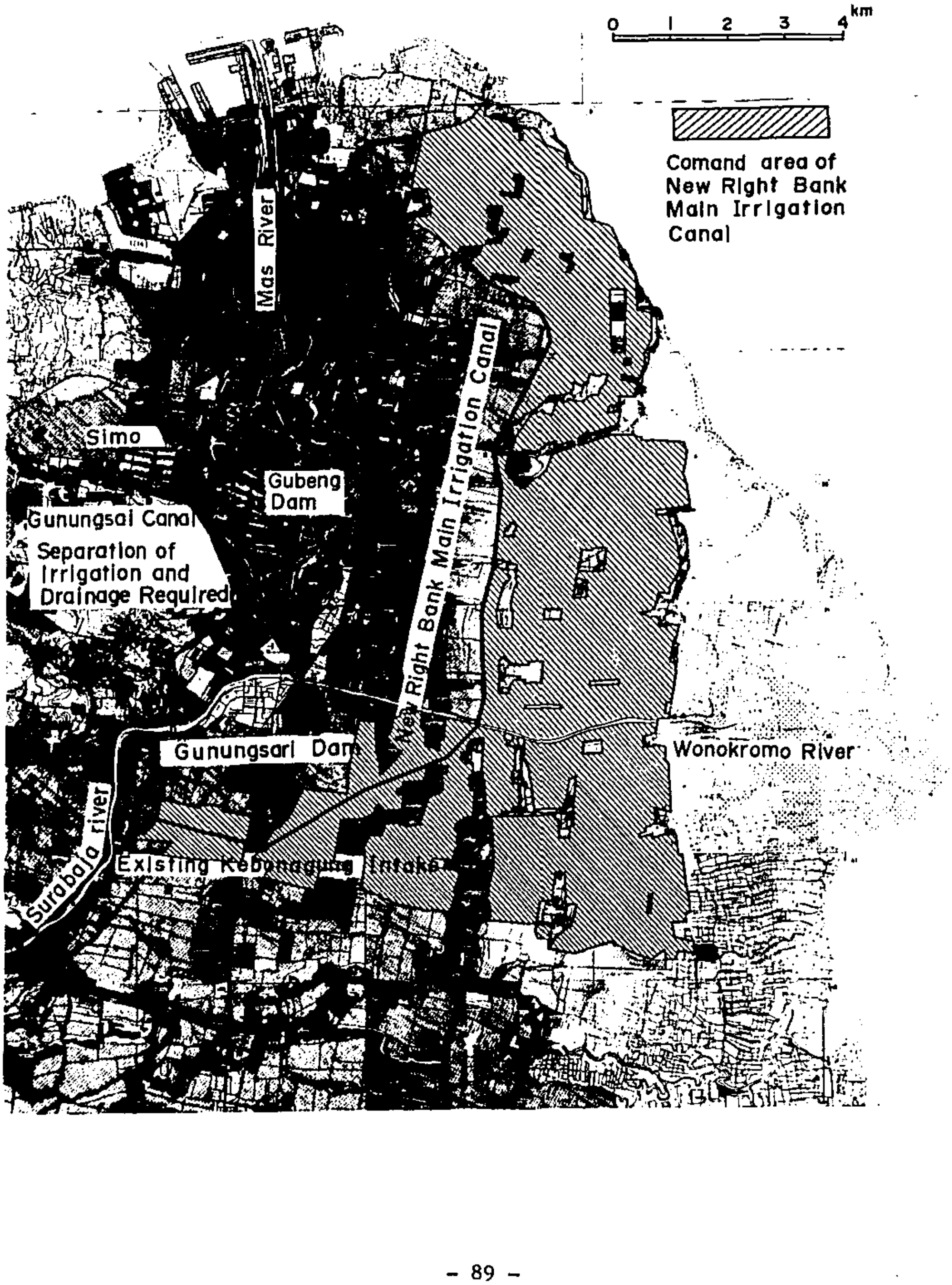
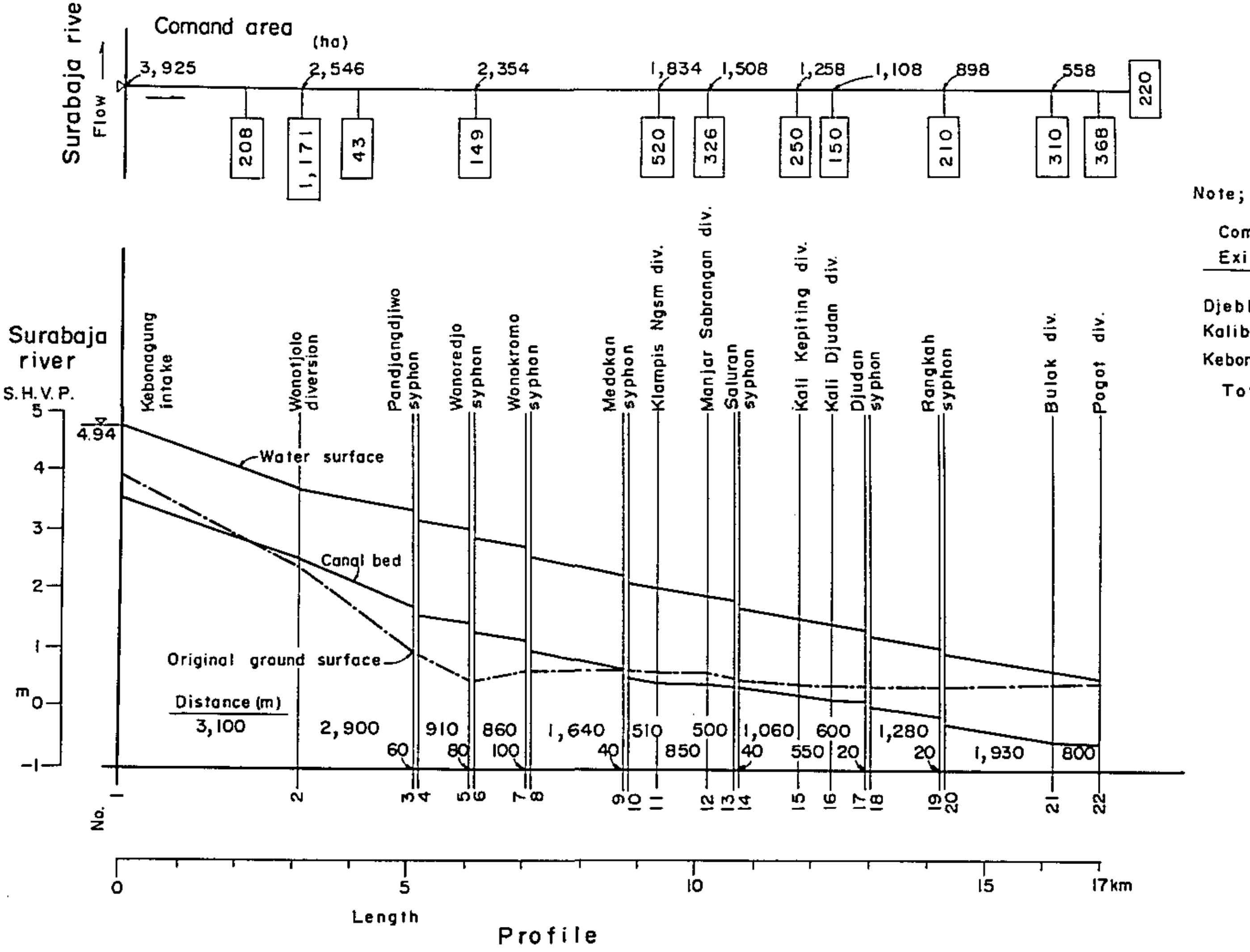


Fig. 4 Diagram of Irrigation Blocks of New Right Bank Main Canal



Note;

Comand area of Existing systems	ha
Djeblokan	1,808
Kalibokor	1,109
Kebonagung	1,571
Total	4,488

PART 2

THE SURABAJA RIVER IMPROVEMENT PROJECT

CHAPTER I

SCOPE OF THE PROJECT

At the time when the Study Team was returning to Japan after the study in Indonesia, they submitted to the Director General of Water Resources Development an interim conclusion on the scope of work for the study. This conclusion which was agreed by the Director General is as follows.

a. It was agreed that the study of improvement of the Marmojo river shall be included in the scope of work of the study. It was also agreed that the team will cooperate with the Brantas River Survey Team in studying the flood discharge distribution of the Brantas river to the Surabaja river, within the frame work of "hinterland" which is described in the Terms of Reference.

b. The Surabaja river and Wonokromo canal including important facilities such as dams and sluices shall be included in the scope of work of the study.

c. The Mas river should be dealt with at the following stage of drainage and sewerage systems of Surabaja city, because the flood water of the Surabaja river should not be poured into the Mas river through Wonokromo sluice and the Mas river should be taken into consideration as a main drainage canal within the drainage and sewerage systems of the city.

d. The inundations in some parts of the urban area could not be diminished unless the secondary and tertiary channels of drainage were rehabilitated or improved, however the pumping stations were strengthened in their capacity. There is an essential unbalance between them. So the rehabilitation or improvement of the pumping stations should be inclusively dealt with at the following stage of study of drainage and sewerage systems of Surabaja city.

e. The carrying capacity of Gunungsari canal is less than 3.5 m^3/s. so that it is essentially difficult that the canal accepts the flood runoff from Gunungsari hill. The rehabilitation or improvement of Gunungsari canal should be inclusively dealt with at the following stage of study of drainage and sewerage systems and irrigation rehabilitation, because the disposal of the flood runoff from the hill should be separated from the irrigation system and, therefore, this problem can not be solved independently of an integrated study on the urban drainage and sewerage systems and the irrigation systems.

f. Concerning the rehabilitation or improvement of the existing irrigation system, the team showed an idea that the intakes of Djeblokan canal and Kali Bokor might be moved to the upstream of Gunungsari dam when Gubeng dam was desired to be removed in order to lower the water level upstream of the dam for the purpose of improvement of drainage of the city and improvement of water quality for irrigation. Furthermore, the problems of main drainage canals in farm land should be studied at the following stage of study on irrigation systems and drainage and sewerage systems of the urban area, because the main drainage canals in farm land cannot be dealt with independently of the drainage and sewerage systems of the urban area since those canals also serve at present as drainage of the urban area and, further, the urban area is so energetically expanding to the farm land year by year that the farm land is expected by the Team Master Plan of Surabaja city to vanish by the year 1990.

g. The team suggested the benefits of lowering the normal water level upstream of Djagir dam and that this idea is desired to be studied at the following drainage and sewerage systems and irrigation systems.

h. The sea-dike was originally built in order to protect the city from the menace of intrusion of the sea, while the present condition of sea-dike is critical owing to frequent overtopping of the sea and devastation of dike body itself. In view of such critical status of the existing sea-dike which also contains extremely devastated flap gates, the rehabilitation of sea-dike shall be included in the scope of work of the study.

i. Concerning the rehabilitation problem of Morokrembangan Boezem, this problem, at least rehabilitation of Mitre gates at the mouth of the boezem shall be included in the scope of work of the study from the view-point of the menace of intrusion of the sea.

j. Study of water demand shall be included in the scope of work of the study.

In deference to the above conclusions and on the basis of the Minutes shown in Appendix D, the following improvement works were adopted in this report as the scope of the feasibility study for the present project.

a. The Marmojo river improvement work.

b. The Surabaja/Wonokromo river improvement work.

c. The Mas river improvement work.

d. Morokrembangan Boezem improvement work.

e. Sea dike improvement work.

The study on water requirements for municipal, industrial, irrigation, and river dilution use will be mentioned later in Part 3.

CHAPTER II
THE MARMOJO RIVER

1. Principles of Improvement.

The principles of improvement on the Marmojo river are as follows:

(1) Carrying capacity of river channel.

The carrying capacity of the existing river channel is as shown in Fig. 1, and the cross-sectional area of river is small over the entire length of river. It is so in particular for the lower part of the confluence of the Gedeg river and causes the flood water level to rise menacing the levees near the confluence with any danger.

(2) The Marmojo river upstream of the confluence of the Gedeg river.

As shown in the Part dealing with Analysis and Study, when the diversion of flood from the Brantas river through the Gedeg sluice is stopped, the inundation of the Marmojo river at the upper part of the confluence of the Gedeg river which so far continued over a long period of more than ten days will become only of several hours duration, with the amount of damage due to it reduced significantly. However, any river improvement work to prevent this inundation of short duration will require a huge sum of cost and not be economically feasible as explained in the aforesaid Part. Therefore, any work will not be adopted for this area, which will be left as it is.

(3) Excavation of river channel at the lower part of the confluence of the Gedeg river.

As explained in the Part dealing with Analysis and Study, excavation of river channel will prevent the levees from overtopping or breaking by flood and, at the same time, have a favourable effect to diminish any inundation at the upper section of the confluence. Also it is economically feasible. The excavation of river channel from P.0 to P.70 and the mebankment work from P.0 to P.15 will be adopted, with the levee provided with a freeboard of 0.8 m.

(4) Design high-water discharge and level.

The water level of the Marmojo river at the design high-water discharge described in Section 1, Chapter II, Part 1, is as shown in Fig. 2, and that at the confluence of the Gedeg river water level will be lowered giving a favourable effect to reduce the inundation depth as well.

(5) Revetment and other works.

Revetment works will be executed at several places where the main levee is in a dangerous condition due to the destruction of river bank in the section between P.0 and P.70. In addition, some repair works will be taken into consideration as it seems that such temporary mending as the protection of foundation is needed for the sluices or bridges in the same section.

Fig. I Carrying Capacity of the Existing Marmojo River

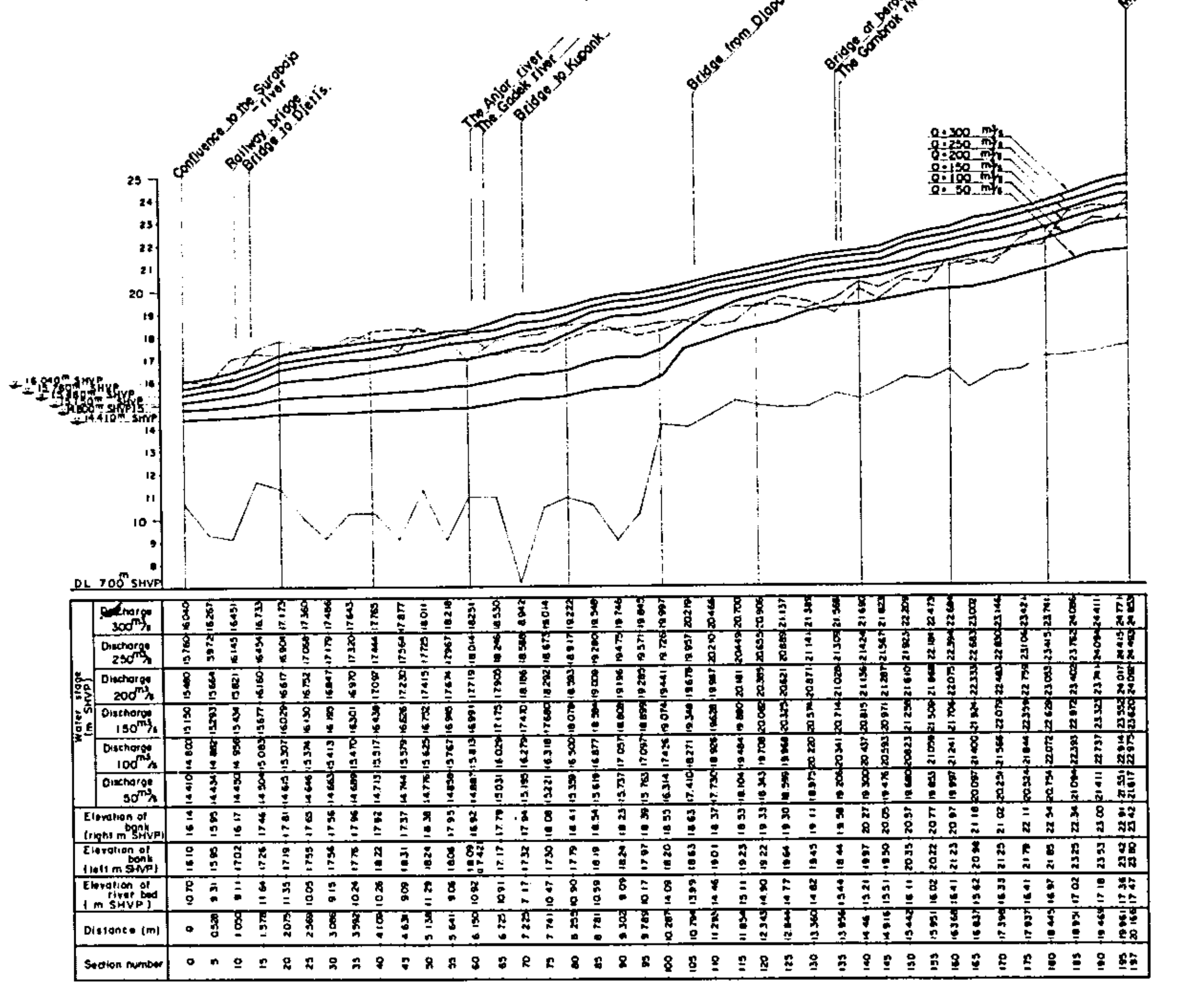
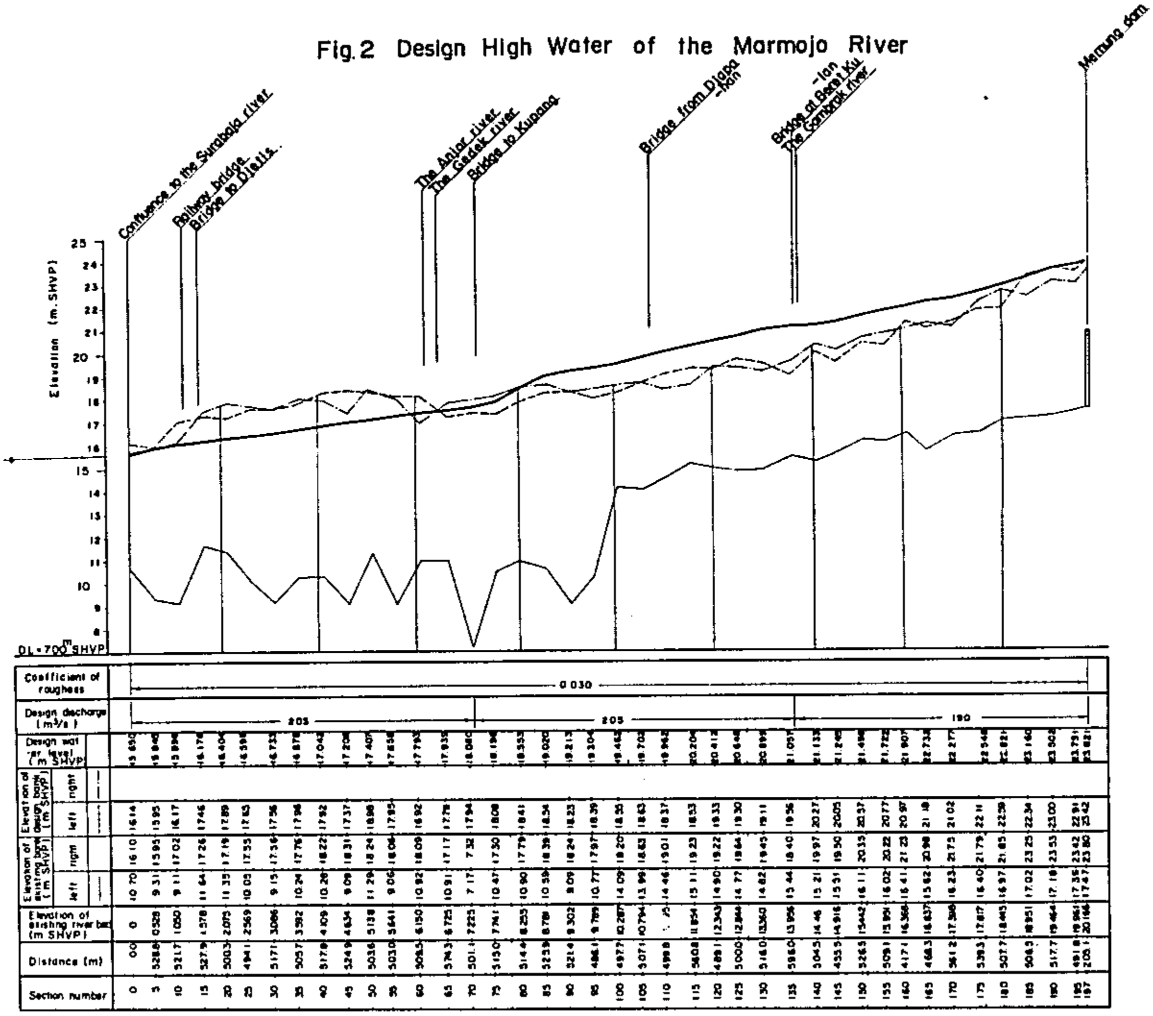


Fig.2 Design High Water of the Marmojo River



2. Construction Works.

(1) Quantity of works.

a.	Qt. of excavated spoil	320,000 m^3
b.	Qt. of earth for banking	15,000 m^3
c.	Length of revetment	1,000 m
d.	Area of land required	152,500 m^2
e.	Appurtenant work	

(2) Execution.

The construction period is 2 years and the works will be carried out with the excavators including 1 set of Hoe and 2 sets of dragline, each of 64 m^3/hr in capacity. If the operation is made on the basis of 6 hrs. per day, 23 days per month and 10 months per yr, the quantity of excavation per year at the operation rate of 70% is

$$E = 64 \times 0.7 \times 6 \times 23 \times 10 \times 3 = 185,472 \ m^3/yr > 160,000 \ m^3/yr$$

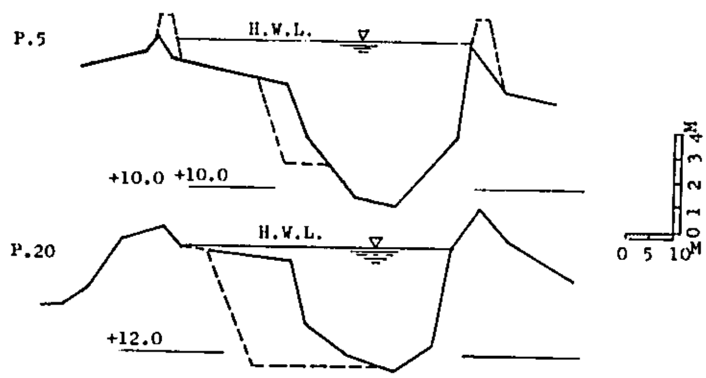
A half of the whole quantity of spoil shall be transported by man power and the other half shall be transported by ten dump trucks, each having a loading capacity of 6 tons. If the trucks make 12 return trips per day carrying earth of 3.7 m^3 per trip over the distance of 3 km, the annual quantity of transportation at the operation rate of 80% is

$$T = 3.7 \times 0.8 \times 12 \times 23 \times 10 \times 10 = 81,696 \ m^3/yr > 80,000 \ m^3/yr$$

Thus, using these equipments the works can be completed within about 2 years. The size of land for dumping the spoil has been calculated on the assumption that the total quantity of earth from which the banking material has been deducted is to be filled up in 2 m high. The works of revetment and others will be performed by man power using only the small equipments of general types, without taking into consideration the use of any particular machine.

(3) Typical cross-section

The typical cross-sections for the project are as follows:



CHAPTER III

THE SURABAJA-WONOKROMO RIVER

1. Principles of Improvement.

The principles of improvement of the Surabaja-Wonokromo river are as follows:

(1) Carrying capacity of the river channel.

The carrying capacity of the existing river channel is shown in Fig. 1 and the water level of the river channel at the design flood discharge described in Section 1, Chapter II, Part 1, is shown in Fig. 2. According to these figures, it is judged that the carrying capacity of the river channel is sufficient over the entire length of river.

(2) Embankment and revetment.

Some parts of levee where their heights are insufficient will be strengthened by an additional banking. The length of such strengthening embankment is 10,000 m in total. Also, revetment will be executed on the parts of levee where the levee body is in danger of breaking due to a severe local scour of the river bank. The length of such revetment work is 1,000 m in total.

(3) Improvement works of Mlirip sluice.

The main structure of the sluice will be left as it is, the improvement will be made on the stop logs of entrance replacing them with steel gates to be operated by a power hoist, and also on the mitre gates of lock replacing them with those that can be power operated.

(4) Improvement works of Gunungsari dam.

A dam will be constructed newly at a site directly downstream of the existing dam. This new Gunungsari dam is to be equipped with steel gates which are power-operated, but not with a lock. Also the works of successive revetment will be executed for 290 m along the stream on the lower side of new dam.

The water stage on the upper side of the new Gunungsari dam will be maintained at SHVP + 5.0m so that intaking of irrigation water will be secured in the future.

(5) Improvement works of Djagir dam.

The main structure of the dam will be left as it is and the improvement will be made on the skin plates of gate with the hand hoist replaced by a power hoist. The apron and about 260 m portion of the successive revetment which have deteriorated will be repaired too.

(6) Wonokromo sluice.

As the improvement of this sluice must be dealt with together with that of Gubeng dam on the Mas river according to the results of study of ground water now under way, it should be considered inclusively in planning the urban drainage system at the second stage of the project.

2. Construction Works.

(1) *Quantity of works.*

a. Length of banking 10,000 m

b. Length of revetment 1,290 m

c. Improvement works of Mlirip sluice

Preparatory and temporary works, improvement works of gate grooves, building of hoist tower, installation of gate and hoist, and miscellaneous materials.

d. Improvement works of Gunungsari dam

Preparatory and temporary works, foundation and cutoff of water, works on main structure, concrete revetment, bed protection, inspection bridge, earthwork on new banks, installation of gates, hoists, and miscellaneous materials.

e. Improvement works of Djagir dam

Preparatory and temporary works, concrete apron protection, cross-shapted concrete block and riprap works, revetment, renewal of gate skin plates and installation of hoists, and miscellaneous materials.

(2) Execution.

The construction period is 2 years respectively for works on banking and revetment, 1 year respectively for those on the Mlirip sluice and Djagir dam and 3 years for work on Gunungsari dam. One hoe and one dragline is used for banking, of which material will be obtained from the excavation of high-water channel nearby. The works on Mlirip sluice and Gunungsari and Djagir dams are as shown in Fig. 3-7, and these works will be executed using properly various equipments described later in the table of particulars for equipment.

Fig. I Carrying Capacity of the Existing Surabaja / Wonokromo River

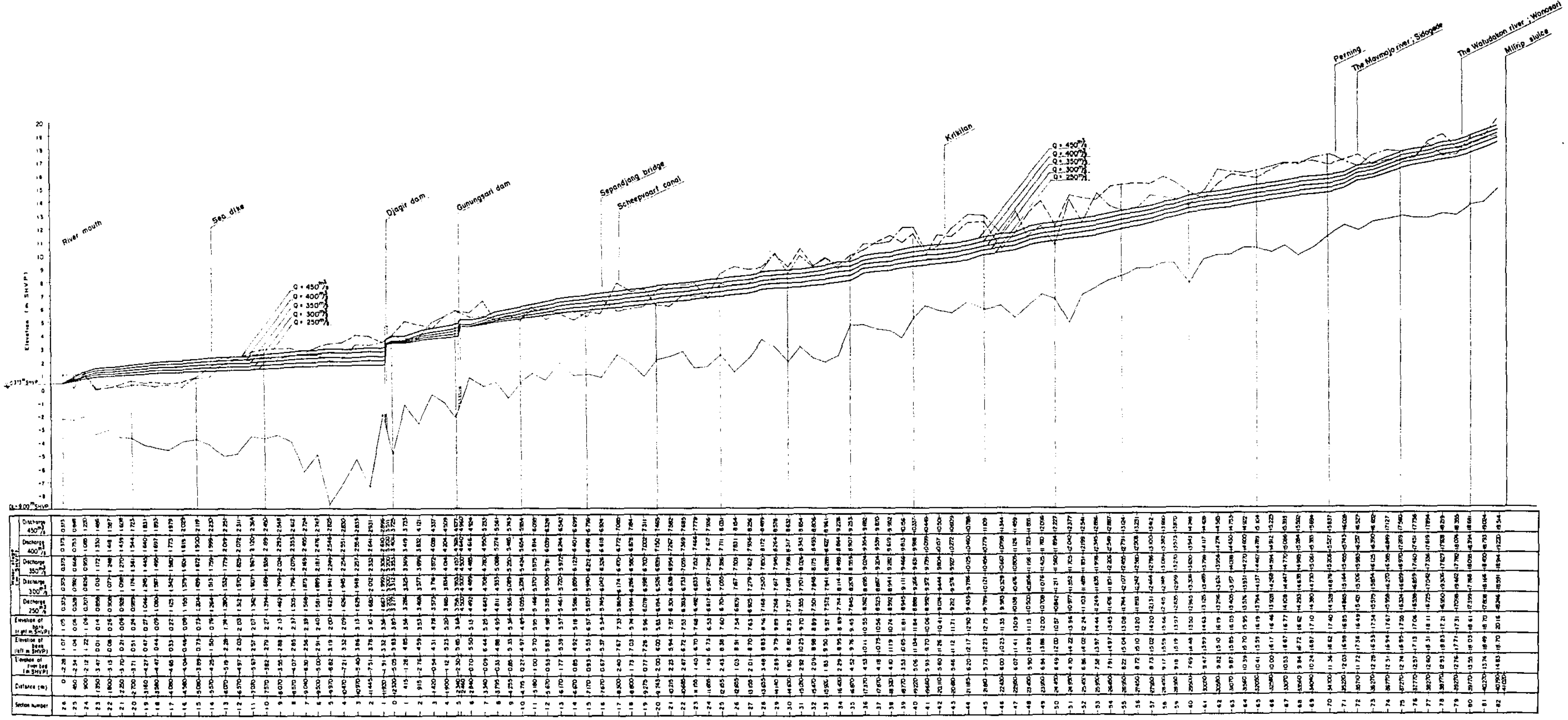


Fig.2 Design High Water of the Surabaja / Wonkromo River

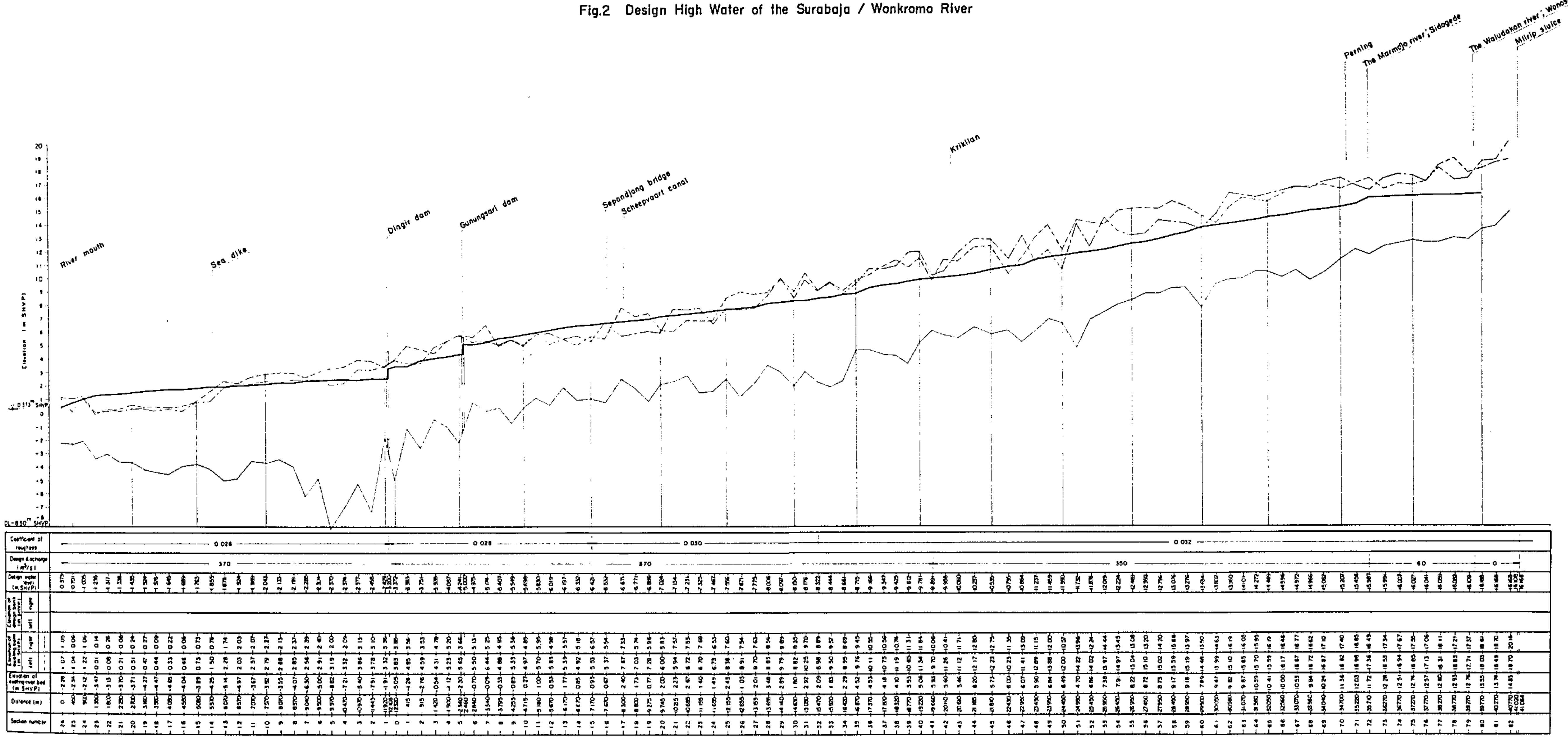


Fig 3 & 4 IMPROVED MLIRIP SLUICE

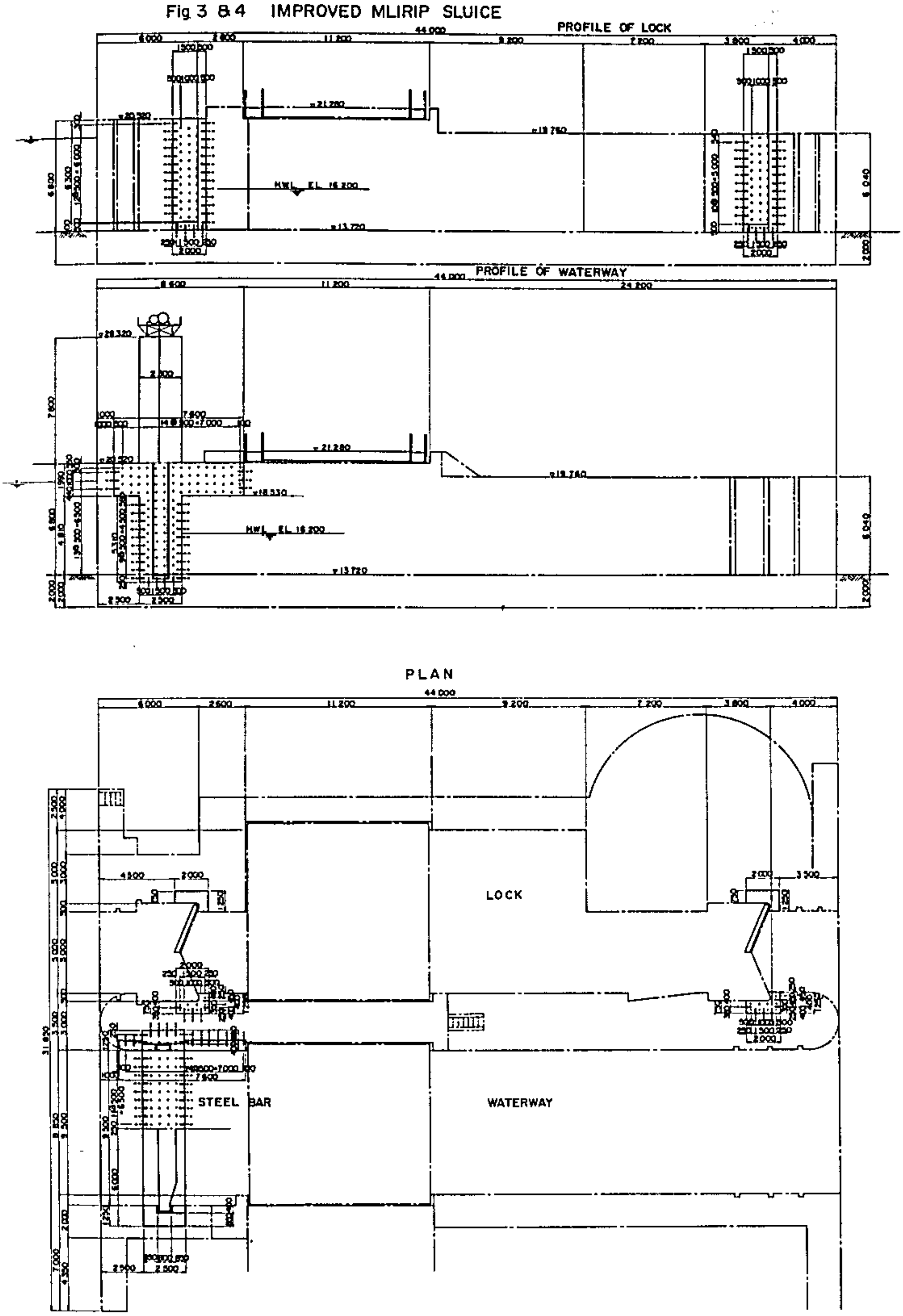


Fig. 5 Plan of New Gunugsari Dam

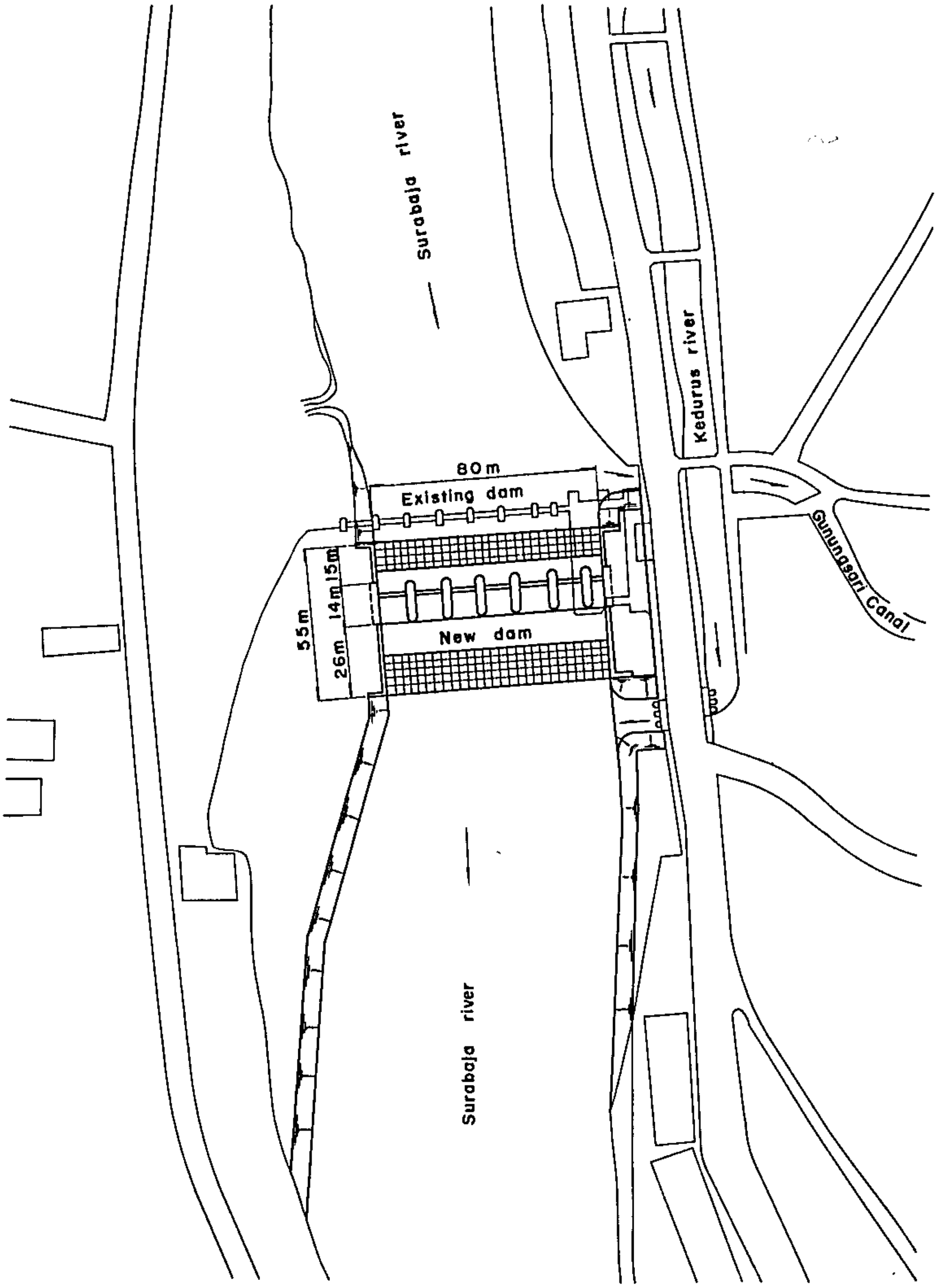
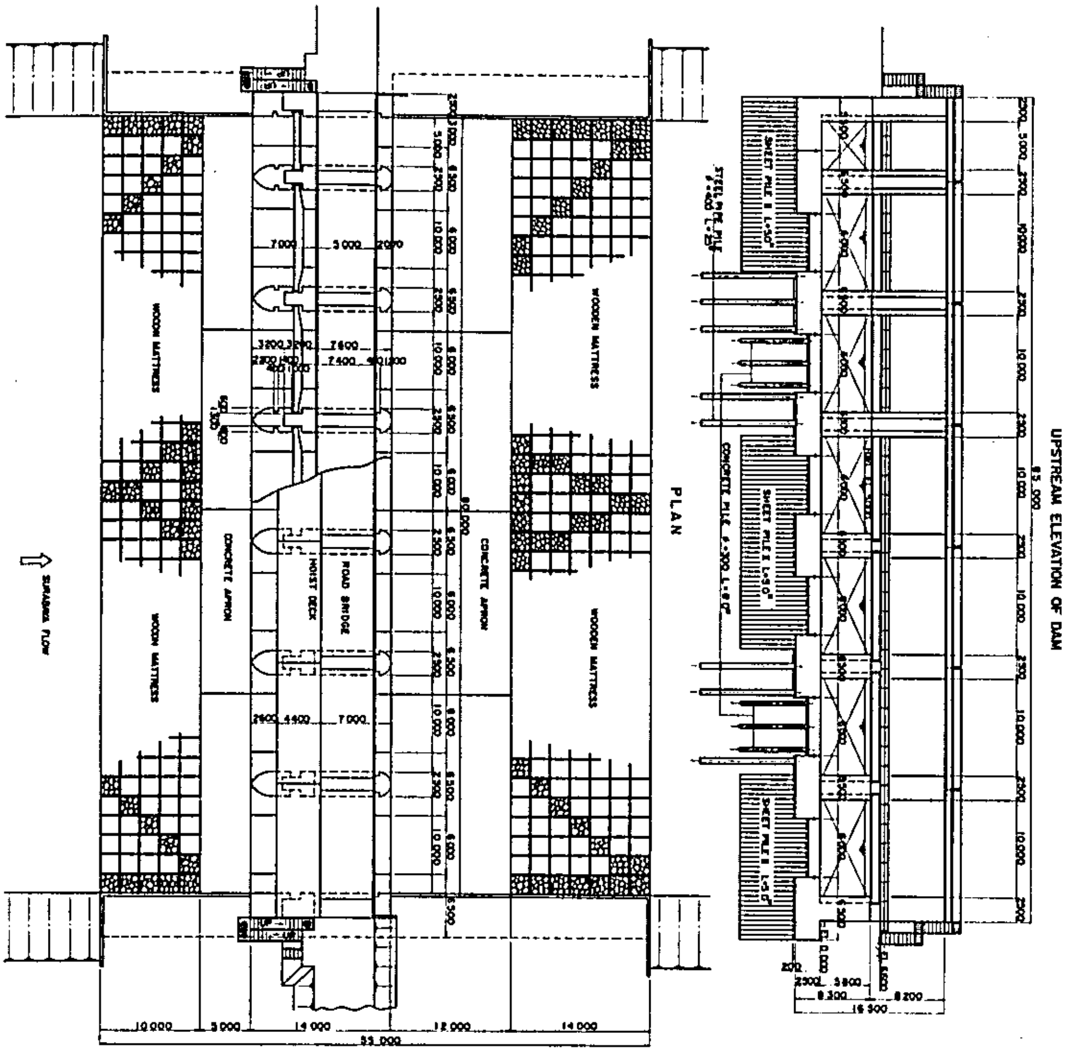


Fig.6 NEW GUNUNGSARI DAM



PROFILE

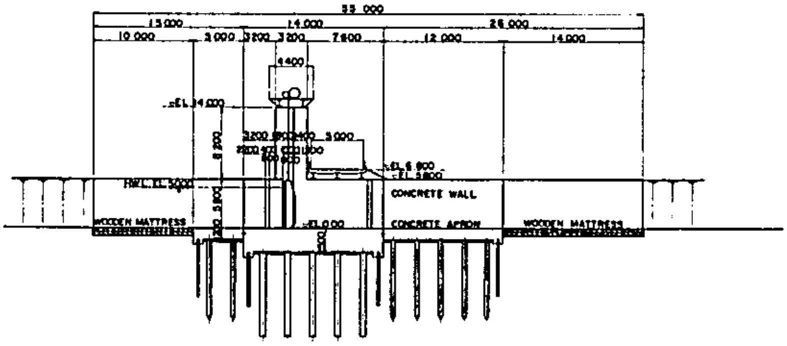
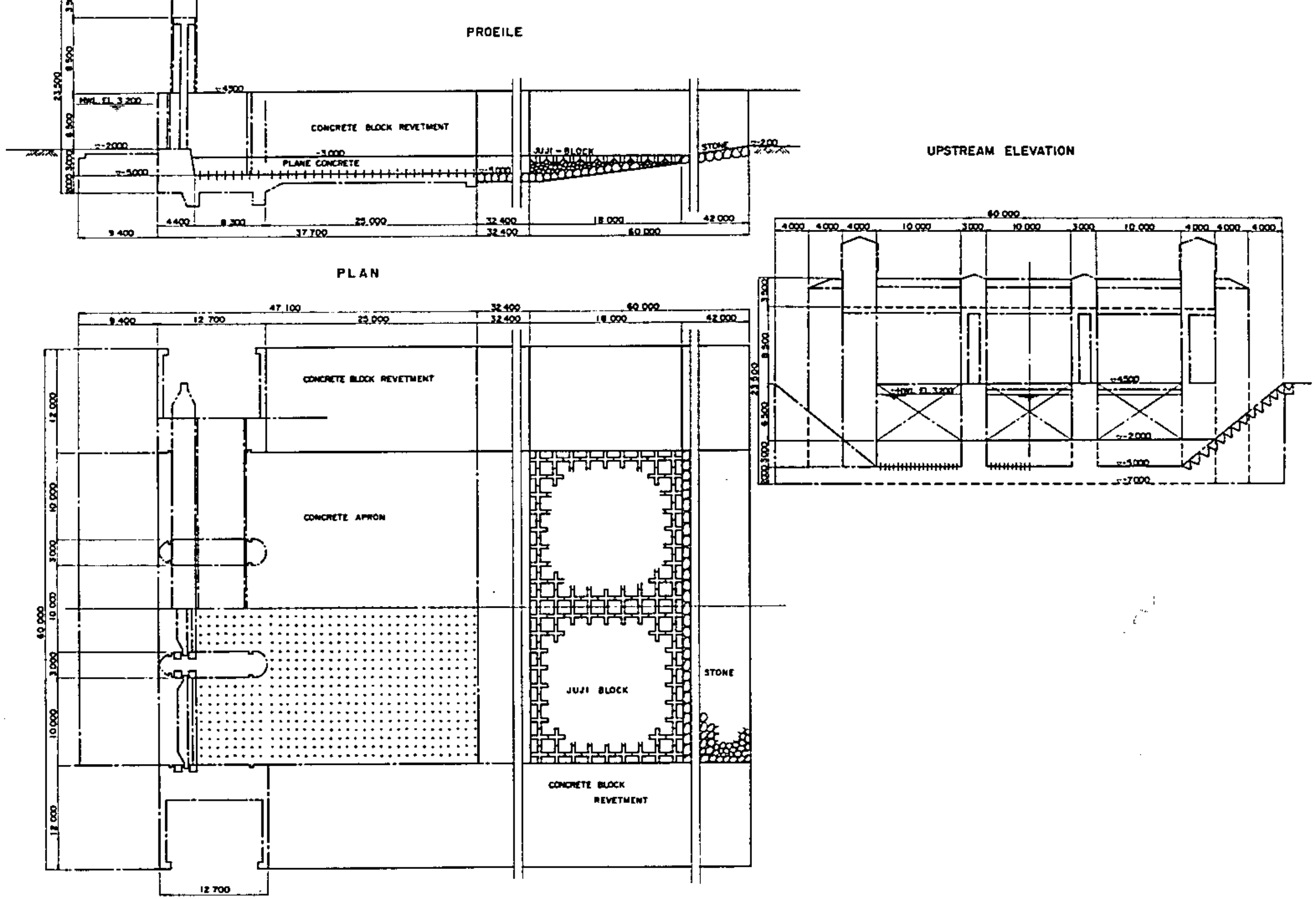


Fig.7 IMPROVED DJAGIR DAM



CHAPTER IV

THE MAS RIVER

1. Principles of Improvement.

The principles of improvement on the Mas river are as follows:

(1) Carrying capacity of river channel.

The carrying capacity of the existing river channel as shown in Fig. 1 is small over the whole length of the river and it is extremely small in the section between Gubeng dam and Wonokromo sluice.

(2) Excavation of river channel.

In order to increase the carrying capacity, the excavation of riverbed and partial widening will be conducted for the reaches from the vicinity of Merah bridge upstream to the Wonokromo sluice.

(3) Design high-water discharge and water level.

The design high-water discharge is decided as the following on the basis of 10-year flood studied in the Part dealing with Analysis and Study.

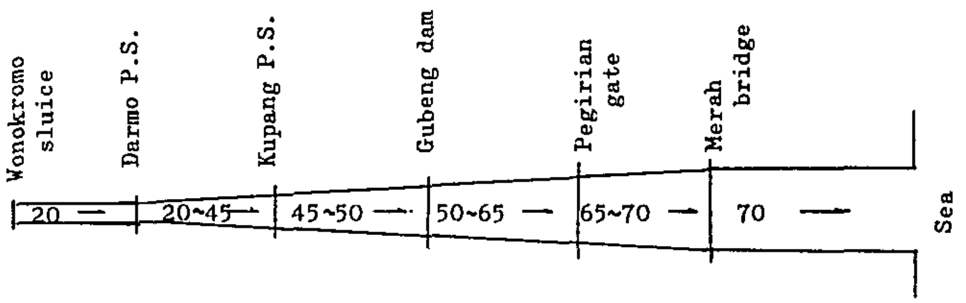


Fig. 2 shows the water level at the time when the above discharge flows through the Mas river after the aforesaid excavation has been executed.

(4) Revetment.

In order to protect the low-water channel, the low-water revetment will be provided to the reaches between P.17 and the Gubeng dam.

(5) Gubeng dam.

As the investigation of ground water is now under way and the improvement of the dam should be studied in the next stage of project, the Gubeng dam will be left as it is for the present stage.

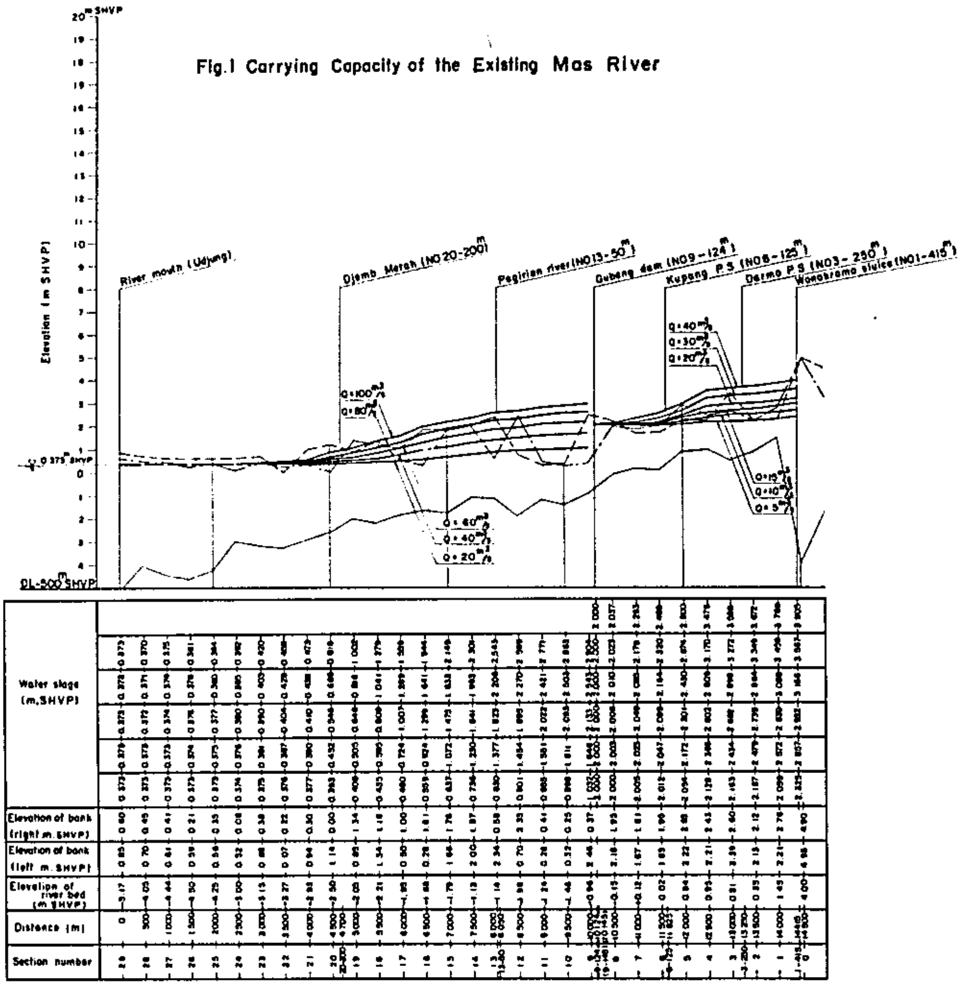


Fig.1 Carrying Capacity of the Existing Mas River

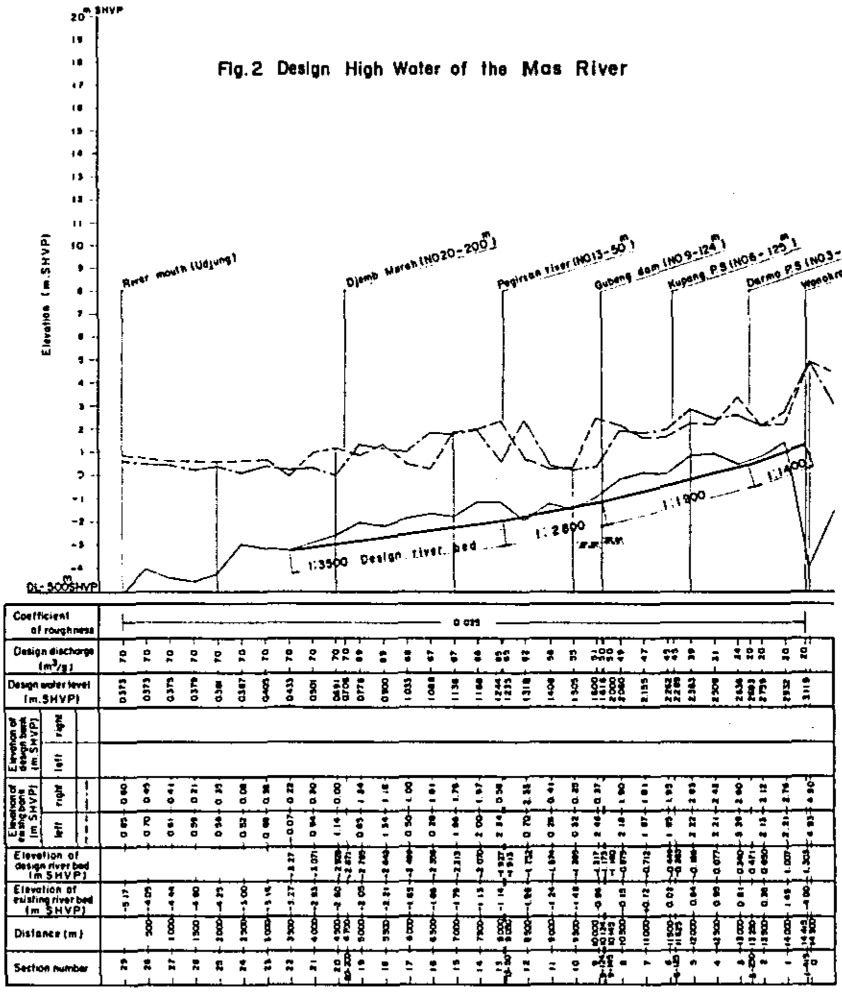


Fig.2 Design High Water of the Mas River

2. Construction Works.

(1) Quantity of works.

a.	Qt. of excavated spoil	210,000 m^3
b.	Area of land required	17,000 m^2
c.	L. of revetment	8,000 m

(2) Execution.

The construction period is 4 years and the works will be performed with one dragline of 64 m^3/hr.

The quantity of excavation per year calculated by the same method as the Marmojo river is

$$E = 64 \times 0.7 \times 6 \times 23 \times 10 = 61,824\ m^3/yr > 52,500\ m^3/yr$$

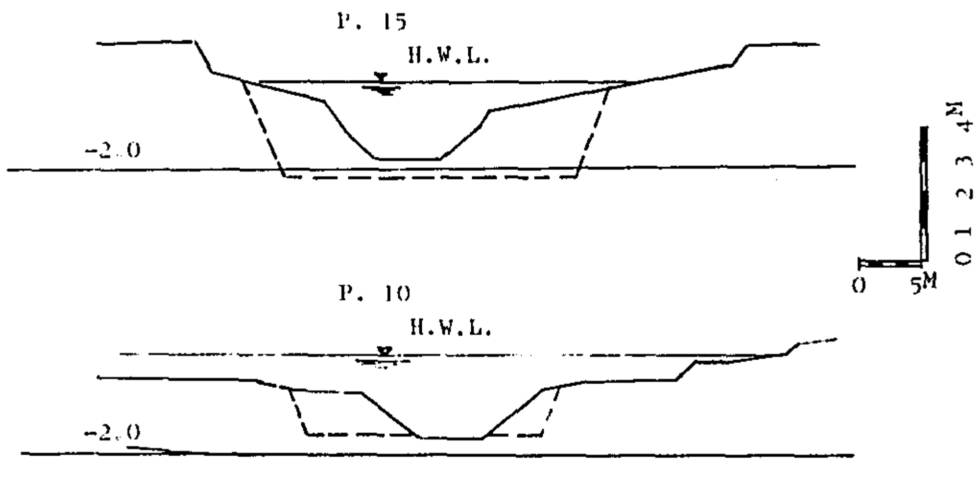
Also, 13 6-ton-dump trucks are used to transport the spoil. If the trucks make 6 return trips per day carrying earth of 3.7 m^3 per trip over the distance of 6 km, the annual quantity of earth transported at the operation rate of 80% is

$$T = 3.7 \times 0.8 \times 6 \times 23 \times 10 \times 13 = 53,102\ m^3 > 52,500\ m^3$$

Thus, using these equipment the works can be completed adequately within about 3 years. The works on revetment will be carried out by man power using only the small equipment of general types, without taking into consideration the use of particular machine.

(3) Typical cross-section.

The typical cross-section for design are as follows:



CHAPTER V

MOROKREMBANGAN BOEZEM

1. *Principles of Improvement.*

The principles of improvement of Morokrembangan Boezem are as follows:

(1) Improvement works of the Mitre gates.

Taking into consideration the corrosion and the frequent breakdowns of hinges due to sea waves, 3 Mitre gates at the outlet of the boezem will be improved by replacing the hinges with those of anticorrosion metal and the Mitre gates with those of stronger structure capable to stand against sea waves.

(2) Dredging works of the boezem.

Since the silt transported by the Greges river has been deposited around the river mouth over a long period of time, reducing the storage capacity of the boezem to a considerable extent, it shall be dredged to -2.0 m, or near to the low water level in order to secure the storage capacity as much as possible. On this occasion, 5-year flood was adopted for examination of storage capacity of the boezem, because it is conceivable that 5-year storm will be adopted for design of the urban drainage at the next stage.

It is expected from the study in Part 4 that the dredging will take effect on the improvement of drainage not only of the area around the boezem including the land between the two portions of the boezem but also of the Greges basin.

The area of such dredging is shown in Fig. 1.

(3) Reclamation.

At the same time as dredging, a proper low-lying land such as fish ponds near the boezem shall be reclaimed with dredged earth in an area of 16.4 ha and about 1.5 m in depth. It is naturally expected that this land will be utilized as a land capable of absorbing an increase of population.

(4) Partial change of drainage basin of the Greges river.

Out of 15.8 km^2 of the existing drainage basin of the Greges river, 1.9 km^2 of the middle and upper basin will be absorbed to the drainage basin of the Mas river in the next program for urban drainage system.

2. Construction Works.

(1) Quantity of construction.

a. Quantity of earth to be dredged	246,000 m^3
b. Area of land required	164,000 m^2

c. Improvement work of Mitre gates

This work will contain preparatory and temporary works, improvement of gate stopping, installation of gates, and miscellaneous materials.

(2) Execution.

The construction period is over 3 years for dredging and 1 year for the improvement of Mitre gates. If two pump dredgers of 24 m^3/hr in capacity is used, the annual quantity of dredged earth on the operation basis of 10 months/yr is

$$D = 24 \times 8 \times 23 \times 10 \times 2 = 88,320 \ m^3/yr > 82,000 \ m^3/yr$$

The improvement works of Mitre gates shown in Fig. 2 will be executed using proper equipment in the equipment list described later.

(3) Typical design sections of the boezem.

The typical design sections of the boezem is as follow:

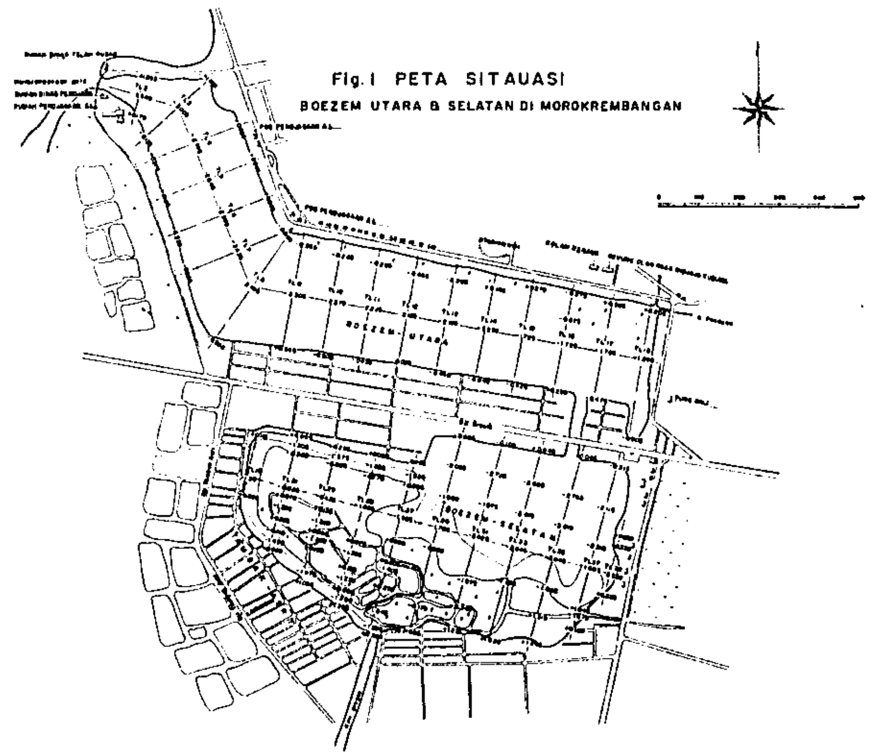


Fig. 2 IMPROVED MOROKREMBANGAN GATE

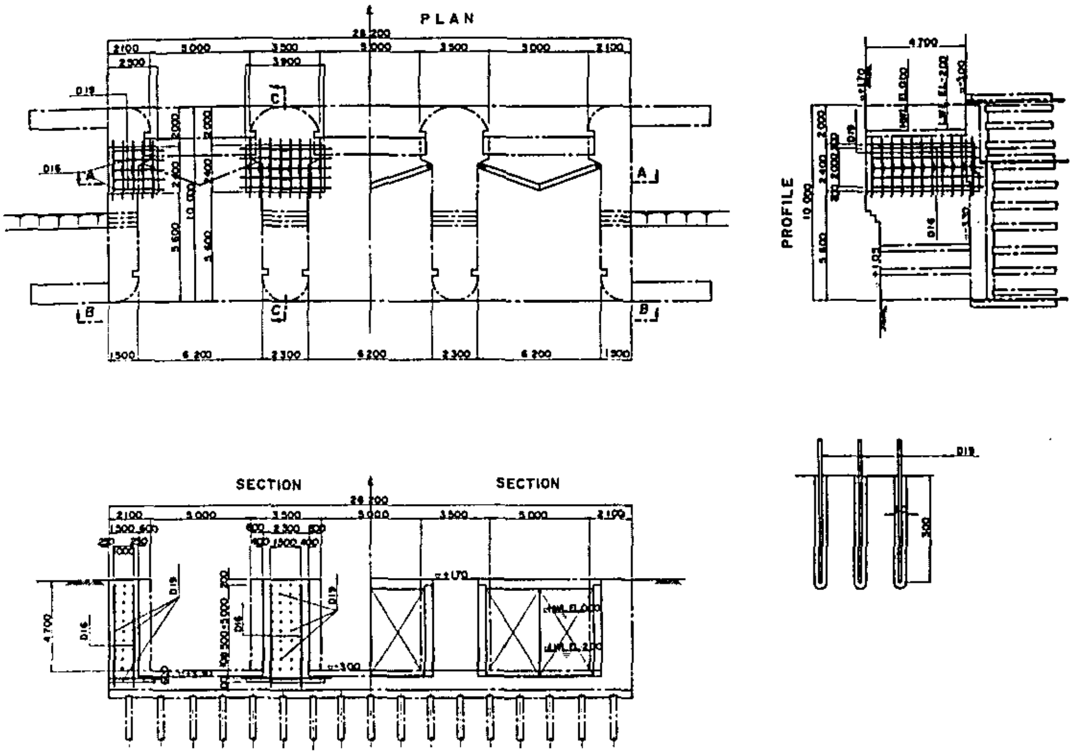
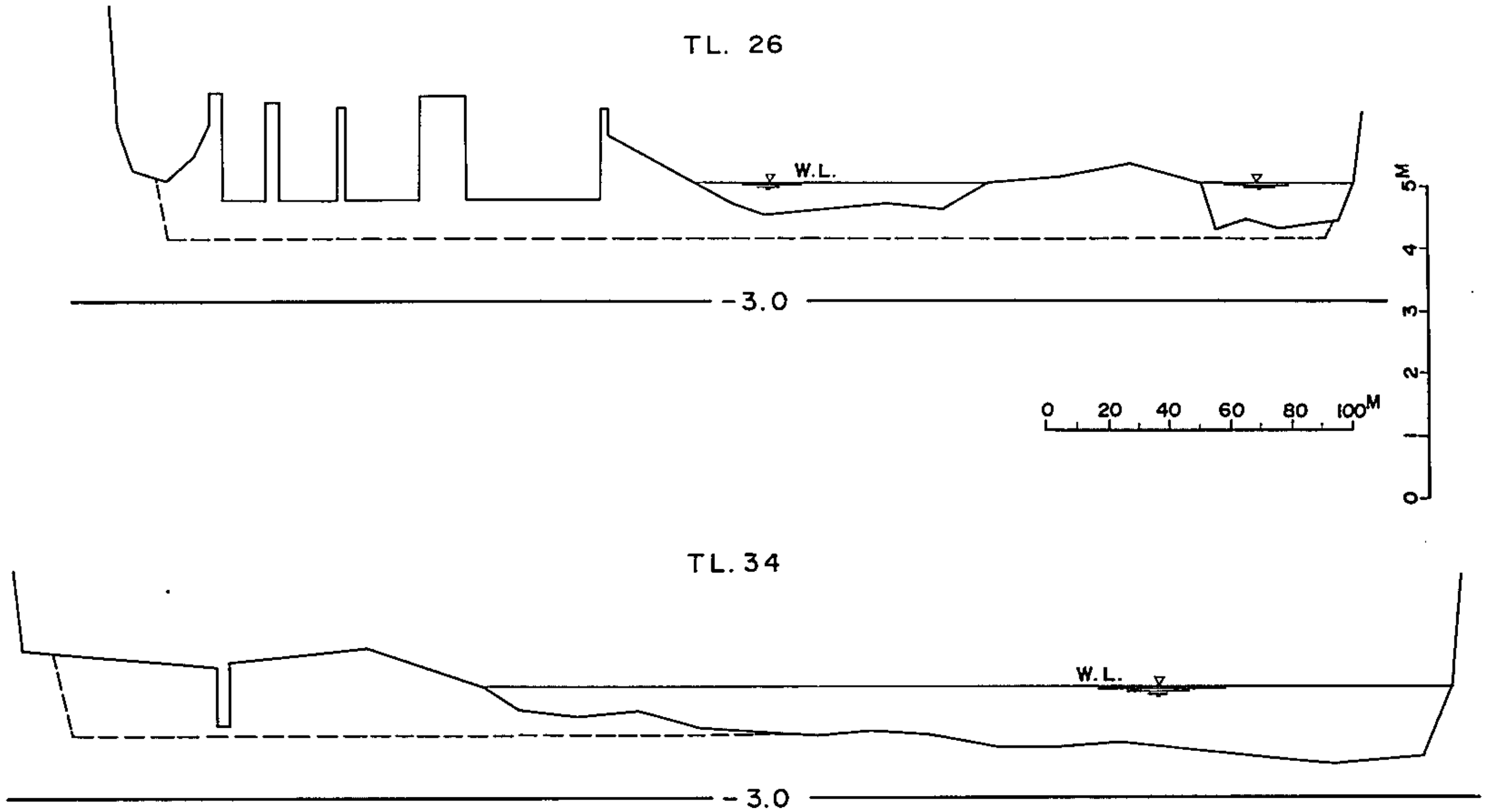


Fig. 3 Typical Cross Sections of Morokrembangan Boezem



CHAPTER VI

SEA DIKE AND SLUICES

1. Principle of Improvement.

a. The sea force due to waves and tides for the design of dike shall be of 50-year return period. This condition has given the following criterion for design.

tide leve + 0.44 m SHVP

wind velocity 25 knot = 13 m/s

as a result of study explained in Part 4.

b. The sea dike of about 17 km stretches from Semampir in the north up to Medokan in the south on the northeastern coast of Surabaja city. Of this length, two portions of the dike which total to 0.58 km at Sukolilo and Kalitindjang shall be raised and protected with revetment. The elevation of dike crest shall be +1.5 m SHVP according to the above-mentioned study.

c. The discharging capacity of sluices shall be of 5-year return period keeping pace with the urban drainage.

d. All the sluices which are located at the crossing points of drainage canals and the dike shall be renewed except Tambakwedi sluice, because the latter has a discharging capacity for only 9.6 km^2, accordingly, a new Tambakwedi sluice should be built next to the existing one for the drainage of 9.8 km^2.

e. Ten sluices shall be in charge of drainage of each area shown in the following.

Name of sluice	Catchment area (km^2)	Name of sluice	Catchment area (km^2)
Tambakwedi	9.6	Larangan	2.8
New Tambakwedi	9.8	Wonosari	4.7
Tjumpat		Kalidami	7.5
Kendjeran	8.0	Keputih	5.5
Sukolilo		Medokan	11.5

Of the above, the sluices of Tjumpat, Kendjerah, and Sukolilo are exposed to littoral draft. Therefore, sluice gates with motorized equipment shall be adopted for them. For the others, mitre gates shall be adopted.

f. The elevation of bottom face of conduit at the sluices of Tjumpat, Kendjeran, and Sukolilo shall be -1.0 m SHVP, that of Larangan, Wonosari, Kalidami, and Keputih shall be -1.5 m SHVP, and that of New Tambakwedi and Medokan shall be -2.2 m SHVP.

2. Construction Works.

(1) *Quantity of works.*

a. Embankment and revetment.

Sukolilo	230 m
Kalitindjang	350 m

b. Sluices 9

Standard structures of sluices to be adopted for the improvement are illustrated in Fig. 2 to Fig. 6 and nine sluices shall take new structures respectively as shown in the following.

Type A ; Sukolilo, Kendjeran, Tjumpat

Type B_1 ; Larangan, Wonosari, Keputih

Type B_2 ; Kalidami

Type C_1 ; New Tambakwedi

Type C_2 ; *Medokan*

The construction works will contain preparation and clearance, *foundation and water stopping*, concrete culvert, concrete-block channel, installation of gates and winches, and miscellaneous materials.

(2) Execution.

The sea dike in Sukolilo area shall be improved first and that in Kalitindjang area secondly in accordance with the standard shown in Fig. 1. Particular equipment is not considered in these works.

The northern 5 sluices of Larangan, Sukolilo, Kendjeran, Tjumpat, and New Tambakwedi shall be constructed in the first year, and the other 4 sluices shall be constructed in the second year.

Those works shall be done utilizing suitable construction equipment shown in the particulars for purchase described later.

Fig. 1 Standard Cross Section of Sea Dike for Improvement

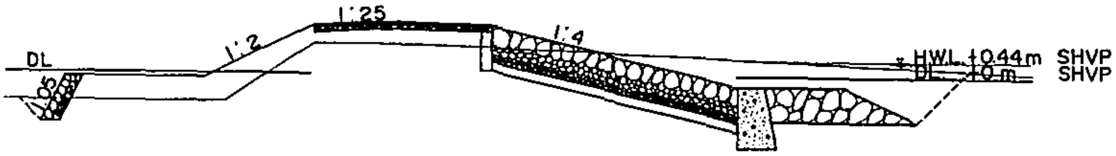


Fig. 2 NEW STRUCTURE OF SEA-DIKE SLUICE TYPE A(200^{m}x150^m)

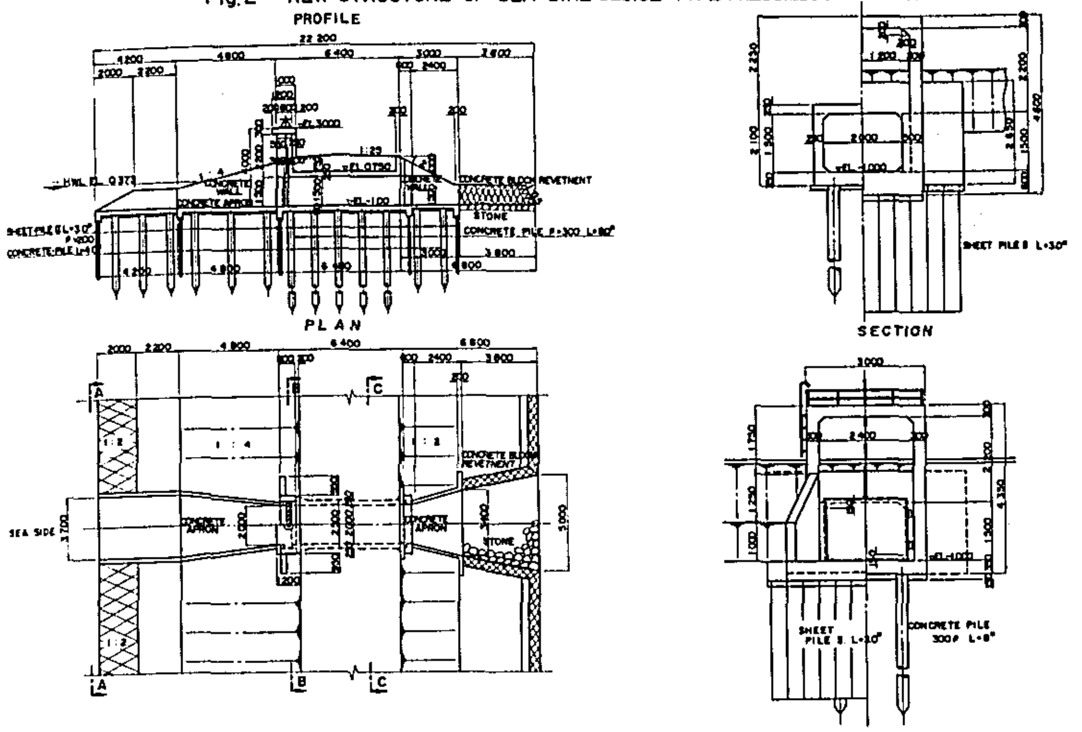


Fig. 3 NEW STRUCTURE OF SEA-DIKE SLUICE TYPE B$_1$ (300^{m}x200^m)

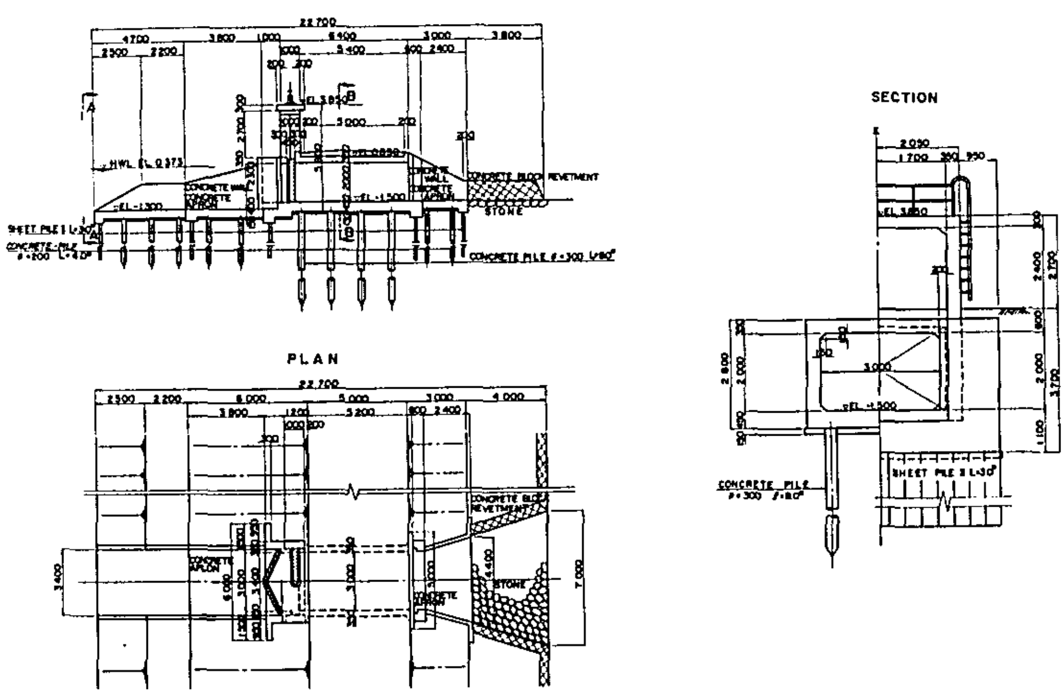


Fig.4 NEW STRUCTURE OF SEA-DIKE SLUICE TYPE $B_2(2x3.00^m x2.00^m)$

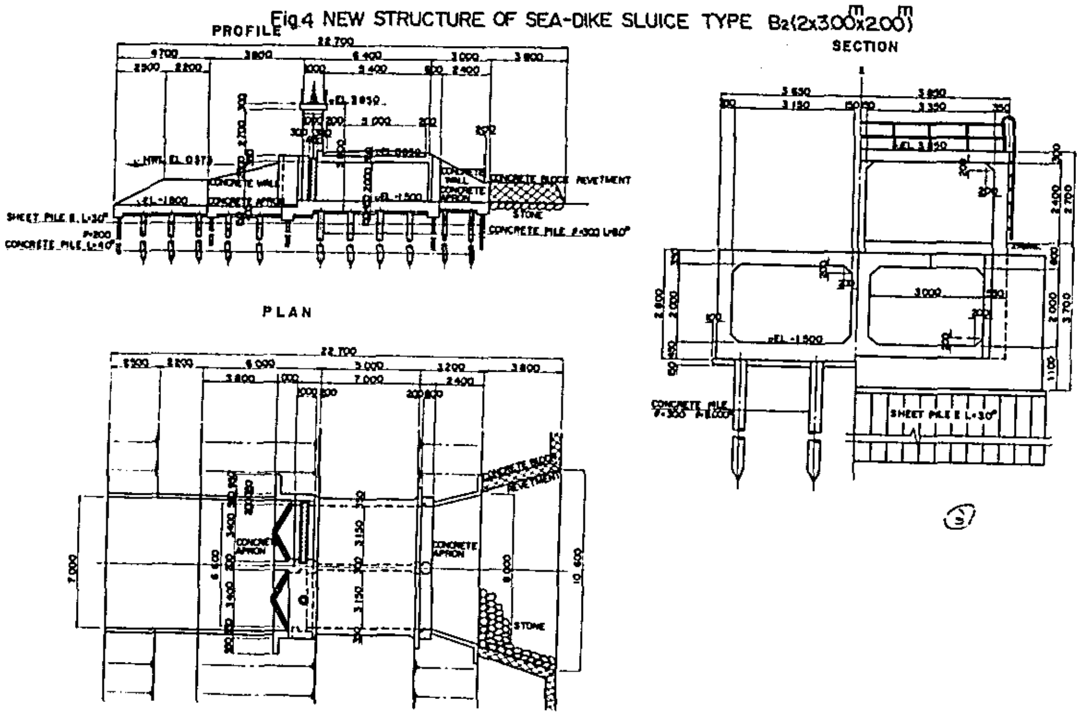


Fig.5 NEW STRUCTURE OF SEA-DIKE SLUICE TYPE $C_1(5.00^m x3.20^m)$

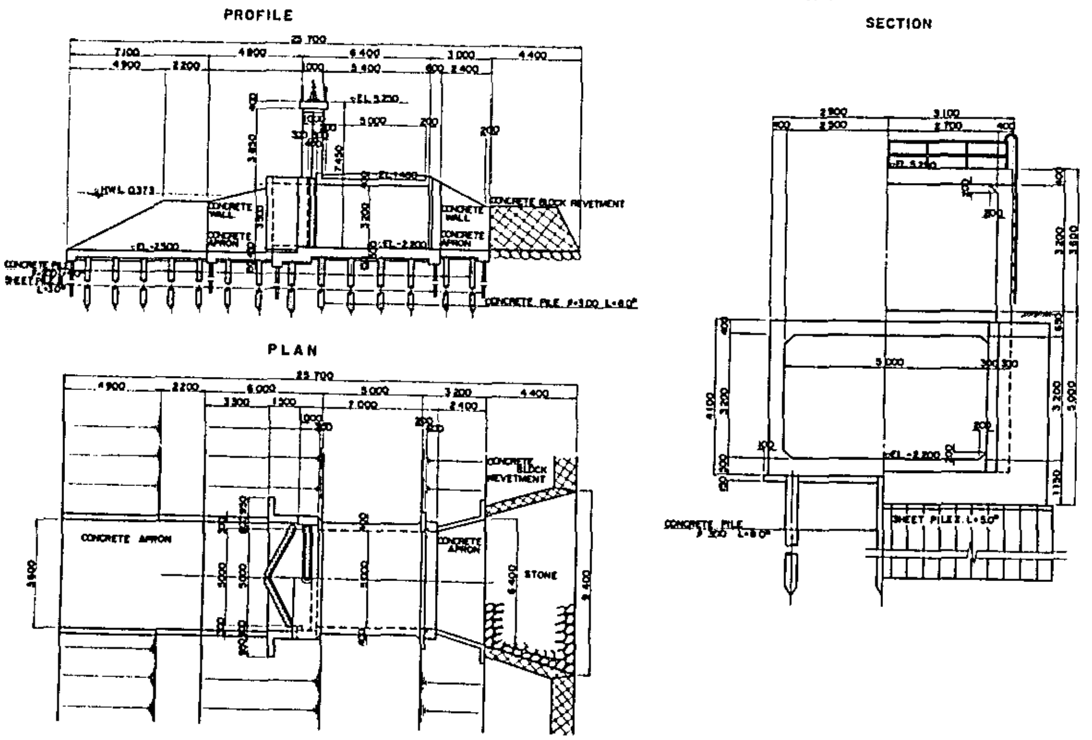


Fig.6 NEW STRUCTURE OF SEA-DIKE SLUICE TYPE $C_2 (2 \times 3.00^m \times 3.27^m)$

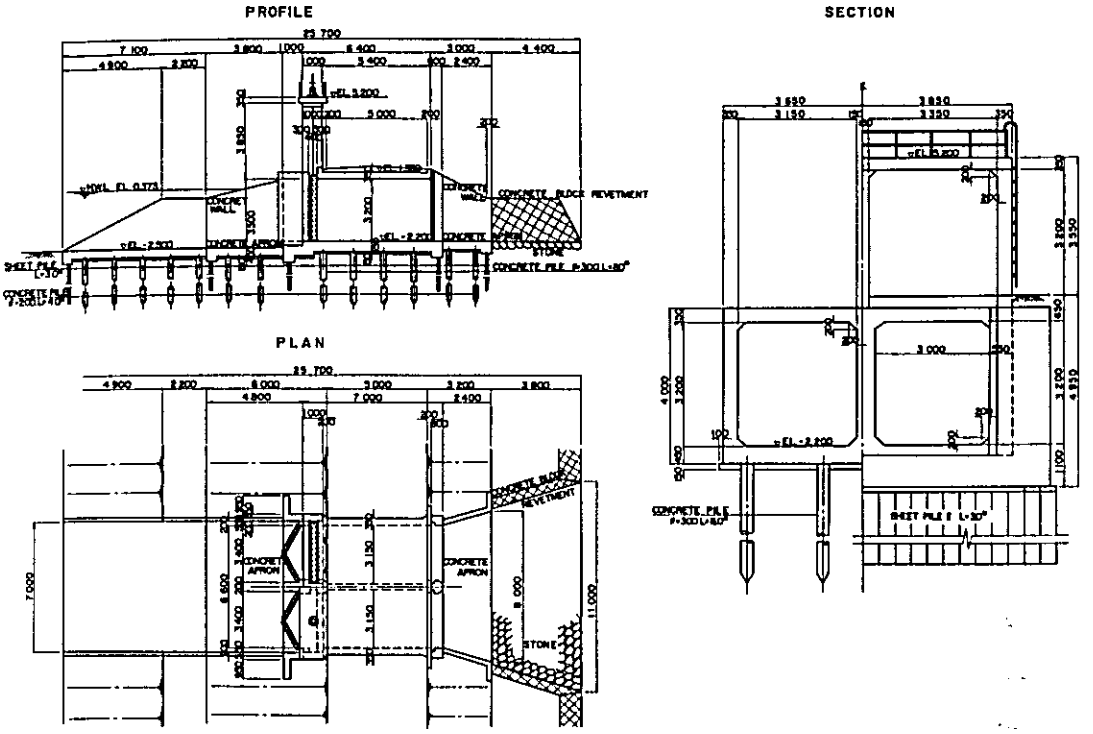
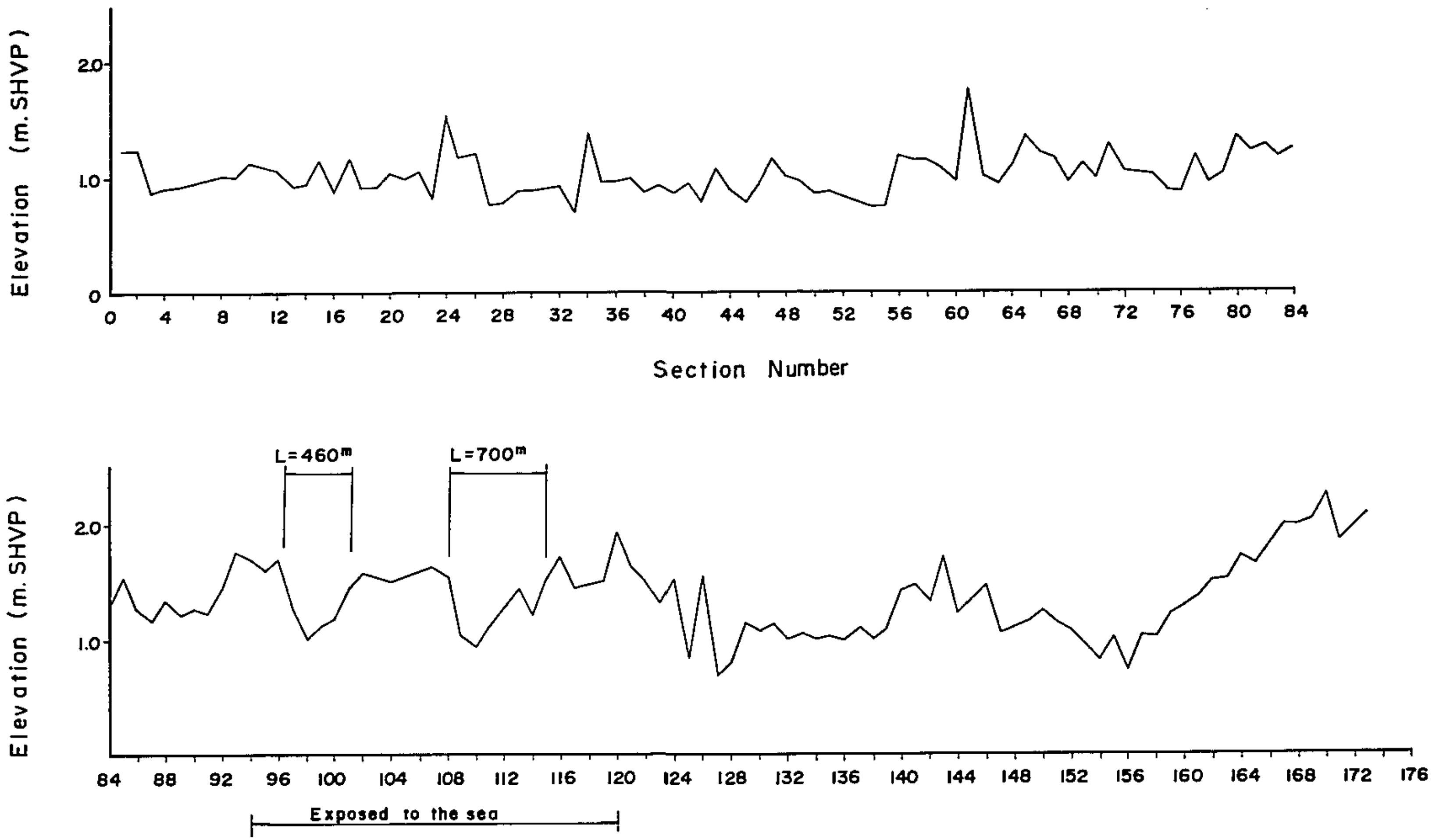


Fig. 7 Profile of Existing Dike



CHAPTER VII

APPURTENANT FACILITIES

1. Facilities for a Concrete-Block Yard and Concrete Tests.

Concrete blocks to be used for revetment and others should be mass-produced and controlled in quality at a yard in order to finish the work within a scheduled term and to insure the designed quality.

Essential facilities required for the concrete-block yard and the concrete tests are shown in Table 1 and 2. Flow of concrete-block production is illustrated in Fig. 1.

The concrete-test facilities will also be used for the programme of quality control of the other concrete work.

Table 1 Facilities for Concrete-Block Yard

Required equipment	Specifications	Quantity
Concrete mixer	0.5 m^3 tilt-type	1
Jaw crusher	1.2 t/h with engine	1
Moulds for blocks	$0.5^m \times 0.5^m \times 0.1^m$	300
Other equipment	-	suit
Water pool for curing	-	suit
Warehouse and others	-	suit

Table 2 Facilities for Concrete Tests

Required equipment	Specifications	Quantity
Compression testing machine	100 t	1
Concrete mixer	54 l	1
Other machine or apparatus	-	suit
Laboratory	-	1
Other facilities	-	1

2. Rain and Water Gaging Stations.

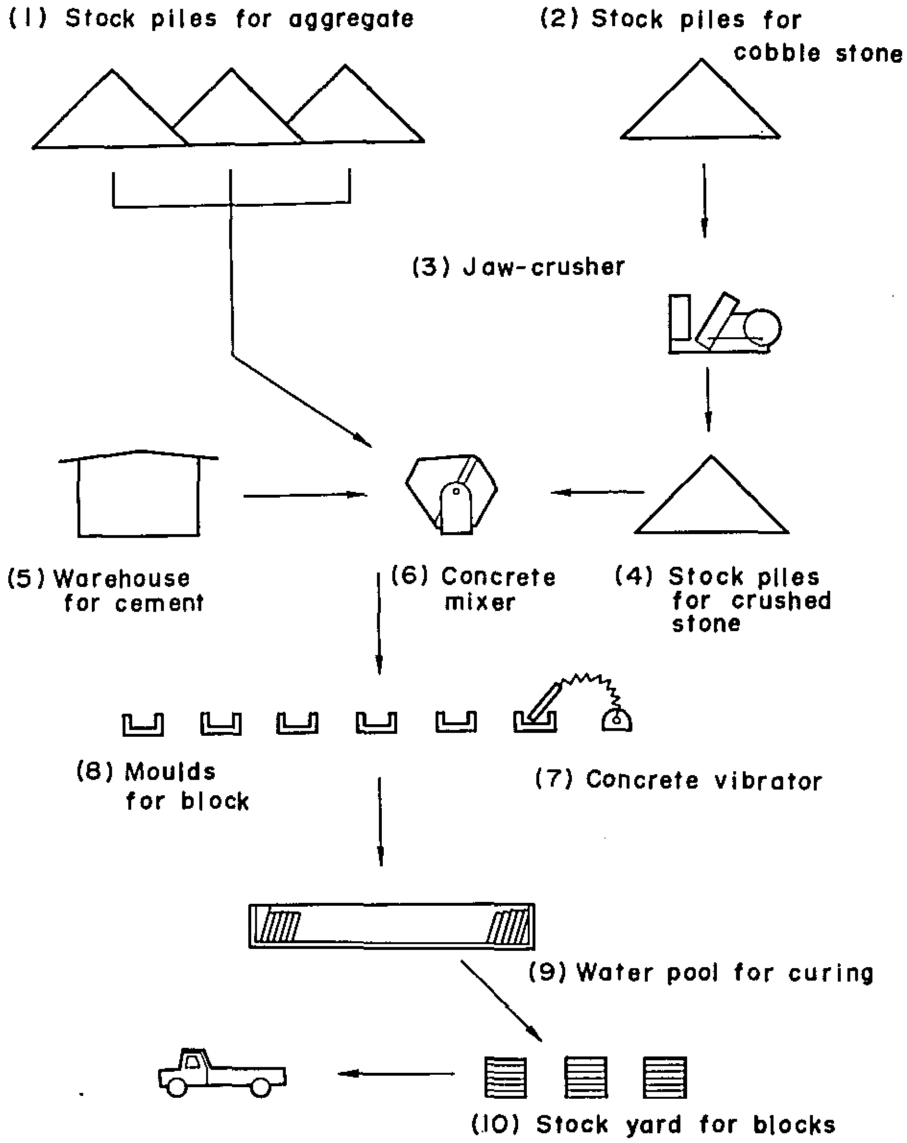
At present, in the project area, there are two recording-rain-gage stations, seventeen nonrecoring-rain-gage stations, one recording-water-gage station, and sixteen staff-gage stations which are as shown in Fig. 2.

However, six recording-rain-gage stations and nine recording-water-gage stations shall additonally be installed as shown also in Fig. 2, for the purpose of (1) management of construction works of this project, (2) management of rivers and maintenance of hydraulic structures after the completion of the project, and (3) further study of hydrology.

3. Communication Net.

Prompt communication of information plays an important role not only in flood control but also in water management. Considering the expected net

Fig. I Flow Diagram for Concrete Block Yard



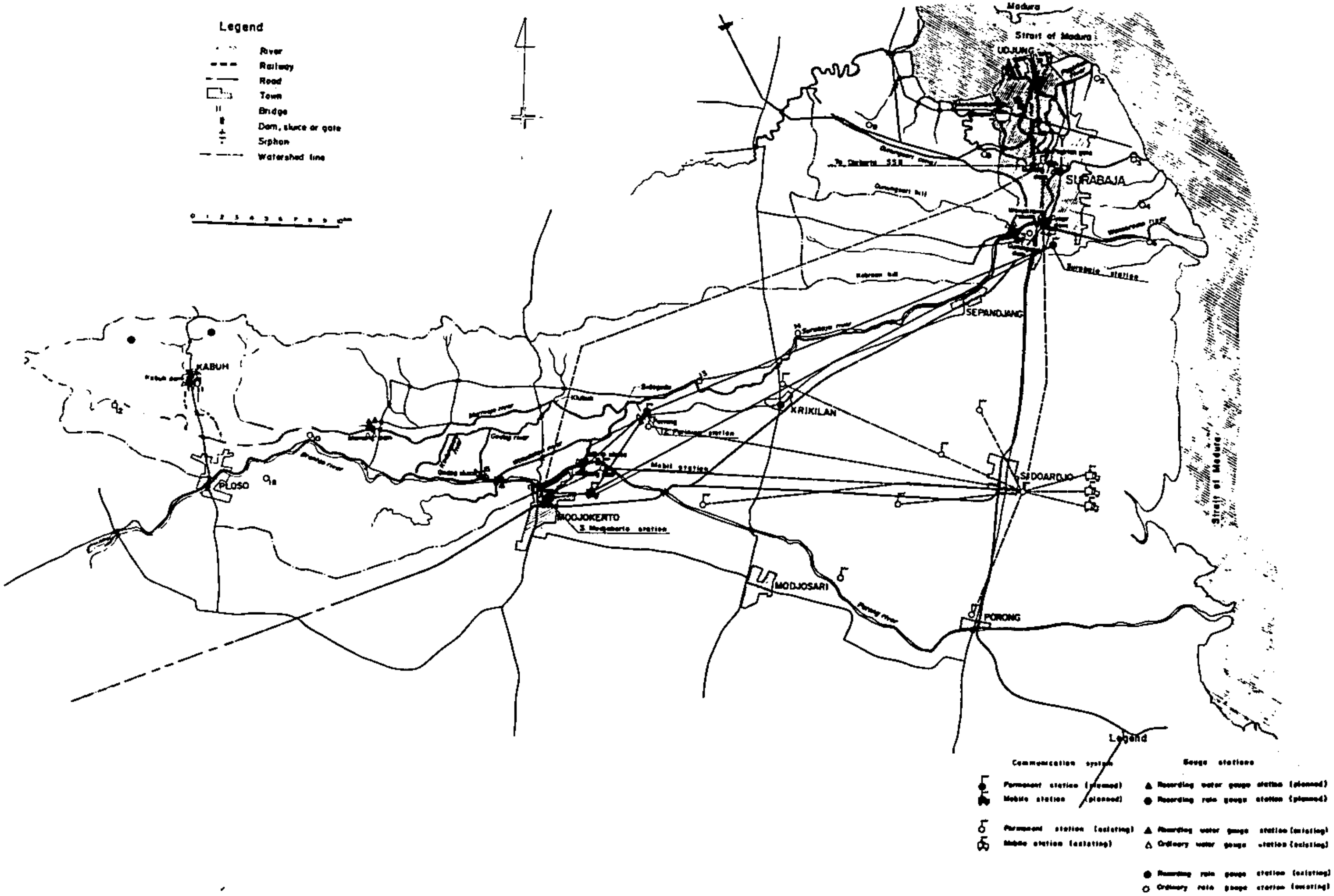
in Brantas Delta Irrigation Rehabilitation Project, three VHF-radio stations in Surabaja, Perning, and Modjokerto and one mobile station have been planned, and their disposition is as shown in Fig. 2.

These stations shall be provided with the equipment shown in Table 3 necessary for mutual communication.

Table 3 VHF Radio Equipment

Station	Specifications	Quantity
Surabaja station	150 MHZ FM Tranceiver 2 sets and others	suit
Perning station	150 MHZ FM Tranceiver 3 sets and others	suit
Modjokerto station	150 MHZ FM Tranceiver 2 sets and others	suit
Mobil station	150 MHZ FM Tranceiver 1 set and others	suit

Fig. 2 Disposition of Rain and Water Gage Stations and Communication Stations



CHAPTER VIII

CONSTRUCTION SCHEDULE

1. Construction Schedule.

The schedule for implementation of the works mentioned in the previous chapters is as follows:

Table 1 Construction Schedule of the Surabaja River Improvement Project

Item	1st Year	2nd Year	3rd Year	4th Year	5th Year	6th Year
Imp. works of Marmojo river						
Excavation & embankment						
Revetment & etc.						
Imp. works of Surabaja/Wonokromo river						
Embankment & revetment						
Mlirip sluice						
Gunungsari dam						
Djagir dam						
Imp. works of Mas river						
Excavation						
Revetment						
Imp. works of Sea dike						
Embankment & revetment						
Sluices						
Imp. works of Morokrembangan boezem						
Dredging						
Mitre gates						
Equipment and materials (purchase)						
Appurtenant facilities (construction)						
Concrete facilities						
Rain & Water gaging stations						
Communication net						
Supervision						
Detail design & on-the-spot survey						

2. Equipment and Materials.

Equipment and materials required for the project are shown in the following table.

Table 2 Equipment and Materials Required for the Surabaja River Improvement Project

Equipment and materials	Specification	Quantity	Foreign cur. 10^3 Yen	Residual value 10^3 Yen
(1) Construction equipment			176,430	16,943
Hoe	$0.6m^3, 64m^3/hr$	1	12,000	1,200
Dump truck	6 ton	10	30,000	3,000
Truck crane	10 ton	1	12,000	1,200
Truck crane	20 ton	1	22,000	2,200
Diesel hammer	4 ton	1	13,000	1,300
Tower crane	3 - 12.5 ton	1	3,000	300
Concrete mixer	$0.5\ m^3$	2	2,600	260
Air-compressor	52.5 P.S.	2	4,000	400
Electric welding machine	300 A	2	600	60
Pick hammer		10	800	80
Vibrator		10	500	50
Belt-conveyor	$400^{mm} \times 10^{m}$	10	3,000	300
Pump	3.7 KW	8	1,200	120
Winch	7.5 KW	3	1,200	120
Diesel generator	12.5 KW	2	2,600	260
Jeep		3	4,500	450
Land cruiser		4	6,000	600
Spare parts of Dragline		suit	5,400	540
Spare parts of Dump truck		suit	4,400	440
Spare parts of Dredger		suit	30,000	3,000
Spare parts of Ordinary truck		suit	630	63
Other equipment		suit	10,000	1,000
Spare parts of above items		suit	7,000	0
(2) Construction materials			479,390	0
Gate and appurtenant			325,865	0
Steel and others			153,525	0
(3) Other equipment			46,150	0
Surveying instrument			11,000	0
Stationeries			6,000	0
General equipment			4,900	0
Recording-rain-gage		6	2,220	0
Recording-water-gage		9	4,230	0
Wireless telephone		suit	8,400	0
Concrete apparatus		suit	9,300	0
Total			701,970	16,943

3. Construction Costs.

The Construction costs required for the project are shown in the following table.

Table 3 Construction Costs for the Surabaja River Improvement Project

Item	Particulars	Quantity	Unit price (Rp)	Domestic cur. ($x10^3$ Rp)	Foreign cur. ($x10^3$ Yen)
Improvement works of the Marmojo river	Excavation	320,000 m^3	193	61,760	
	Embankment	15,000 m^3	420	6,300	
	Land	152,500 m^2	60	9,150	
	Revetment	1,000 m	22,400	22,400	
	Appurtenant works	1 suit		10,390	
Improvement works of the Surabaja/Wonokromo river	Embankment	10,000 m	2,100	21,000	
	Revetment	1,290 m	74,419	96,000	
	Mlirip sluice	1 suit		50,110	48,110
	Gunungsari dam	1 suit		333,460	193,310
	Djagir dam	1 suit		120,330	25,790
Improvement works of the Mas river	Excavation	210,000 m^3	319	66,990	
	Land	17,000 m^2	250	4,250	
	Revetment	8,000 m	18,595	148,760	
Improvement works of the Sea dike	Embank. & revetment	580 m	69,000	40,020	
	Sluices	1 suit		77,268	36,925
Improvement works of the Morokrembangan boezem	Dredging	246,000 m^3	188	46,248	
	Land	164,000 m^2	60	9,840	
	Others	1 suit		3,912	
	Mitre gate	1 suit		36,160	21,730
Material and equipment		1 suit		75,100	329,955
Appurtenant facilities	Concrete facilities	1 suit		3,590	9,300
	Rain & water gaging S.	1 suit		19,800	6,450
	Communication net	1 suit			8,400
Other equipment		1 suit			22,000
Supervision				133,962	379,705
Contingencies for works				155,200	120,186
Subtotal				1,552,000	1,201,861
Detail design & on the spot survey				32,146	177,629
Contingencies for engineering services				3,572	19,737
Subtotal				35,718	197,366
Total				1,587,718	1,399,227
$ 8,368,772 eq.				$ 3,825,827 eq.	$ 4,542,945 eq.

Table 4 Annual Distribution of the Construction Costs for the Surabaja River Improvement Project

		1st Year		2nd Year		3rd Year		4th Year		5th Year		6th Year		Total	
		Dom. 10^3 Rp	For. 10^3 Yen	Dom. 10^3 Rp	For. 10^3 Yen	Dom. 10^3 Rp	For. 10^3 Yen	Dom. 10^3 Rp	For. 10^3 Yen	Dom. 10^3 Rp	For. 10^3 Yen	Dom. 10^3 Rp	For. 10^3 Yen	Dom. 10^3 Rp	For. 10^3 Yen
Imp. works of the Marmojo R.	Excav. & embank									38,605		38,605		77,210	
	Revetm. & others									16,395		16,395		32,790	
Imp. works of The Surabaja/ Wonokromo R.	Embank & revetm.							58,500		58,500				117,000	
	Mlirip sluice										48,110	50,110		50,110	48,110
	Gunungsari dam						64,437	111,153	64,437	111,153	64,436	111,154		333,460	193,310
	Djagir dam				25,790	120,330								120,330	25,790
Imp. works of the Mas R.	Excavation					17,810		17,810		17,810		17,810		71,240	
	Revetment									74,380		74,380		148,760	
Imp. works of Sea dike	Embank & revetm.			10,900		14,560		14,560						40,020	
	Sluices				18,462	38,634	18,463	38,634						77,268	36,925
Imp. works of M. Boezem	Dredging			12,000		16,000		16,000		16,000				60,000	
	Mitre gates						21,730	36,160						36,160	21,730
Equip. & Mat.		500	7,500	74,600	322,455									75,100	329,955
Appurtenant facilities	Concrete facilities			3,590	9,300									3,590	9,300
	Rain & water g.s.										6,450	19,800		19,800	6,450
	Communication net										8,400				8,400
Other equip.			21,573								427				22,000
Supervision				13,072	29,124	35,732	87,531	35,732	107,932	26,767	92,902	22,659	62,216	133,962	379,705
Conting. for works		56	3,230	12,685	45,015	27,007	21,351	36,505	19,152	39,957	24,525	38,990	6,913	155,200	120,186
Subtotal		556	32,303	126,847	450,146	270,073	213,512	365,054	191,521	399,567	245,250	389,903	69,129	1,552,000	1,201,861
Detail design		15,214	47,524	16,932	130,105									32,146	177,629
Conting for eng.		1,691	5,281	1,881	14,456									3,572	19,737
Subtotal		16,905	52,805	18,813	144,561									35,718	197,366
Total		17,461	85,108	145,660	594,707	270,073	213,512	365,054	191,521	399,567	245,250	389,903	69,129	1,587,718	1,399,227

CHAPTER IX

ECONOMIC STUDY

1. Estimation of Benefits.

(1) Definition of benefits.

Benefits of river improvement project may generally be classified according to two different criteria, direct benefits and indirect benefits. Direct benefits are the value of the immediate products and services resulting from the project, such as reduction of damages to properties, prevention of decrease in production, conversion of land use and the stabilization of people's livelihood. Indirect benefits are the value in addition to the direct benefits as a result of activities derived from or induced by the project, such as stimulation to agricultural production, extension of social and economic activities of inhabitants, increase in profits of all enterprises that supply goods to or purchase products from those people who received the direct benefits.

Out of the benefits mentioned above, direct and tangible benefits, such as reduction of damages to properties, prevention of decrease in production and conversion of land use were adopted for the present project. Tangible benefit means a benefit that can be expressed in monetary terms.

Flood damages were analyzed by a method of possible floods which relate to the probability of exceedance of flood and the amount of damage. Reduction of damage which results from improvement works was obtained as the difference of flood damages between before and after the improvement works.

The economic benefits for the present project were first estimated for each of five construction works; the Marmojo river, the Surabaja/Wonokromo river, the Mas river, Morokrembangan Boezem and sea dike works, and benefit-cost analysis has been made with regard to the total benefit of the above works and the total cost to be invested in the works described in the preceeding chapter. In this case, the economic justification was made by comparing the present value of benefits with the present value of costs.

(2) The Marmojo River Improvement Works.

Amount of damages caused by floods in this area consists of loss of properties and decrease of amount of valuation of properties. They were estimated by a method of possible floods, adopting the return-periods of 20, 10, 5 and 2 years. Although 20-year flood has been adopted for the improvement of this river, inundation in the basin will extremely be reduced, because the Marmojo river is improved in the present project and the Brantas flood into the Marmojo river shall be stopped after the completion of New Lengkong Dam. Fig. 1 shows the area which may be inundated by 20-year flood, if the improvement works will not be carried out. The total area of inundation covers about 520 ha which comprize farm land of 395 ha and residential area of 125 ha, and the maximum submergence depth may reach 107 cm. Number of farmhouses in the inundated area were estimated at about 1,096. Table 1 shows number of farmhouses and area of paddy field together with ground height for each mesh in the inundated area.

Fig. I Inundation Area in
Right Lower Basin of the Mamojo River
(for 20-year flood)

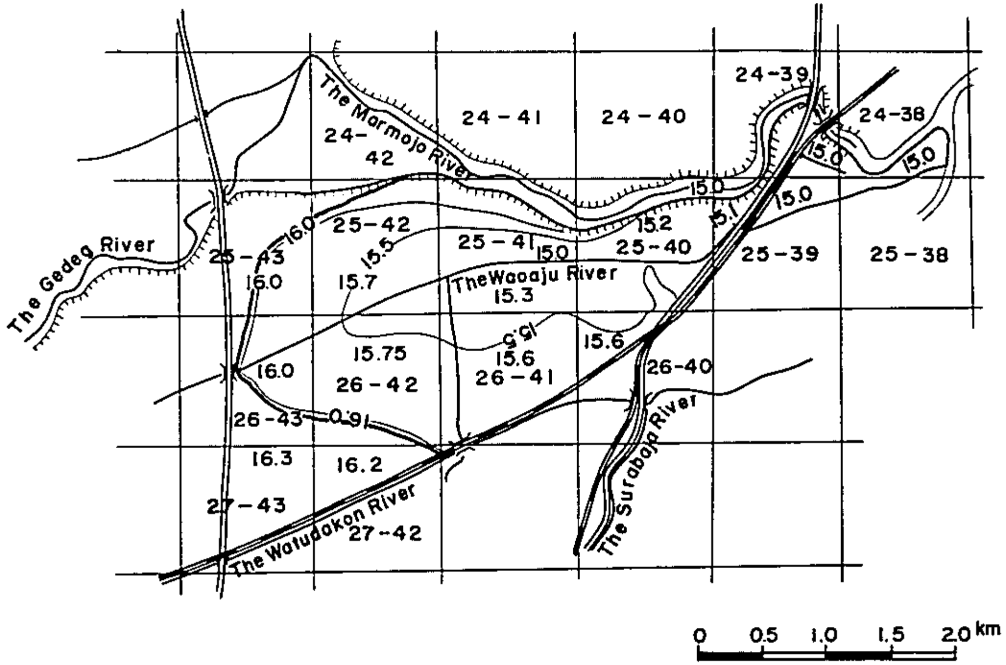


Table 1 Ground Height and Properties in the Inundated Area

No. of mesh	Ground height (m)	Number of houses			Area of paddy field (ha)
		Farmhouse	Office	School	
25-39	15.1	15 (15)	-	-	10 (10)
25-40	15.2	16 (11)	-	-	51 (46)
25-41	15.3	194(194)	-	-	68 (41)
25-42	15.7	323	1	-	68
25-43	16.0	206	1	1	16
26-40	15.6	96	-	-	16
26-41	15.6	124	1	-	56
26-42	15.7	10	-	-	80
26-43	15.9	112	-	-	30
Total		1,096	3	1	395

note : Figures in ( ) show number of farmhouses and area of planted paddy which may be lost by the strong flow of water.

Flood damages to properties shown in Table 1 were obtained from two kinds of damages; loss of properties due to flood flow and decrease of valuation of properties damaged by submergence. However, the damage after the improvement work will be caused only by the inundation of the rain water.

It was assumed that loss of properties due to flood flow may occur in the low-lying zone along the Marmojo river owing to the strong flow of water which has overtopped the right dike at the time of the flooding of the Marmojo. The estimated losses of properties under this condition were nearly 220 farmhouses and planted paddy of 97 ha.

Besides, damages due to inundation will occur to properties shown in Table 1. According to Chapter XVII, Part 4, the highest water level in the area inundated by overflows from the Marmojo river and rain water fallen to this area are as follows.

Highest Water Level

unit : m

Return period (years)	20	10	5	2
Before improvement	16.07	15.85	15.61	15.56
After improvement	15.65	15.63	15.61	15.56

The amount of damages were calculated under the following conditions and assumptions.

a. Amount of valuation of a house.

unit : Rp 1,000

Kind of properties	Farmhouse	Office	School
Building	430	2020	3500
Household effects	100	760	460

note : see Table 3 of Chapter XXIV, Part 4.

b. Unit yield and unit price of dry stalked paddy were assumed as follows,

(a) Unit yield : 4,700 kg/ha,

(b) Unit price : Rp 15/kg,

where unit yield was quoted from that of Ketjamatan Tarik because the data collected by the field survey in the basin were not available for the present purpose.

c. Others.

(a) Usual water level in a paddy field was assumed at 10 cm above ground surface.

(b) Floor level of a house was assumed at 10 cm above ground surface.

(c) Submergence depth harmless to planted paddy was assumed under 30% of its height.

(d) Duration of flood was assumed at 3.5 days according to Chapter XVII, Part 4.

Flood damages to planted paddy were calculated for each of the four months; December, January, February and March, for the purpose of finding the largest damage throughout a year. In this case, areas and height of planted paddy to be used in the calculation of amount of damage for each of the above months are as follows.

Month	Dec.	Jan.	Feb.	Mar.
Ratio of area of planted paddy to cultivated area	0.196	0.609	0.775	0.803
Plant height (m)	0.40	0.57	0.91	1.20

note : The above figures were quoted from those of the Brantas Delta shown in Table 2-25 of Chapter XXII, Part 4.

The amount of the average annual flood damages for each of the said four months were calculated using an electronic computer according to the way described in Chapter XXIV, Part 4, under the above conditions and assumptions. The results are shown in Table 2.

Table 2 Amount of the Average Annual Flood Damage

unit: Rp 1,000

Month	Dec.	Jan.	Feb.	Mar.
Before improvement	13,574	13,947	13,977	14,245
After improvement	1,698	1,792	1,710	1,979
Difference	11,876	12,155	12,267	12,266

It is seen from Table 2 that the difference of the damages between before and after improvement takes the largest value in February. Consequently, the amount of decrease of flood damage of Rp 12,267,000 in February was adopted as the economic benefit for the Marmojo river improvement works.

(3) The Surabaja/Wonokromo River Improvement Works.

The economic benefits for these improvement works are represented mainly by these from the improvement work of Gunungsari dam and divided into two different categories, the prevention of flood and the security of irrigation.

The former presents a benefit that may be brought by modernization of the operation of gates of Gunungsari dam and Djagir dam, while the latter presents a benefit that may be brought by rehabilitation of the function of extremely superannuated Gunungsari dam.

The damages due to floods were estimated by the method of possible flooding just similarly to the case of the Marmojo river improvement works, and return-periods of floods of 50, 20, 10 and 5-year were adopted for estimation of reasonable amount of annual damage.

Fig. 2 shows the area which may be inundated by 50-year flood, if modernization of the operation of gates of Gunungsari and Djagir dams will not be made. Although the whole inundated area may cover about 1,300 ha as shown in Fig. 2 the area which may be submerged over 0.20 m in depth was estimated at about 400 ha. Table 3 shows submergence depth of each mesh which was made from Figs. 2-1, 2-2 and 2-3 of Chapter XVIII, Part 4.

Table 3 Submergence Depths due to Floods of the Surabaja River

unit : m

No. of mesh	Return period (year)			
	50	20	10	5
9-10	0.35	0.25	0.23	0
9-11	0.35	0.25	0.23	0
10-10	0.40	0.34	0.33	0
10-11	0.37	0.33	0.28	0
11-10	0.45	0.38	0.37	0
11-11	0.55	0.45	0.40	0
12-11	0.58	0.50	0.42	0
12-12	0.62	0.55	0.43	0
13-12	0.65	0.62	0.50	0

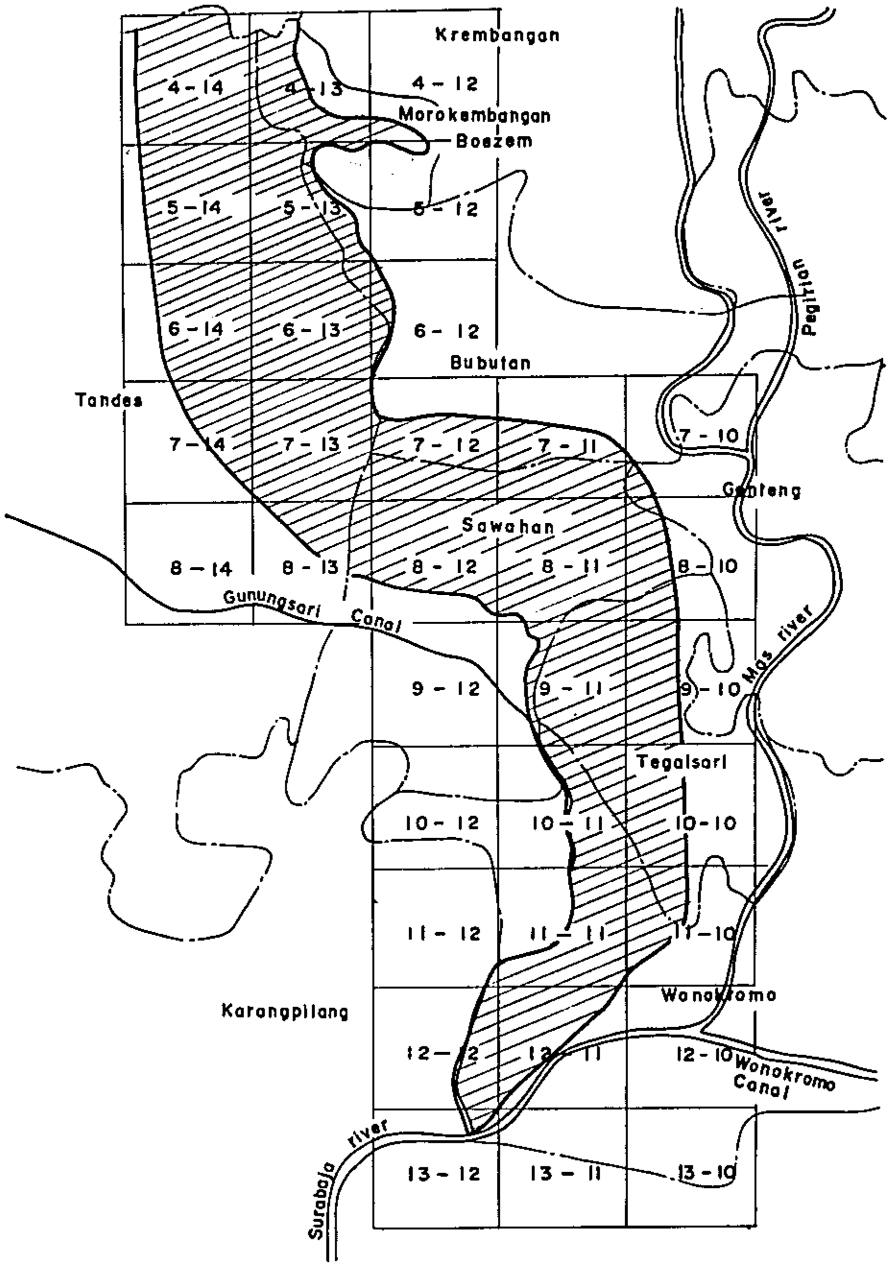
note : No. of mesh are shown in Fig. 1 of Chapter XXIV, Part 4.

Most of direct damages in the present area were composed of decrease of valuation of submerged properties such as buildings and household effects. Table 4 shows number of submerged houses which were classified according to the kinds in the inundated area shown in Table 3. Darmo region in the inundated area is one of the highest class residential area in Surabaja. Accordingly, it is expected that the area suffers huge damages, once it be inundated by floods.

Table 4 Number of Houses in the Inundated Area

No. of mesh	Office	Residence	Shop	Factory	School
9-10	2	1981	161	-	-
9-11	2	2227	181	-	-
10-10	2	2298	186	-	-
10-11	3	2368	192	1	-

Fig. 2 Area to be Inundated due to Gunungsari Dam Flood



11-10	1	1130	93	9	1
11-11	2	2136	177	44	1
12-11	-	184	20	68	-
12-12	-	115	9	90	-
13-12	-	68	6	5	-
Total	12	12,507	1,025	217	2

The amount of damages were calculated under the following conditions and assumptions.

a. For the valuation of a house, the following values were adopted according to Table 3 of Chapter XXIV, Part 4.

unit : Rp 1,000

Kind of properties	Office	Residence High class	Residence Middle class	Shop	Factory	School
Building	2,020	9,850	1,850	440	3,300	3,500
Household effects	760	4,680	480	800	1,800	460

b. It was assumed that number of residences shown in Table 4 consists of high-class and middle-class ones, and ratio of the former to the latter was assumed at 1 : 2 on the basis of aerophotographs which were provided by DPUT.

c. Floor level of house was assumed at 20 cm above ground surface.

Under the above conditions and assumptions, the amount of the average annual flood damage was calculated by using a computer according to the way described in Chapter XXIV, Part 4, and was estimated at Rp 271,240,000.

On the other hand, a benefit which may be brought by recovering the function of extremely superannuated Gunungsari dam was studied in detail in Chapter XXV, Part 4. Namely, if existing Gunungsari dam which have been being utilized for the purpose of obtaining irrigation water should be destroyed by reason of superannuation of function, the irrigation area would be unable to obtain water from the Surabaja river and would be obliged to be converted into rainfed farming. When new Gunungsari dam is constructed for the purpose of improving the superannuated facilities, the present irrigation may still be continued in the future.

Accordingly, economic benefit for the construction of new Gunungsari dam is obtained as the difference of yield of agricultural products between irrigation farming and rainfed one.

As a result, the amount of annual benefits for each year in the fifty years from present was calculated in consideration of the decrease of farming area in the future as shown in Table 2 of Chapter XXV, Part 4. Those results in this table are used for benefit-cost analysis described later.

According to the said table, the amount of average annual benefit was estimated at about Rp 85,000,000. Accordingly, the total sum of benefits for two works, modernization of the operation of gates and reconstruction of the body of Gunungsari dam was estimated at about Rp 356,000,000.

(4) The Mas River Improvement Works.

The economic benefits for these improvement works were obtained as decrease of inundation damages in the Mas river basin caused by local rainfall, or as the difference of inundation damages between before and after the improvement works.

The inundation damages were calculated using the method similar to the previous studies according to possible floods of 10, 5 and 2-year return-periods.

The total inundated area due to the 10-year flood was estimated at nearly 70 ha, which comprize the Darmo region of about 30 ha, the Kupang region of about 30 ha and the Kedondong region of about 10 ha.

The maximum depths of submergence for each return period before and after improvement works were estimated in the study made in Chapter XIX, Part 4. These are shown in the following table.

The Maximum Depth of Submergence Unit: m

Return period (years)	10	5	2
Before improvement works	0.49	0.48	0.25
After improvement works	0.38	0.37	0.23

Most damages due to inundation were composed of those of submerged properties such as buildings and household effects. Table 5 shows number of houses which were classified according to the kinds in each inundated area.

Table 5 Number of Houses in the Inundated Area

Region	Office	Residence		Shop	Factory
		High class	Middle class		
Darmo	1	934	234	11	17
Kupang	1	681	680	51	4
Kedondong	0	238	238	18	0
Total	2	1,853	1,152	80	21

note : Number of houses were counted using aerophotographs which were provided by DPUT.

The amount of damages was calculated under the following conditions and assumptions.

a. For the valuation of properties such as buildings and household effects, the same values as the preceeding section were adopted.

b. Floor level of a house was assumed at 20 cm above ground surface.

The amount of the average annual flood damage was calculated under the above conditions and assumptions using the method similar to the previous section, and was shown in Table 6.

Table 6 Amount of the Average Annual Flood Damage

	unit : Rp 1,000
Before improvement	412,131
After improvement	105,685
Difference	306,446

Consequently, the amount of decrease of the average annual flood damage was estimated at Rp 306,446,000 as shown in Table 6. This value was adopted as economic benefit for the Mas river improvement works.

(5) Morokrembangan Boezem Improvement Works.

The economic benefits resulting from the present improvement works may be derived from new land to be formed by spoil of dredging work.

According to the study in Chapter V, Part 2, it was programed that a proper low-lying land near the boezem would be reclaimed with dredged earth in an area of 16.4 ha. It is desired that the reclaimed ground will be utilized as a land of residential use as a countermeasure of population increase of Surabaja city.

Unit price of land was assumed at Rp 3,000/m^2 in consideration of the situation of Surabaja city and its surrounding areas and the opinion of officials of the authorities concerned. As a result, the value of land to be reclaimed with dredged earth was estimated at Rp 492,000,000 as follows,

$$164{,}000 \text{ m}^2 \times \text{Rp } 3{,}000/\text{m}^2 = \text{Rp } 492{,}000{,}000.$$

(6) Sea Dike Improvement Works.

These works were divided into two kinds, improvement of sluices and sea dike, as mentioned in Chapter VI, Part 2.

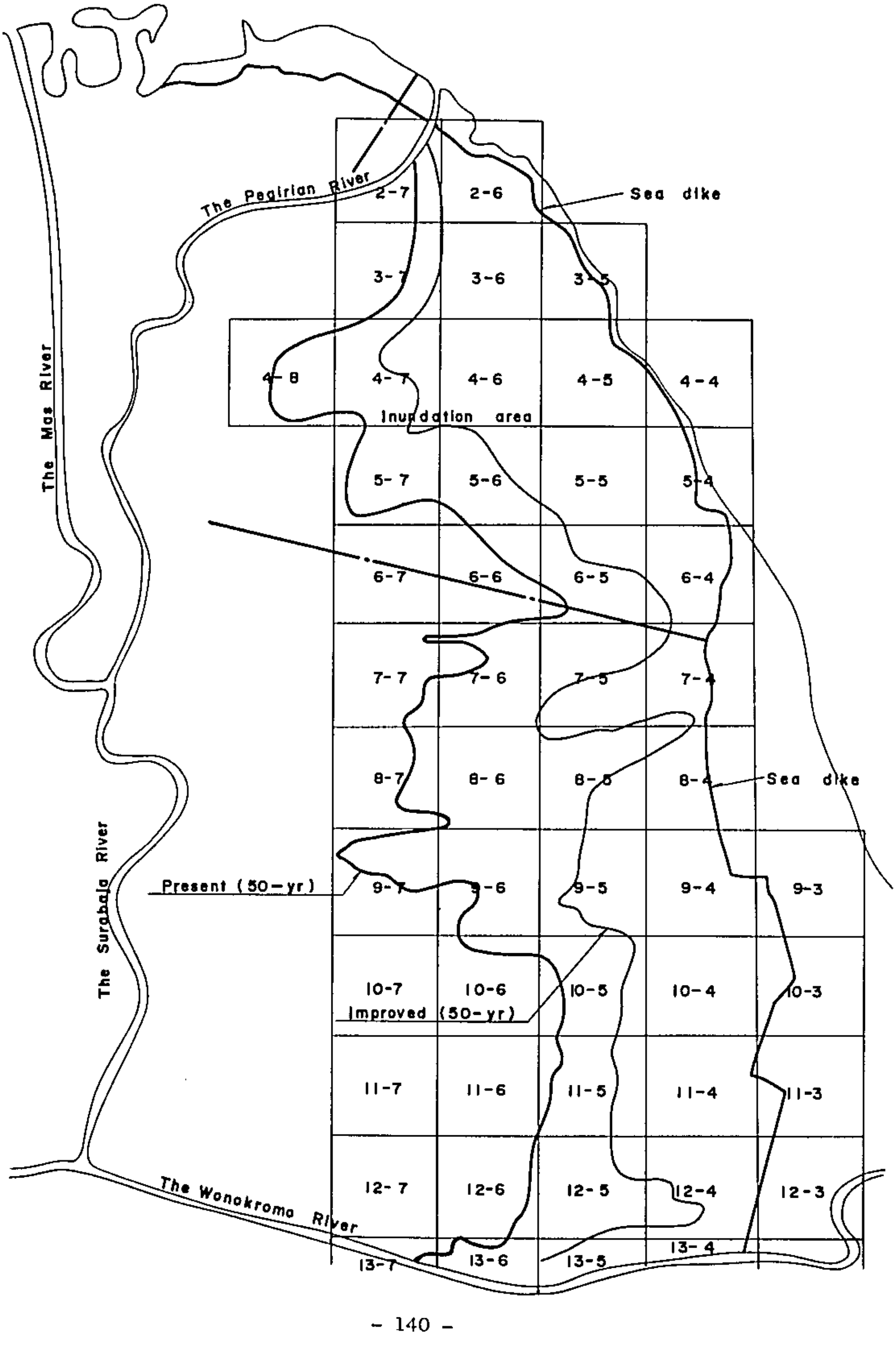
i. Economic benefits of the improvement works of the sluices on the sea dike.

The economic benefits of these improvement works were presented by the decrease of inundation damages caused by storm water in the land-side area of the sea dike. The amount of decrease of damages was obtained as difference of inundation damages between before and after the improvement works.

The method of possible flooding was also applied in order to estimate the amount of the inundation damages, and return-periods of 50, 10, 5, 2 and 1.05-year were adopted. The damage consists of the decrease of valuation of properties of houses and production decrease of paddy due to damaged by submergence.

As described in Chapter XVI, Part 4, the area protected by the sea dike was divided into two basins of north basin and south basin according to the catchment basins of drainage canals. Accordingly, the estimation of inundation damages were also made for each of two basins. Fig. 3 shows the area which may be inundated by 50-year storm before and after improvement works. The total area of inundation covers about 3,000 ha which comprize the north basin of 1,200 ha and the south basin of 1,800 ha. Water level in the

Fig. 3 Inundation Area in the Eastern part of Surabajo City



inundated area for each return period of storm were shown in Tables 8 and 9 of Chapter XVI, Part 4. According to these tables, the highest water level before improvement showed 0.406 m SHVP in the north basin and 0.490 m SHVP in the south basin, respectively. The following tables show the highest water level for each return period before and after improvement works.

The Highest Water Level in the North Basin

unit : m, SHVP

Return period (years)	50	10	5	2	1.05
Before improvement	0.406	0.368	0.338	0.274	0.160
After improvement	0.249	0.215	0.184	0.128	0.035

The Highest Water Level in the South Basin

unit : m, SHVP

Return period (years)	50	10	5	2	1.05
Before improvement	0.490	0.429	0.397	0.341	0.251
After improvement	0.285	0.248	0.218	0.149	0.035

Tables 7 and 8 show number of houses and area of paddy field in each mesh in the inundated area together with ground height.

Table 7 Ground Height and Properties in the Inundated North Basin

No. of mesh	Ground height (m)	Number of houses						Area of paddy field (ha)
		Office	Residence	Farmhouse	Shop	Factory	School	
2-6	0.09	0	0	129	0	0	0	40
2-7	0.32	0	0	39	0	0	0	81
3-5	0.04	0	0	0	0	0	0	29
3-6	0.14	0	0	45	0	0	0	90
3-7	0.37	0	0	15	0	0	0	93
4-4	0.01	0	0	10	0	0	0	0
4-5	0.07	0	0	179	0	0	0	65
4-6	0.15	1	0	370	0	0	1	87
4-7	0.26	1	0	487	0	0	1	79
4-8	0.48	3	2169	565	85	2	3	40
5-4	0.04	0	0	30	0	2	1	3
5-5	0.15	0	0	3	0	1	0	96
5-6	0.27	0	0	135	0	0	0	94
5-7	0.38	2	901	406	41	0	2	63
6-4	0.10	0	0	25	0	4	0	53
6-5	0.28	0	0	32	0	1	0	71
6-6	0.45	1	386	211	32	2	1	44
7-4	0.11	0	0	0	0	1	0	0
Total		8	3,456	2,681	158	13	9	1,028

Table 8 Ground Height and Properties in the Inundated South Basin

No. of mesh	Ground height (m)	Number of houses						Area of paddy field (ha)
		Office	Residence	Farmhouse	Shop	Factory	School	
6-5	0.45	0	0	5	0	0	0	18
6-6	0.55	1	316	291	24	0	0	30
7-4	0.10	0	0	40	0	0	0	30
7-5	0.30	0	0	340	0	0	0	83
7-6	0.44	1	208	215	11	0	0	85
8-4	0.10	0	0	7	0	0	0	42
8-5	0.30	0	0	111	0	0	0	88
8-6	0.40	1	110	433	8	0	0	56
8-7	0.60	2	2218	262	131	1	1	37
9-3	0.01	0	0	64	0	0	0	0
9-4	0.09	0	0	57	0	0	0	75
9-5	0.27	0	0	0	0	0	0	99
9-6	0.42	0	0	65	0	0	0	93
9-7	0.51	2	1165	136	61	0	0	68
10-3	0.02	0	0	446	0	0	0	6
10-4	0.15	0	0	0	0	0	0	100
10-5	0.35	0	0	170	0	0	0	82
10-6	0.54	0	0	105	0	0	0	88
11-3	0.01	0	0	65	0	0	0	6
11-4	0.13	0	0	363	0	0	0	94
11-5	0.35	0	0	30	0	0	0	96
12-4	0.14	0	0	40	0	0	0	97
12-5	0.35	0	0	80	0	0	0	96
12-6	0.57	0	0	239	0	0	0	76
13-4	0.06	0	0	36	0	0	0	20
13-5	0.20	0	0	5	0	0	0	34
13-6	0.33	0	0	5	0	0	0	34
Total		7	4,017	3,610	235	1	1	1,633

In addition to matters mentioned above, the following conditions and assumptions were put for the estimation of inundation damages similarly to the previous section.

a. For the valuation of building and household effects, the same values as in the cases of the Marmojo and the Surabaja/Wonokromo river improvement works were adopted.

b. Unit yield and unit price of dry paddy were assumed as follows,

(a) Unit yield : 3,790 kg/ha,

(b) Unit price : Rp 19/kg,

where unit yield means an average yield per ha of dry paddy in Sukolilo for the past five years from 1965 to 1969 and Rp 19/kg of dry paddy is nearly equivalent to Rp 15/kg of dry stalked paddy.

c. The following three items were assumed similarly to the Marmojo river improvement works.

(a) Usual water level of paddy field was assumed at 10 cm above ground surface.

(b) Floor level in a house was assumed at 10 cm above ground surface.

(c) Submergence depth harmless to planted paddy was assumed under 30% of its height.

Although the durations of inundation are shown in Figs. 9 through 12 of the said Chapter XVI, net durations of submergence by which the planted paddy is damaged were found out from Tables 8 and 9 of Chapter XVI using an electronic computer under the conditions of (a) and (c) of c mentioned above.

The inundation damages to planted paddy were calculated for each of the four months; December, January, February and March, for the purpose of finding the largest damage throughout a year similarly to the case of the estimation of damages of the Marmojo river improvement works. In this case, area and height of planted paddy for each of the above months are as follows.

Month	Dec.	Jan.	Feb.	Mar.
Ratio of area of planted paddy to cultivated area	0.188	0.622	0.705	0.993
Plant height (m)	0.37	0.58	0.98	1.14

note : The above figures were quoted from those of rainy season paddy of Djeblokan W-8 shown in Table 2-21 of Chapter XXII, Part 4.

The amount of the average annual storm damages for each of the said four months was calculated according to the way described in Chapter XXIV, Part 4. The results are shown in Table 9.

Table 9 Amount of the Average Annual Storm Damage

unit : Rp 1,000

Month	Dec.	Jan.	Feb.	Mar.
North basin				
Before improvement	2,152	2,222	1,802	1,912
After improvement	14	9	5	5
Difference	2,138	2,213	1,797	1,907
South basin				
Before improvement	14,152	13,964	13,086	13,771
After improvement	439	430	419	419
Difference	13,713	13,534	12,667	13,352

It is seen from Table 9 that the difference of damages between before and after improvement is largest in January in the North basin, in December in the South basin. The sum of the largest values of decrease of inundation damages in both North and South basins was counted at Rp 15,926,000 and this was adopted as the economic benefit from the improvement works of sluices on the sea dike.

ii. Economic benefits from the improvement works of the sea dike.

The economic benefits of the improvement works of the sea dike is to be obtained as the difference of damages which might because by inundation of sea water between before and after the improvement of the sea dike. However, sea water will not intrude into landside by overtopping of waves after the improvement. Accordingly, the flood damage was studied only as to before improvement. It directly corresponds to economic benefit of the present works.

In Chapter XV, Part 4, sea forces of several return-periods were used to examine the condition that sea water intrudes into landside by overtopping of waves. After analysis, it was found that the existing sea dike might be overtopped by the sea forces larger than that of 7-year return period. Consequently, return-periods from 50 to 7 were adopted for the estimation of the inundation damage.

The inundated area due to 50-year return-period was estimated at about 1,200 ha which comprize a residential area of 200 ha and a paddy field of 1,000 ha. It is nearly equal to the area inundated by the storm in the north basin in Fig. 3 shown previously. It was taken that the water level which would occur in landside if a low-crest part of the dike should be broken by continued overtopping of waves is 0.44m SHVP which is equal to the highest tide level in the past.

Conditions and assumptions which are required for the estimation of amount of damage are quite the same as those of the north basin mentioned above, excepting a condition that the planted paddy completely die by submergence in sea water unrelated to the depth of submergence.

Consequently, the amount of the average annual flood damage for each of the four months was counted as shown in Table 10.

Table 10 Amount of the Average Annual Flood Damage due to Sea Water

unit : Rp 1,000

Month	Dec.	Jan.	Feb.	Mar.
Amount of damage	3,978	8,362	9,201	12,110

As obvious from Table 10, the amount of flood damage due to the intrusion of sea water takes the largest value in March.

After all, the whole economic benefits for the sea dike improvement works which includes that of sluices were estimated at Rp 28,036,000 as the sum of two benefits of the present and the preceeding paragraphs.

2. Benefit-Cost Analysis

Average annual benefits which may be brought by the works, as described in the preceeding section, are summarized in Table 11. It is assumed that the average annual benefits appear from the year following the completion of the works.

Table 11 Average Annual Benefits

unit : Rp 1,000

Kind of improvement works	5th Year	6th Year	7th Year	.....	nth Year /1
Marmojo river	0	0	12,267	.....	12,267
Surabaja/Wonokromo river	0	0	271,240 /2	.....	271,240 /2
Mas river	0	0	306,446	.....	306,446
Morokrembangan Boezem	0	492,000	0	.....	0
Sea dike	28,036	28,036	28,036	.....	28,036
Total	28,036	520,036	617,989 /2		617,989 /2

note : /1 : n is taken as 50, life of project.

/2 : The average annual benefits which may be brought by rehabilitating the function of Gunungsari Dam should be added to this value. Those values are shown in Table 2 of Chapter XXV, Part 4.

On the other hand, annual construction costs including those of equipment, materials, and engineering services are expressed in Rupiah, as shown in Table 12 using Table 4 of Chapter VIII.

Table 12 Annual Construction Costs

unit : Rp 1,000

	1st Year	2nd Year	3rd Year	4th Year	5th Year	6th Year	Total
Construction costs	132,136	946,970	557,760	623,110	730,018	483,047	3,473,041

Total construction cost shown in Table 12 includes Rp 945,836,000 (¥701,970,000 equivalent) of equipment and materials costs shown in Table 2 of Chapter VIII. Out of them, values of some equipment may be left as residual after the completion of the works as shown in Table 13.

Table 13 Annual Residual Values of the Equipment

unit : Rp 1,000

	5th Year	6th Year	Total
Residual values	4,042	18,787	22,829

In this Table, the amount of Rp 4,042,000 at the fifth year is the residual value of booster for dredger and others after the completion of the improvement works of Morokrembangan Boezem, and residual values of other equipment, except the above, are given in the sixth year in which the whole works of the project are completed.

The annual operation and maintenance costs required for operating and repairing the new facilities may appear from the year following the completion of the works, similarly to the case of the average annual benefit mentioned above. On the contrary, the annual operation and maintenance costs for the old facilities shall be treated as a benefit from the year following the completion of the works.

Table 14 shows the average annual operation and maintenance costs before and after the completion of the works. The former were given by the latest costs which have been spent for operating and repairing to the existing facilities, while the latter were estimated on the basis of the above lastest costs taking into consideration the modernization of facilities.

Table 14 Average Annual Operation and Maintenance Costs

unit : Rp 1,000

Items	4th Year	5th Year	6th Year	7th Year	nth/1 Year
Before the completion of works	500/2	500/2	500/2	2,400/5.........	2,400/5
After the completion of works	130/2	484/3	672/4	1,974/5........	1,974/5

note : /1 : n is taken as 50, life of project.

/2 : for improvement works of Djagir dam.

/3 : for Djagir dam and sea dike.

/4 : for Djagir dam, sea dike and Morokrembangan Boezem.

/5 : for all the works of the present project.

By summarizing the benefits and costs shown in Tables 11 through 14 annually, for the period of Project life, the costs to be invested for the construction works and the benefits to be brought by these works are given in Table 15.

Table 15 Costs to be Invested and Benefits to be Brought during the Period of Project Life

unit : Rp 1,000

Year	Construction cost	Operation & maintenance costs.	Benefit	Residual value
1	132,136	0	0	0
2	946,970	0	0	0
3	557,760	0	0	0
4	623,110	130	500	0
5	730,018	484	28,536	4,042
6	483,047	672	520,536	18,787
7	0	1,942	620,389	0
.	.	.	.	.
.	.	.	.	.
.	.	.	.	.
50	0	1,942	620,389	0

note 1 : Benefits include the operation and maintenance costs for the existing facilities after the completion of works.

note 2 : Irrigation benefits to be brought from the seventh year by reconstruction of Gunungsari dam are not yet added to the benefits in the above Table.

On the basis of the values in Table15, and Table 2 of Chapter XXV, Part 4, the economic feasibility of the present project has been examined by comparing the present values of the benefits with the present values of the costs. The analysis has been conducted by two methods, as well known, benefit-cost ratio and internal rate of return methods. Some results according to the former method are shown in Tables 17 and 18. The calculations have been made for the period of 50 years of project life assuming two cases of discount rates of 3% and 6%.

The calculations according to the method of internal rate of return also have been made for the period of 50 years, and present values of benefits and costs are shown in Tables 19 through 22. Internal rates of return have been estimated from present values of benefits and costs shown in the Tables, assuming economic lives of 20 and 50 years. The results, together with the benefit-cost ratio, are shown in Table 16.

Table 16 Results of Benefit-Cost Analysis

Project life	20-year		50-year	
Discount rate	3%	6%	3%	6%
Benefit-cost ratio	2.28	1.78	4.58	2.80
Internal rate of return	13.9%		15.8%	

These results indicate that the present project is sufficiently feasible from the economic point of view. However, the benefits considered in this analysis were only principal direct and tangible ones. Therefore, if indirect and/or intangible benefits other that the above are taken into consideration, it is obvious that the present project will be the more feasible.

Table 17 Present Value of Benefits and Costs

Discount rate : 3%　　　　unit : Rp 1,000

Year	Discounted Costs		Present Value of Cost	Discounted Benefits		Present Value of Benefit	Net Present Value	Benefit-Cost Ratio
	Construction Cost	Ope. & Maint. Costs		Benefit	Residual Value			
1	128287.	0.	128287.	0.	0.	0.	-128287.	0.0
2	892610.	0.	1020897.	0.	0.	0.	-1020897.	0.0
3	510429.	0.	1531327.	0.	0.	0.	-1531327.	0.0
4	553625.	116.	2085068.	444.	0.	444.	-2084623.	0.000213
5	629720.	418.	2715205.	24615.	3487.	28546.	-2686659.	0.010514
6	404544.	563.	3120312.	435941.	15734.	480221.	-2640091.	0.153902
7	0.	1579.	3121891.	575195.	0.	1055416.	-2066475.	0.338069
8	0.	1533.	3123424.	558284.	0.	1613699.	-1509725.	0.516644
9	0.	1488.	3124912.	541870.	0.	2155569.	-969344.	0.689801
10	0.	1445.	3126357.	525907.	0.	2681476.	-444882.	0.857700
11	0.	1403.	3127760.	510413.	0.	3191889.	64128.	1.020503
12	0.	1362.	3129122.	495375.	0.	3687264.	558142.	1.178370
13	0.	1322.	3130445.	480805.	0.	4168070.	1037625.	1.331462
14	0.	1284.	3131729.	466544.	0.	4634613.	1502885.	1.479890
15	0.	1246.	3132975.	452742.	0.	5087355.	1954380.	1.623810
16	0.	1210.	3134185.	439335.	0.	5526690.	2392504.	1.763358
17	0.	1175.	3135360.	426311.	0.	5953001.	2817640.	1.898666
18	0.	1141.	3136501.	413646.	0.	6366647.	3230384.	2.029856
19	0.	1107.	3137609.	401346.	0.	6767993.	3630384.	2.157055
20	0.	1075.	3138684.	389398.	0.	7157392.	4018708.	2.280380
21	0.	1044.	3139728.	377784.	0.	7535175.	4395448.	2.399945
22	0.	1014.	3140741.	366491.	0.	7901667.	4760925.	2.515860
23	0.	984.	3141725.	355525.	0.	8257191.	5115466.	2.628235
24	0.	955.	3142681.	344854.	0.	8602045.	5459365.	2.737168
25	0.	928.	3143608.	334482.	0.	8936527.	5792919.	2.842761
26	0.	900.	3144509.	324400.	0.	9260928.	6116419.	2.945111
27	0.	874.	3145383.	314593.	0.	9575521.	6430138.	3.044310
28	0.	849.	3146232.	305063.	0.	9880584.	6734352.	3.140450
29	0.	824.	3147056.	295792.	0.	10176375.	7029320.	3.233618
30	0.	800.	3147856.	286775.	0.	10463151.	7315295.	3.323898
31	0.	777.	3148633.	277954.	0.	10741105.	7592472.	3.411355
32	0.	754.	3149387.	268400.	0.	11009505.	7860118.	3.495761
33	0.	732.	3150119.	259011.	0.	11268516.	8118397.	3.577171
34	0.	711.	3150830.	249821.	0.	11518337.	8367507.	3.655652
35	0.	690.	3151520.	240821.	0.	11759157.	8607637.	3.731265
36	0.	670.	3152190.	231746.	0.	11990904.	8838714.	3.803991
37	0.	651.	3152841.	223395.	0.	12214299.	9061458.	3.874062
38	0.	632.	3153472.	214938.	0.	12429237.	9275765.	3.941445
39	0.	613.	3154085.	206622.	0.	12635859.	9481774.	4.006188
40	0.	595.	3154681.	198479.	0.	12834338.	9679657.	4.068348
41	0.	578.	3155259.	190467.	0.	13024804.	9869546.	4.127967
42	0.	561.	3155820.	180458.	0.	13205263.	10049443.	4.184416
43	0.	545.	3156365.	174848.	0.	13380110.	10223746.	4.239089
44	0.	529.	3156894.	168976.	0.	13549087.	10392193.	4.291905
45	0.	514.	3157407.	164055.	0.	13713141.	10555734.	4.343165
46	0.	499.	3157906.	159277.	0.	13872418.	10714512.	4.392917
47	0.	484.	3158390.	154637.	0.	14027055.	10868666.	4.441205
48	0.	470.	3158860.	150133.	0.	14177189.	11018329.	4.488072
49	0.	456.	3159316.	145761.	0.	14322949.	11163633.	4.533560
50	0.	443.	3159759.	141515.	0.	14464464.	11304705.	4.577711

Table 18 Present Value of Benefits and Costs

Discount rate : 6% unit : Rp 1,000

Year	Discounted Costs: Construction Cost	Discounted Costs: Ope. & Maint. Costs	Present Value of Cost	Discounted Benefits: Benefit	Discounted Benefits: Residual Value	Present Value of Benefit	Net Present Value	Benefit-Cost Ratio
1	124657.	0.	124657.	0.	0.	0.	-124657.	0.0
2	842800.	0.	967457.	0.	0.	0.	-967457.	0.0
3	468306.	0.	1435763.	0.	0.	0.	-1435763.	0.0
4	493561.	103.	1929427.	396.	0.	396.	-1929031.	0.000205
5	545512.	362.	2475301.	21324.	3020.	24740.	-2450560.	0.009995
6	340529.	474.	2816303.	366957.	13244.	404942.	-2411362.	0.143785
7	0.	1292.	2817595.	470473.	0.	875414.	-1942181.	0.310696
8	0.	1218.	2818813.	443717.	0.	1319131.	-1499682.	0.467974
9	0.	1149.	2819963.	418482.	0.	1737613.	-1082350.	0.616183
10	0.	1084.	2821047.	394659.	0.	2132273.	-688774.	0.755844
11	0.	1023.	2822070.	372192.	0.	2504465.	-317606.	0.887456
12	0.	965.	2823035.	351003.	0.	2855468.	32432.	1.011488
13	0.	910.	2823946.	331037.	0.	3186505.	362559.	1.128387
14	0.	859.	2824805.	312127.	0.	3498632.	673827.	1.238539
15	0.	810.	2825615.	294321.	0.	3792953.	967337.	1.342346
16	0.	764.	2826380.	277522.	0.	4070474.	1244095.	1.440173
17	0.	721.	2827101.	261673.	0.	4332148.	1505047.	1.532364
18	0.	680.	2827781.	246714.	0.	4578862.	1751081.	1.619242
19	0.	642.	2828423.	232603.	0.	4811465.	1983041.	1.701112
20	0.	606.	2829029.	219291.	0.	5030756.	2201727.	1.778263
21	0.	571.	2829600.	206729.	0.	5237485.	2407885.	1.850963
22	0.	539.	2830139.	194874.	0.	5432359.	2602220.	1.919467
23	0.	508.	2830647.	183692.	0.	5616051.	2785404.	1.984017
24	0.	480.	2831127.	173136.	0.	5789187.	2958061.	2.044835
25	0.	452.	2831579.	163176.	0.	5952364.	3120784.	2.102136
26	0.	427.	2832006.	153779.	0.	6106142.	3274136.	2.156119
27	0.	403.	2832409.	144909.	0.	6251052.	3418643.	2.206974
28	0.	380.	2832789.	136542.	0.	6387594.	3554805.	2.254878
29	0.	358.	2833147.	128646.	0.	6516240	3683093.	2.300000
30	0.	338.	2833485.	121194.	0.	6637434.	3803949.	2.342498
31	0.	319.	2833804.	114142.	0.	6751576.	3917772.	2.382513
32	0.	301.	2834105.	107099.	0.	6858675.	4024570.	2.420050
33	0.	284.	2834389.	100427.	0.	6959103.	4124714.	2.455239
34	0.	268.	2834657.	94123.	0.	7053226.	4218569.	2.488211
35	0.	253.	2834910.	88164.	0.	7141390.	4306480.	2.519089
36	0.	238.	2835148.	82441.	0.	7223830.	4388682.	2.547955
37	0.	225.	2835373.	77221.	0.	7301051.	4465678.	2.574988
38	0.	212.	2835585.	72195.	0.	7373246.	4537661.	2.600256
39	0.	200.	2835785.	67437.	0.	7440683.	4604898.	2.623853
40	0.	189.	2835974.	62946.	0.	7503629.	4667655.	2.645874
41	0.	319.	2833804.	58696.	0.	7562325.	4726173.	2.666403
42	0.	301.	2834105.	54037.	0.	7616362.	4780042.	2.685297
43	0.	284.	2834389.	50875.	0.	7667237.	4830759.	2.703083
44	0.	268.	2834657.	47776.	0.	7715013.	4878385.	2.719783
45	0.	253.	2834910.	45071.	0.	7760084.	4923315.	2.735536
46	0.	238.	2835148.	42520.	0.	7802604.	4965702.	2.750396
47	0.	126.	2837028.	40113.	0.	7842718.	5005690.	2.764413
48	0.	118.	2837146.	37843.	0.	7880560.	5043414.	2.777636
49	0.	112.	2837258.	35701.	0.	7916261.	5079003.	2.790110
50	0.	105.	2837364.	33680.	0.	7949941.	5112577.	2.801876

Table 19 Present Value of Benefits and Costs

Discount rate : 13% unit : Rp 1,000

Year	Discounted Costs		Present Value of Cost	Discounted Benefits		Present Value of Benefit	Net Present Value	Benefit-Cost Ratio
	Construction Cost	Ope. & Maint. Costs		Benefit	Residual Value			
1	116935.	0.	116935.	0.	0.	0.	-116935.	0.0
2	741616.	0.	858551.	0.	00.	0.	-858551.	0.0
3	386556.	0.	1245107.	0.	0.	0.	-1245107.	0.0
4	382165.	80.	1627351.	307.	0.	307.	-1627045.	0.000188
5	396225.	263.	2023839.	15488.	2194.	17989.	-2005850.	0.008888
6	232016.	323.	2256178.	250023.	9024.	277036.	-1979142.	0.122790
7	0.	825.	2257003.	300695.	0.	577731.	-1679273.	0.255972
8	0.	731.	2257734.	266027.	0.	843757.	-1413976.	0.373719
9	0.	646.	2258380.	235355.	0.	1079113.	-1179268.	0.477826
10	0.	572.	2258952.	208208.	0.	1287320.	-971632.	0.569875
11	0.	506.	2259459.	184191.	0.	1471511.	-787947.	0.651267
12	0.	448.	2259907.	162945.	0.	1634456.	-625451.	0.723240
13	0.	396.	2260303.	144156.	0.	1778612.	-481691.	0.786891
14	0.	351.	2260654.	127501.	0.	1906113.	-354540.	0.843169
15	0.	311.	2260964.	112780.	0.	2018894.	-242071.	0.892935
16	0.	275.	2261239.	99755.	0.	2118649.	-142590.	0.936941
17	0.	243.	2261482.	88232.	0.	2206881.	-54602.	0.975856
18	0.	215.	2261698.	78035.	0.	2284915.	23218.	1.010266
19	0.	190.	2261888.	69014.	0.	2353929.	92041.	1.040692
20	0.	169.	2262057.	61034.	0.	2414963.	152906.	1.067596
21	0.	149.	2262206.	53973.	0.	2468936.	206730.	1.091384
22	0.	132.	2262338.	47726.	0.	2516662.	254324.	1.112417
23	0.	117.	2262455.	42201.	0.	2558863.	296409.	1.131012
24	0.	103.	2262558.	37312.	0.	2596175.	333617.	1.147451
25	0.	91.	2262649.	32987.	0.	2629162.	366512.	1.161984
26	0.	81.	2262730.	29161.	0.	2658323.	395593.	1.174830
27	0.	72.	2262802.	25777.	0.	2684101.	421299.	1.186184
28	0.	63.	2262865.	22784.	0.	2706885.	444019.	1.196220
29	0.	56.	2262921.	20137.	0.	2727022.	464100.	1.205089
30	0.	50.	2262971.	17795.	0.	2744817.	481846.	1.212926
31	0.	44.	2263015.	15722.	0.	2760538.	497523.	1.219850
32	0.	39.	2263054.	13838.	0.	2774376.	511322.	1.225943
33	0.	34.	2263088.	12172.	0.	2786548.	523460.	1.231303
34	0.	30.	2263119.	10701.	0.	2797249.	534130.	1.236015
35	0.	27.	2263146.	9403.	0.	2806652.	543506.	1.240155
36	0.	24.	2263170.	8248.	0.	2814899.	551730.	1.243786
37	0.	21.	2263191.	7247.	0.	2822146.	558956.	1.246977
38	0.	19.	2263209.	6355.	0.	2828502.	565292.	1.249775
39	0.	17.	2263226.	5569.	0.	2834071.	570845.	1.252226
40	0.	15.	2263240.	4876.	0.	2838947.	575706.	1.254373
41	0.	13.	2263253.	4265.	0.	2843212.	579958.	1.256250
42	0.	11.	2263265.	3683.	0.	2846895.	583630.	1.257871
43	0.	10.	2263275.	3253.	0.	2850148.	586873.	1.259303
44	0.	9.	2263284.	2866.	0.	2853014.	589730.	1.260564
45	0.	8.	2263292.	2536.	0.	2855550.	592258.	1.261680
46	0.	7.	2263299.	2244.	0.	2857794.	594495.	1.262667
47	0.	6.	2263305.	1986.	0.	2859780.	596475.	1.263541
48	0.	6.	2263311.	1758.	0.	2861537.	598227.	1.264315
49	0.	5.	2263316.	1555.	0.	2863093.	599777.	1.264999
50	0.	4.	2263320.	1376.	0.	2864469.	601149.	1.265605

Table 20 Present Value of Benefits and Costs

Discount rate : 14%

unit : Rp 1,000

Year	Discounted Cost: Construction Cost	Discounted Cost: Ope. & Maint. Costs	Present Value of Cost	Discounted Benefits: Benefit	Discounted Benefits: Residual Value	Present Value of Benefit	Net Present Value	Benefit-Cost Ratio
1	115909.	0.	115909.	0.	0.	0.	-115909.	0.0
2	728663.	0.	844571.	0.	0.	0.	-844571.	0.0
3	376472.	0.	1221044.	0.	0.	0.	-1221044.	0.0
4	368931.	77.	1590052.	296.	0.	296.	-1589756.	0.000186
5	379148.	251.	1969452.	14821.	2099.	17216.	-1952235.	0.008742
6	220070.	306.	2189827.	237149.	8559.	262924.	-1926903.	0.120066
7		776.	2190603.	282710.	0.	545635.	-1644969.	0.249080
8		681.	2191284.	247921.	0.	793556.	-1397728.	0.362142
9		597.	2191881.	217413.	0.	1010969.	-1180912.	0.461234
10		524.	2192405.	190648.	0.	1201617.	-990788.	0.548082
11		460.	2192865.	167178.	0.	1368795.	-824070.	0.624204
12		403.	2193268.	146596.	0.	1515391.	-677877.	0.690928
13		354.	2193621.	128555.	0.	1643947.	-549675.	0.749421
14		310.	2193932.	112706.	0.	1756652.	-437279.	0.800687
15		272.	2194204.	98818.	0.	1855470.	-338733.	0.845624
16		239.	2194442.	86639.	0.	1942109.	-252333.	0.885013
17		209.	2194652.	75959.	0.	2018068.	-176584.	0.919539
18		184.	2194835.	66590.	0.	2084658.	-110177.	0.949802
19		161.	2194996.	58376.	0.	2143034.	-51962.	0.976327
20		141.	2195138.	51173.	0.	2194207.	-930.	0.999576
21		124.	2195262.	44856.	0.	2239064.	43802.	1.019953
22		109.	2195370.	39317.	0.	2278380.	83010.	1.037811
23		95.	2195466.	34460.	0.	2312840.	117374.	1.053462
24		84.	2195549.	30200.	0.	2343040.	147491.	1.067177
25		73.	2195623.	26466.	0.	2369506.	173883.	1.079195
26		64.	2195687.	23191.	0.	2392697.	197010.	1.089726
27		56.	2195744.	20320.	0.	2413017.	217274.	1.098952
28		50.	2195793.	17803.	0.	2430820.	235027.	1.107035
29		43.	2195837.	15596.	0.	2446417.	250580.	1.114116
30		38.	2195875.	13662.	0.	2460079.	264204.	1.120318
31		33.	2195908.	11964.	0.	2472043.	276135.	1.125750
32		29.	2195938.	10438.	0.	2482481.	286543.	1.130488
33		26.	2195963.	9101.	0.	2491582.	295618.	1.134619
34		23.	2195986.	7931.	0.	2499513.	303527.	1.138219
35		20.	2196006.	6908.	0.	2506420.	310415.	1.141354
36		17.	2196023.	6006.	0.	2512426.	316403.	1.144080
37		15.	2196038.	5231.	0.	2517657.	321619.	1.146454
38		13.	2196052.	4547.	0.	2522204.	326153.	1.148518
39		12.	2196063.	3949.	0.	2526154.	330090.	1.150310
40		10.	2196074.	3428.	0.	2529581.	333508.	1.151866
41		9.	2196083.	2972.	0.	2532553.	336471.	1.153214
42		8.	2196091.	2544.	0.	2535098.	339007.	1.154368
43		7.	2196097.	2227.	0.	2537325.	341227.	1.155379
44		6.	2196104.	2945.	0.	2539269.	343166.	1.156261
45		5.	2196109.	1706.	0.	2540975.	344866.	1.157035
46		5.	2196114.	1496.	0.	2542472.	346358.	1.157714
47		4.	2196118.	1313.	0.	2543784.	347667.	1.158310
48		4.	2196121.	1151.	0.	2544936.	348814.	1.158832
49		3.	2196124.	1010.	0.	2545946.	349821.	1.159290
50		3.	2196127.	886.	0.	2546832.	350704.	1.159692

Table 21 Present Value of Benefits and Costs

Discount rate : 15%

unit : Rp 1,000

Year	Discounted Costs		Present Value of Cost	Discounted Benefits		Present Value of Benefit	Net Present Value	Benefit-Cost Ratio
	Construction Cost	Ope. & Maint. Costs		Benefit	Residual Value			
1	114901.	0.	114901.	0.	0.	0.	-114901.	0.0
2	716045.	0.	830946.	0.	0.	0.	-830946.	0.0
3	366736.	0.	1197682.	0.	0.	0.	-1197682.	0.0
4	356265.	74.	1554022.	286.	0.	286.	-1553736.	0.000184
5	362948.	241.	1917211.	14187.	2010.	16483.	-1900728.	0.008597
6	208835.	291.	2126336.	225042.	8122.	249647.	-1876689.	0.117407
7	0.	730.	2127066.	265944.	0.	515591.	-1611474.	0.242396
8	0.	635.	2127701.	231190.	0.	746782.	-1380919.	0.350981
9	0.	552.	2128253.	200978.	0.	947760.	-1180492.	0.445323
10	0.	480.	2128733.	174704.	0.	1122464.	-1006268.	0.527292
11	0.	417.	2129150.	151864.	0.	1274328.	-854822.	0.598515
12	0.	363.	2129513.	132010.	0.	1406338.	-721375.	0.660404
13	0.	316.	2129829.	114758.	0.	1521096;	-608733.	0.714187
14	0.	274.	2130103.	99734.	0.	1620830.	-509273.	0.760916
15	0.	239.	2130342.	86684.	0.	1707514.	-422827.	0.801521
16	0.	208.	2130549.	75340.	0.	1782854.	-347695.	0.836805
17	0.	180.	2130730.	65478.	0.	1848333.	-282397.	0.867465
18	0.	157.	2130887.	56903.	0.	1905236.	-225651.	0.894105
19	0.	136.	2131023.	49450.	0.	1954686.	-176337.	0.917252
20	0.	119.	2131142.	42972.	0.	1997658.	-133484.	0.937365
21	0.	103.	2131245.	37340.	0.	2034997.	-96248.	0.954840
22	0.	90.	2131335.	32444.	0.	2067441.	-63894.	0.970022
23	0.	78.	2131413.	28189.	0.	2095630.	-35783.	0.983212
24	0.	68.	2131481.	24490.	0.	2120119.	-11361.	0.994670
25	0.	59.	2131540.	21274.	0.	2141394.	9854.	1.004623
26	0.	51.	2131591.	18480.	0.	2159874.	28283.	1.013268
27	0.	45.	2131635.	16051.	0.	2175925.	44290.	1.020777
28	0.	39.	2131674.	13941.	0.	2189866.	58192.	1.027299
29	0.	34.	2131708.	12107.	0.	2201973.	70265.	1.032962
30	0.	29.	2131737.	10513.	0.	2212486.	80749.	1.037879
31	0.	26.	2131763.	9126.	0.	2221612.	89849.	1.042148
32	0.	22.	2131785.	7893.	0.	2229505.	97720.	1.045840
33	0.	19.	2131804.	6822.	0.	2236327.	104523.	1.049030
34	0.	17.	2131821.	5893.	0.	2242221.	110400.	1.051787
35	0.	15.	2131836.	5088.	0.	2247309.	115473.	1.054166
36	0.	13.	2131848.	4386.	0.	2251695.	119846.	1.056217
37	0.	11.	2131859.	3786.	0.	2255481.	123622.	1.057988
38	0.	10.	2131869.	3263.	0.	2258744.	126875.	1.059514
39	0.	8.	2131877.	2809.	0.	2261553.	129676.	1.060827
40	0.	7.	2131885.	2417.	0.	2263970.	132086.	1.061957
41	0.	6.	2131891.	2077.	0.	2266048.	134157.	1.062929
42	0.	5.	2131896.	1763.	0.	2267811.	135915.	1.063753
43	0.	5.	2131901.	1530.	0.	2269341.	137440.	1.064468
44	0.	4.	2131905.	1324.	0.	2270665.	138760.	1.065087
45	0.	4.	2131909.	1151.	0.	2271816.	139908.	1.065625
46	0.	3.	2131912.	1001.	0.	2272818.	140906.	1.066094
47	0.	3.	2131915.	871.	0.	2273688.	141774.	1.066501
48	0.	2.	2131917.	757.	0.	2274446.	142528.	1.066855
49	0.	2.	2131919.	658.	0.	2275104.	143185.	1.067162
50	0.	2.	2131921.	572.	0.	2275676.	143755.	1.067430

Table 22 Present Value of Benefits and Costs

Discount ratio : 16% unit : Rp 1,000

Year	Discounted Costs		Present Value of Cost	Discounted Benefits		Present Value of Benefit	Net Present Value	Benefit-Cost Ratio
	Construction Cost	Ope. & Maint. Costs		Benefit	Residual Value			
1	113910.	0.	113910.	0.	0.	0.	-113910.	0.0
2	703753.	0.	817663.	0.	0.	0.	-817663.	0.0
3	357333.	0.	1174997.	0.	0.	0.	-1174997.	0.0
4	344138.	72.	1519206.	276.	0.	276.	-1518930.	0.000182
5	347571.	230.	1867008.	13586.	1924.	15787.	-1851221.	0.008456
6	198263.	276.	2065547.	213650.	7711.	237148.	-1851221.	0.114811
7	0.	687.	2066234.	250305.	0.	487453.	-1578781.	0.235914
8	0.	592.	2066826.	215719.	0.	703172.	-1363654.	0.430063
9	0.	511.	2067337.	185912.	0.	889084.	-1178252.	0.430063
10	0.	440.	2067777.	160214.	0.	1049299.	-1018478.	0.507453
11	0.	379.	2068157.	138068.	0.	1187367.	-880790.	0.574118
12	0.	327.	2068484.	118983.	0.	1306350.	-762134.	0.631550
13	0.	282.	2068766.	102541.	0.	1408891.	-659874.	0.681030
14	0.	243.	2069009.	88349.	0.	1497240.	-571769.	0.723651
15	0.	210.	2069218.	76127.	0.	1573367.	-495851.	0.760368
16	0.	181.	2069399.	65594.	0.	1638961.	-430438.	0.791999
17	0.	156.	2069555.	56516.	0.	1695477.	-374077.	0.819247
18	0.	134.	2069689.	48692.	0.	1744169.	-325520.	0.842720
19	0.	116.	2069805.	41949.	0.	1786119.	-283686.	0.862941
20	0.	100	2069905.	36139.	0.	1822258.	-247647.	0.880358
21	0.	86.	2069991.	31132.	0.	1853390.	-216601.	0.895361
22	0.	74.	2070065.	26817.	0.	1880207.	-189858.	0.908284
23	0.	64.	2070129.	23099.	0.	1903305.	-166823.	0.919414
24	0.	55.	2070184.	19895.	0.	1923200.	-146984.	0.929000
25	0.	48.	2070231.	17134.	0.	1940334.	-129898.	0.937255
26	0.	41.	2070272.	14755.	0.	1955089.	-115183.	0.944363
27	0.	35.	2070308.	12705.	0.	1967794.	-102513.	0.950484
28	0.	30.	2070338.	10940.	0.	1978734.	-91604.	0.955754
29	0.	26.	2070364.	9419.	0.	1988153.	-82212.	0.960291
30	0.	23.	2070387.	8108.	0.	1996261.	-74126.	0.964197
31	0.	20.	2070407.	6978.	0.	2003239.	-67168.	0.967558
32	0.	17.	2070423.	5983.	0.	2009222.	-61202.	0.970440
33	0.	14.	2070438.	5127.	0.	2014348.	-56089.	0.972909
34	0.	12.	2070450.	4391.	0.	2018739.	-51711.	0.975024
35	0.	11.	2070461.	3758.	0.	2022497.	-47964.	0.976834
36	0.	9.	2070470.	3211.	0.	2025708.	-44762.	0.978381
37	0.	8.	2070478.	2749.	0.	2028457.	-42022.	0.979704
38	0.	7.	2070485.	2348.	0.	2030805.	-39680.	0.980835
39	0.	6.	2070491.	2004.	0.	2032809.	-37682.	0.981801
40	0.	5.	2070496.	1710.	0.	2034519.	-35977.	0.982624
41	0.	4.	2070501.	1457.	0.	2035976.	-34525.	0.983325
42	0.	4.	2070505.	1225.	0.	2037201.	-33304.	0.983915
43	0.	3.	2070508.	1054.	0.	2038255.	-32253.	0.984423
44	0.	3.	2070511.	905.	0.	2039160.	-31351.	0.984859
45	0.	2.	2070513.	780.	0.	2039940.	-30573.	0.985234
46	0.	2.	2070515.	672.	0.	2040612.	-29903.	0.985558
47	0.	2.	2070517.	580.	0.	2041192.	-29325.	0.985837
48	0.	2.	2070519.	500.	0.	2041692.	-28827.	0.986077
49	0.	1.	2070520.	431.	0.	2042122.	-28398.	0.986285
50	0.	1.	2070521.	371.	0.	2042494.	-28027.	0.986464

CHAPTER X

ORGANIZATION AND FACILITIES

1. Organization.

As mentioned before, the Surabaja river improvement project requires about one and a half years to prepare the detailed designs and about four years to carry out the construction. It comprises various works as aforementioned such as those on excavation, embankment and revetment of the river; those on river structures as dams and sluices and on dredging. In particular, it includes the construction of new Gunungsari dam which requires an execution of complicated works with a high technique, and thus in order to complete the whole works in only four years it is necessary to assign the engineers with sufficient experience in accordance with the organization chart shown in Fig. 1.

Also it is necessary to establish a head office in Surabaja city near Gunungsari or Djagir dam and to provide field superintendent offices according to each work as shown in Fig. 1 so that perfect supervision and control of execution of work can be performed by the superintendent in charge of that work.

In addition to a chief, several assistants need to be assigned to each section of Head Office shown in the organization chart. In particular, it is necessary to assign each two or three engineers respectively specialized in surveying, river planning and designing and structure designing to Design & Investigation Section and each two or three engineers respectively experienced in river works and building of structures to Operation Section.

Since Laboratory will conduct the test of concrete to be used for concrete works and soil test accompanying boring and provide the direction and quality control for manufacturing concrete piles to be used as a foundation of structure and concrete blocks for revetment, it needs to assign some engineers capable of dealing with the test of this kind. In addition, the Head Office should be equipped with sufficient staffs together with enough typewriting, tracing and printing facilities.

For each field superintendent office it is necessary, in addition to a chief engineer with deep experience in the works under his supervision, to assign some clerks who deals with the issuing and receiving of materials and some technical staffs possessing skill in surveying for the supervision and control of works.

In particular, as the office for Gunungsari Dam which will have to undertake the supervision and control of complicated works, it needs to assign excellent engineers experienced in the construction of such dam.

2. Facilities.

In preparing the detail designs and carrying out the construction for the project, it is necessary to equip with the facilities and equipment at least listed in Table 1. A part of the equipment shown in Table 1 is to be procured by using foreign currency but the remaining facilities and equipment need to be supplied from the local government for which the expenditures are not included in the construction costs.

Fig. 1 Organization of Surabaja River Improvement Project

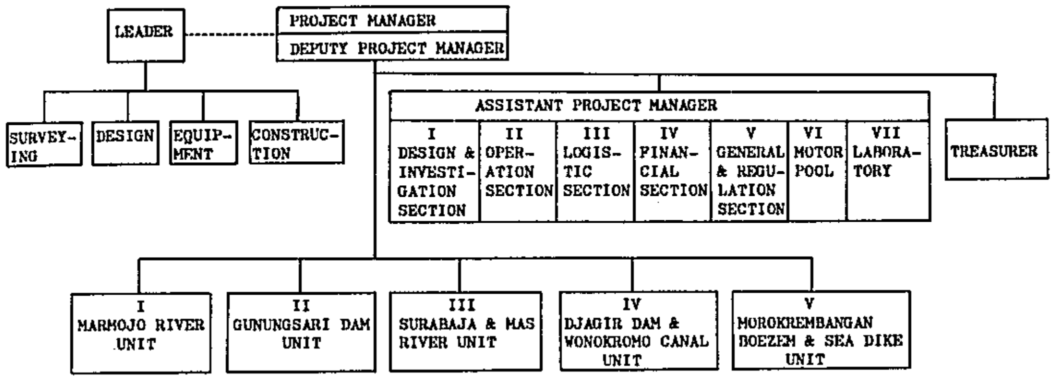


Table 1 List of Major Facilities for the Above-mentioned Organization

Facilities	Quantity	Remarks
(1) Surveying		
Full set of instruments for angular surveying	10 suits	o
Full set of instruments for Leveling	10 suits	o
Full set of instruments for plane table surveying	10 suits	o
Others for surveying	suit	o
(2) Geological surveying		
Boring machine set	1 suit	o
Soil auger set	2 suits	o
Swedish sounding set	2 suits	o
Soil test set	suit	o
Spare parts and others	suit	o
(3) Stationeries		
Calculation machines	12	o
Portable electronic computers	6	o
Copying machines, Large	2	o
Copying machines, Middle	4	o
Copying machines, Small	5	o
Portable typewriters	some	o
Typewriters	some	
Telephone	some	
Transceivers	10	o
Spare parts and others	suit	o
(4) Concrete		
Compression testing machine	1	o
Mixer	1	o
Apparatus for making specimen	suit	o
Slump testing apparatus	3	o
Air-meter	3	o
Apparatus for testing of aggregate	suit	o
Spare parts and others	suit	o
(5) General facilities		
Land cruisers	4	o
Jeeps	3	o
Jeeps	some	
Motorcycles	7	o
Generators, Large	2	o
Generators, Small	3	o
Air-conditioners, Large	2	o
Air-conditioners, small	3	o
Spare parts and others	suit	o
(6) Buildings and Lands		
Head office	1	
Field offices	5	
Laboratory	1	
Motor pool	1	
Storehouses	6	
Accommodations	2	
Water service	suit	
Electric service	suit	
Others	some	

note : Facilities of mark o are to be procured by using foreign currency.

PART 3

WATER REQUIREMENT

CHAPTER I

WATER REQUIREMENT IN URBAN AREA

1. General Consideration.

In establishing a sound planning of the Surabaja River Improvement Programme, it is of a great importance to grasp precise future water demands of municipal, industrial, and irrigation water uses. The future water demands in the city area have been predicted taking the following facts and assumptions into consideration:

a. Future population in 1992 will be 3.98 million,

b. city planning area will ultimately be expanded as wide as 29,200 ha, while present city area is 5,300 ha,

c. industrial water consumption may increase in proportion to an expansion of industrial area expected to be nearly 2,300 ha by 1992,

d. future industrial water consumption rate to production unit or unit ground area of factory will be reduced to 80% of present level by recirculating certain part of the water,

e. domestic water consumption will increase keeping a pace with an improvement of living standards, and

f. sewage discharged from tributaries of the Pegirian, Mas, and Surabaja rivers in 1992 will approximately be 71,900 cu m per day or 0.83 cu m per second.

2. Municipal Water Requirement.

(1) Existing water supply.

Water supply system of Surabaja city has been constructed, expanded and operated by Perusahaan Air Minum Kota' Madya Surabaja (Water Supply Department, Surabaja City) since 1920s. The sources of water rely on river and springs in the suburbs of the city. At present, the system is distributing the water to the city, serving the total house connections of 48,199 and supplying the water of 3,787,021 cu m per month (as of September, 1971) as given in Part 4.

The present water sources of the system are as follows:

a. Spring Kasri (or Tamanan)
located at 43.6 km south of Surabaja, an average water intake of 25 l per second.

b. Spring Tojoarang
an average water intake of 66 l per second or 3,960 l per min.

c. Spring Pelintahan
an average water intake of 111 litre per second, or 6.66 cu m per min.

d. Spring Umbulan
74 .4 km south of Surabaja, an average water intake of 300 litre per second or 18 cu m per min.

e. Spring Modjosari
60 km south to Surabaja, an average water intake of 600 litre per second or 36 cu m per min.

f. The Brantas River Intakes
at Djagir dam, an average water intake of 1,350 litre per second or 81 cu m per min.

Main pipes of distribution system are made of cast iron or ductile, ranging from 100 mm to as large as 900 mm in diameter. Water pressures in the existing main pipes are said to be 2 kg/sq cm in southern and middle part, and 1 kg/sq cm in northern part of the city. The distribution pipes are also provided connecting to the main pipes, usually having enough water pressures for domestic use. Quantity of the water consumed by each individual household is measured by water meter. In the central part of the city, the water supply system is well provided and inhabitants in the area tend to use the supply water in lieu of well water. This is further encouraged by the reason that the well water usually containes salt.

The water of the river contains substantial amount of inorganic and organic matters. For example, chemical composition of the river water at Gunungsari indicates, high turbidity of 300 to 15,000 ppm and total hardness as $CaCo_3$ of 100 to 150 ppm, as given in Part 4. At present, the river water is supplied to the system after being purified in water-purification works, the Ngagel I and Ngagel II stations, where the chemical precipitation is being used. All the clear water is first sent to water reservoir in Wonokitri, with a capacity of 21,000 cu m at the height of 22 m above the sea water level, and from there the water is delivered to the distribution system.

(2) Surveys and investigations.

The municipal water of the whole city consists of household use, public uses (public buildings, fountains, fire fighting, and cleaning of public places etc.), the possible loss in the water system, and commercial and small manufacturing uses in the city. In order to grasp the future water demand of these uses, necessary data and information such as quantities of water delivered monthly, number of the families served, chemicals used for water purification, and electricity spent for maintenance and operation of the system etc., have been collected as given in Part 4.

Since the data of only four years are available, it may not be sufficient to predict the future water consumption rate precisely for the year of 1990 on the basis of such a short-term record. Furthermore, from the data per capita water consumption rate, which is the most important unit to be established for the design, was not obtainable and it was necessary to count the number of persons served by the system at some selected area.

(3) Studies on selected areas.

In order to reach the better solution, studies have been conducted on the selected areas namely; Ngaglik, Tambaksari and Diponegoro, that are considered to have represented such distinctive standards of living as high maximum per capita water consumption rates are summarized as follows:

a. low middle class area.
   average 130 l/person/day
   maximum 220 "

b. middle class area.
   average 170-190 l/person/day
   maximum 260 "

c. high class area
   average 350 l/person/day
   maximum 600 "

It should be pointed out however, that in some low income bracket area locating in sprawling suburbs it is common way to buy the city water from relatively wealthy families. This makes it difficult to grasp the actual number of persons served by the system as a whole and it was not possible to obtain the entire picture of this matter. By this reason the figures obtained in the studies may give undue values. It is therefore, necessary to conduct another study to predict the reasonable consumption rates by refering data and records of other cities and by assuming a future standards of living in Surabaja city as discussed in the following clause.

(4) Assumption of household use.

As the household water use is composed of water spent for various purposes such as drinking, cooking, washing, sprinkling, bathing and toilet flushing etc., the total water consumption may vary widely in response to the living mode and standard. It has been considered, however, that there may be an upper limit to the future consumption rate unless the mode of living has completely changed from that of present.

A typical break-down of the ultimate household water consumption rate, which has been widely used for sanitary sewer designs in Japan, and future consumption are given in Table 1.

Table 1 Composition of Household Water (litre per capita daily)

Water use	Ultimate condition	Ratio (%)	Consumption rates in Surabaja, in 1990
Drinking	2	100	2
Cooking	12	80	10
Kitchen (dish washing)	14	70	10
Bathing	59	70	41
Laundry	60	70	42
Cleaning	8	70	6
Disposer	6	0	0
Toilet flushing	30	20	6
Car washing	85	10	9
Air conditioning	143	10	14
Miscellaneous	12	20	2
Total	431		140

The water use for air conditioning is liable to change by its type and/or climate. Recently, air-cooled window type air conditioner is most commonly used in households for which no water is usually required, therefore, this amount of water can be reduced. Other items such as laundry, disposer, toilet flushing and car washing are also to be reduced at some extent, thus finally leading the total household water consumption to approximately 140 litre per capita daily.

For better understanding, other data, as given in Table 2, observed in apartment houses around the Tokyo Metropolitan Area, Japan, have also been discussed. The water spent in the apartment complexes is considered to have consisted mostly of household water and usually the proportion of commercial or industrial water is low or negligible.

In the complexes, floor area of each house ranges between 50 to 100 sq m and is provided with necessary accomodations. Public facilities are also well provided to these communities and living condition of families in the complexes is considered to represent a middle class citizens that is a majority group in Japan. Therefore, these records may be useful for predicting the future water consumption rate in Surabaja city.

Table 2 Household Water Use of Apartment Houses in the Tokyo Metropolitan Area

Name of block	Month & year	Population	Water consumption cu m month	Water consumption per cap daily	Remarks
Motosumiyoshi	4-5, 1964	189	1,091	192.5 l.	
"	6-7 "	"	1,153	203.3 "	
"	4-5, 1965	187	1,063	189.0 "	
"	6-7 "	"	1,121	199.9 "	
Higashikurume	5 "	8,697	40,925	151.9 "	
"	6 "	"	44,165	166.3 "	
"	7 "	"	44,841	169.3 "	
"	8 "	"	53,617	198.9 "	
"	average	-	-	174.5 "	
Sokamatsubara	6, 1965	19,438	91,013	167.3 "	
"	7 "	"	102,854	168.0 "	
"	8 "	"	103,021	168.3 "	
"	average	-	-	167.9 "	

(After H. Yamashita, Sewage Works Journal, Japan.)

These observations clearly indicate that there may exist an upper limit to the average water consumption of average citizens, and if the water consumption rate has increased and reached to some level, it usually tends to remain on the same level and no further increment may be observed. In other word, the upper limit of the water use may exist between 150 and 200 litres per capita daily, and for the household water use in Surabaja, 140 litres per capita daily will be in good agreement for the 1990 conditions.

(5) Overall use.

Water works supply water to dwellings, mercantile or commercial properties, industrial establishments and public buildings. The water used is classified accordingly. The quantities of water delivered to communities tend towards values with wide variations, because of differences in i) climate, ii) standard of living, iii) extent of sewerage, iv) type of mercantile, commercial and industrial activity, v) cost of water, vi) availability of private water supplies, vii) quality or properties of water for domestic, industrial and other usage, and viii) completeness of metrage etc., but if quantitative ratio of water uses in this city could be assumed, the overall water use may also be easily calculated.

In trying to find out the optimum ratio of the water uses in this city, it may be interest to quote figures of other similar cities that are considered to have the same characteristics in its course of urbanization as that of Surabaja city. Breakdown of the whole municipal water use of Japan, as of 1968, apportions 58.2 per cent to the household use, 24.3 per cent to commercial and small industries, 5.8 per cent to public use and 11.7 per cent to other uses. Other information indicates that the ratio of the household use to the overall use vary widely from 14 per cent to 84.2 per cent depending upon the character of cities, as given in Table 3.

Table 3 Classification Water Use in Cities (%)

Name of cities	Classification of water use								
	Household	Commercial	Industrial	Institution	Public bath	Marine	Miscell	Total	Year
Tokyo	53.36	19.38	12.84	7.38	2.24		* 4.80	100	1966
Yokohama	42.30	24.30	19.00	7.95	3.70		* 2.75	"	"
Nagoya	57.90	21.40	11.30	6.80	2.39		* 0.21	"	"
Osaka	30.00	35.60	18.60	5.61	6.73		* 3.45	"	"
Kobe	42.50	20.00	13.70	9.35	3.83		*10.62	"	1967
Musashino	72.50	14.80	-	12.20	0.35		0.15	"	"
Mitaka	84.20	1.96	7.80	4.38	0.23		1.43	"	"
Chiyoda ward Tokyo	14.00	21.10	1.00	63.90	-		-	"	1965
Koto ward Tokyo	46.90	3.70	30.50	18.90	-		-	"	"
Average Japan	58.20	14.80	9.40	5.80	2.10	0.20	9.30	"	1968

(*) Marine use is included.

As described in Table 3, percentage of the classifications of water use differ widely, e.g. city of Osaka, the second largest city in Japan, having been reputed to be commercial city because of its flourishing commercial activities, the household use occupies only 30 per cent to overall water use as against to 35.6 per cent for commercial use. This is reversed, however, in Musashino and Mitaka cities. These cities have developed as satellite towns of the Tokyo Metropolitan Area, providing residences for commuters to Tokyo or other large cities. Accordingly, commercial and industrial activities are being discouraged in these areas.

Surabaja's city planning has pointed to a modern city with enough green areas for comfortable environment to inhabitants without any such environmental disruptions as water and air pollutions etc. On the one hand, Surabaja's potentiality of rapid industrialization and urbanization are also to be taken into account. Putting all these facts and assumptions together, it may be reasonable to assume the ratio 70 per cent for Surabaja city. Then, the overall household water consumption may be calculated as follows:

$$Q = \frac{C}{A}$$

where Q = overall municipal water use, litre per capita daily,
C = household water use, 140 litre per capital daily, and
A = ratio of household to overall use, 70 per cent,

then

$$Q = \frac{142}{0.7} = 202.8 \fallingdotseq 200 \text{ litre per capita daily.}$$

The ratio of the peak demand of water to the average for day will range from less than 130 per cent to more than 200 percent. Moreover, the ratio of the maximum daily to the average daily demand will range from 120 per cent to almost 200 per cent depending upon the climate, size or character of the city. Although the record of the existing water works is not sufficiently enough to permit estimate of such relations, it is considered that the variation of consumption rates is not so wide, and the following assumptions may be in good agreement with actual conditions:

Average daily per capita flow;	200 litre
Maximum "	250 "
Peak "	400 "

Even though the records of average monthly water consumption and the number of families served by the system are obtained as given in Part 4, it should be noted that if we calculate the per capita consumption rate on the basis of the available information, the figure may have given an excess value than real one, because of the reasons discussed in Part 1. Considering these facts we may take average rate as 80 l for 1972 and 200 l for 1990 through which the rate may increase linearly. Thus we obtained the per capita rates in every five years as given in Table 4.

Table 4 Per Capita Consumption Rates in Every Five Years (litre per day)

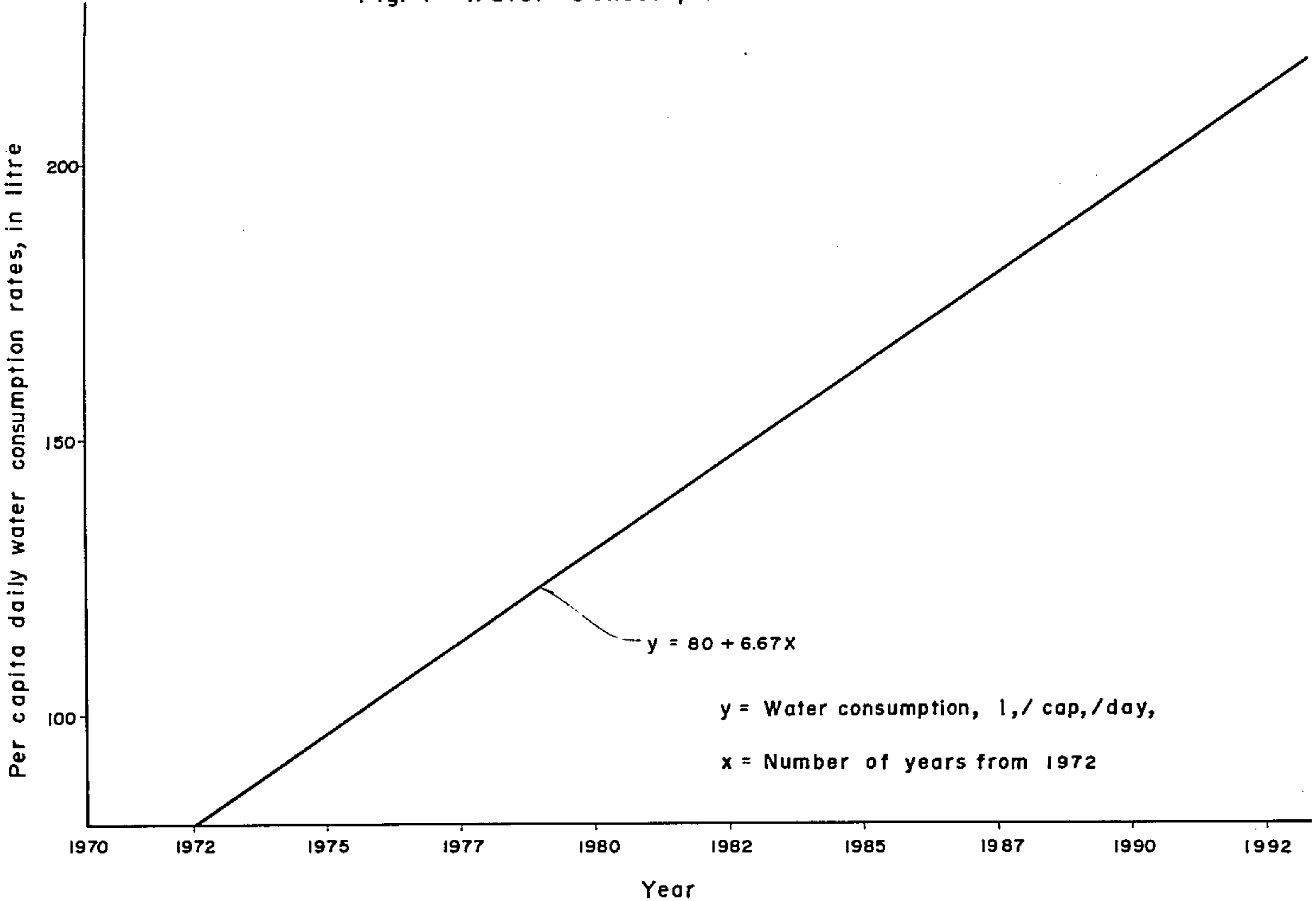
Year	1972	1977	1982	1987	1992
Rate	80	113	147	180	213

Using the figures in the above table, we may be able to estimate the future municipal water requirement as in Table 5.

Table 5 Municipal Water Requirements in Every Five Years (cu m per day)

Year	1972	1977	1982	1987	1992
Population	1,735,000	2,296,000	2,856,000	3,417,000	3,978,000
water requirement	138,800	259,500	419,800	615,100	847,300

Fig. 1 Water Consumption Trends



3. Industrial Water Requirement.

(1) Present condition of industries in Surabaja.

In the city area, there are nearly 2,800 manufacturing establishments ranging from small household scale to modern large scale industry as tabulated in Table 6 below.

Table 6 Present Condition of Manufactures in Surabaja City Area (1967 - 1968)

Serial Nos.	Type of industries	Nos. of classified sizes			Total Nos. of Factories
		Large	Medium	Small	
1	Food	419	155	158	738
2	Tannery	71	9	3	83
3	Chemicals	222	33	62	317
4	Glass	11	13	13	37
5	Jute	96	57	54	207
6	Light industry	102	45	22	169
7	Pottery and concrete	20	24	9	53
8	Rubber	70	31	20	121
9	Wood	235	23	29	287
10	Repairing shop	77	-	-	77
11	Assembling of instrument	12	5	-	17
12	Metal work	275	40	68	283
13	Oil service stn.	145	18	33	196
14	Tobacco	31	15	18	64
	Total	1,786	468	499	2,735

(After the Team Master Plan)

(2) Prediction of future water use.

In evaluating future water demand for industrial purposes, there may be three different measures depending upon the availability of information such as: i) water consumption rate in terms of the unit of daily production of factory, e.g., cu m of spent water per output in money, ii) water consumption rate per unit area of factory's ground or floor, in cu m of spent water per hectare of factory's ground, and iii) water consumed per employee.

The most common way to predict the future industrial water requirement, where firm supporting data are not available, is to survey the present water consumption rate of industries and obtain water consumed per unit area of the industrial zone, then multiply this figure by the future industrial area defined by the city planning.

1) Field survey on major water-using factories.

Industrial survey has been conducted by the engineers. The major water-using factories are visited by the engineers and investigated on the number of factories' employees, products, building and ground areas, and quantity of water used. The water consumption rate per unit ground area is 0.011 cu m/sq m/day as given in Table 7.

Table 7 Industrial Survey of Major Factories in Surabaja and Surrounding Area

Serial Nos.	Name of Factory	Location	Products	Ground Area(m^2)	Building Area(m^2)	Nos. of Employee	Water Use (m^3/day)
1	Petodjo's Ice.	djl. Petodjo	Ice	3,620	2,715	77	9,936
2	Rosela Jute Factory	djl. Kalibokor	Jute	43,829	19,001	1,226	154
3	P.N. Igras.	djl. Ngagel	Glass	13,010	2,800	381	6,048
4	Skin Factory Wong Brothers.	djl. Kendjeran	Skin	8,000	4,000	42	1,382
5	Ngagel's Ice Factory	djl. Ngagel	Ice	5,915	5,549	124	14,688
6	Colibri's Soap Factory	djl. Ngagel	Soap	25,000	14,272	677	11,520
7	Kedurus's Ice Factory	djl. Kedurus	Ice	6,500	5,000	23	2,402
8	P.T. Gaweredjo	"	Under Wear	5,900	2,000	126	87
9	Cement Factory	Gresik	Cement	6,000,000		1,285	(30,240)
10	Petro Kemia	"	Chemical	1,480,000	380,000	750	(12,000)
11	Train Works	Gubeng		100,500	26,030	863	2,592
12	Electrical Manufacturer	Karibokor		26,006	10,402	275	1,944
13	Beer Factory	djl. Kentjana	Beer	26,000	15,549	500	1,409
14	P.N. Pertamina	Wonokromo	Oil	175,753	41,061	563	1,400
15	P.N. Pertamina	djl. Kurukah	Oil	258,000	125,000	41	99
16	P.N.K.A. Sidotopo	Sidotopo		150,853	7,791	1,119	172
	Total			8,328,886	(661,170)	8,072	96,073

2) Future industrial zone.

According to the report by the Team Master Plan Surabaja, the industrial area in 1990 is expected to expand to 2,968 ha. The detailed planning in the zone is, however, not decided yet and it is necessary to assume the future aspect in the city by taking the past and present conditions into account. Component of the industrial area is then assumed as given in Table 8.

Table 8 Assumed Component of Industrial Area in 1990 condition (%)

Residences	Factory ground area	Street, parks etc.
20	50	30

3) Economization of water use.

For the comparision of the result of industrial survey, industrial water use in other countries are also analized. The data are quoted from "the Statistics of Industries in Japan" published in 1971, by the Japanese Government.

Averaged daily water consumption for all manufacturing establishments (not including small establishments with employees of less than 30 persons) in Japan in the year 1969 was in the order of 106,560,000 cu m, in which 74,405,590 cu m pure water was included. The major water-using industries were chemicals, steel and paper. At the same time, total ground area of the factories is 101,654.1 ha, and an average water use per unit ground area is calculated at 731.95 cu m per ha per day. Table 9 is a breakdown of the industrial water use in Japan in 1969.

Table 9 Industrial Water Use in Japan (pure water)

Industry	Water Use cu m/day	Water used for (cu m/day) boiler	product	washing or treatment	cooling	air conditioning	%	Area of factory ground(ha)
Chemicals	25151944	512278	146438	2169809	20478870	1092171	33.8	13964
Steel	14725884	132624	-	1171873	12782434	137213	19.8	13446
Paper	12689252	284206	-	11103597	636932	109239	17.0	4772.6
Other	21838510	878160	418262	5351654	10449642	2743599	29.4	69471.5
Total	74405590	1807268	564700	19796933	44347878	4082222	100.0	
*Per cent	100	2.4	0.8	26.6	59.6	5.5		

*Total of each figure on this line will not be exactly 100%, because recirculated water is excluded from these figures.

It may be seen from the above that nearly 60 per cent of all the water that is used for the manufacturing industries is for cooling purposes. Similar case is also seen in the United States. According to industrial survey, in 1959, more than 90 per cent was used for cooling only. These facts mean that if some suitable measures to reduce the requirement for cooling water were applied, substantial amount of industrial water use could be reduced. Among these measures are to construct an artificial reservoir or pond to recirculate the cooling water, or to build cooling towers for the cooling water. By such measures, some factories have reportedly reduced their water requirements to less than one-tenth of what they used to be.

These facts are substantiated statistically in Japan. The percentage of the recirculated water to all the industrial water was approximately 20 per cent in 1962, 32 per cent in 1964, and 42 per cent in 1965, and is expected to be increased as high as 50 per cent in 1975. Consequently, water use per production unit (or shipment from factory in money) will have been reduced gradually year by year and by 1990 it will reach to the level of 70 per cent to that of present condition.

The most manufacturing establishments are taking their water resources from the rivers.. The river water is not applicable to most processes of the factories without treatment, which requires substantial amount of expenditures for water purifications. This will affect the cost of products and the manufacturer will make their effort to reduce the expenditures for the water purification. Therefore, it would be reasonable to expect that the industrial water demand in 1990 in this city will be reduced to 80 per cent that of present water use rate.

4) Calculation of future demand.

The future industrial water demand is then computed by the following manner:

By dividing the water consumption of the main factories by their ground areas, we obtain the unit water consumption rate in terms of cu m per ha or sq m of the factory's ground area daily as follows,

$$96{,}073 \text{ cu m/day} \times \frac{1}{832.9 \text{ ha}} = 115.34 \text{ cu m/ha of factory ground area/day}$$

above figure is further multiplied by 0.8 since about 20% of the water is expected to be recirculated in future, then basic unit per floor area is,

$$115.34 \text{ cu m/ha/day} \times 0.8 = 92.27 \text{ cu m/ha of factory ground area/day}$$

The manufacturers' ground area will occupy 50% of the total industrial area in future, this finally leading the unit consumption rate of the water in the year of 1990 to;

$$92.27 \text{ cu m/ha/day} \times 0.5 = 46.1 \text{ say } 50 \text{ cu m/ha of industrial area/day}$$

For the comparison, data abroad are also studied. The result was 731.95 cu m/ha of factory's ground area/day, and the unit consumption rate per ha of industrial area is ranging between 50 cu m and 200 cu m depending upon their circumstances. This fact may substantiate the appropriateness of the assumptions.

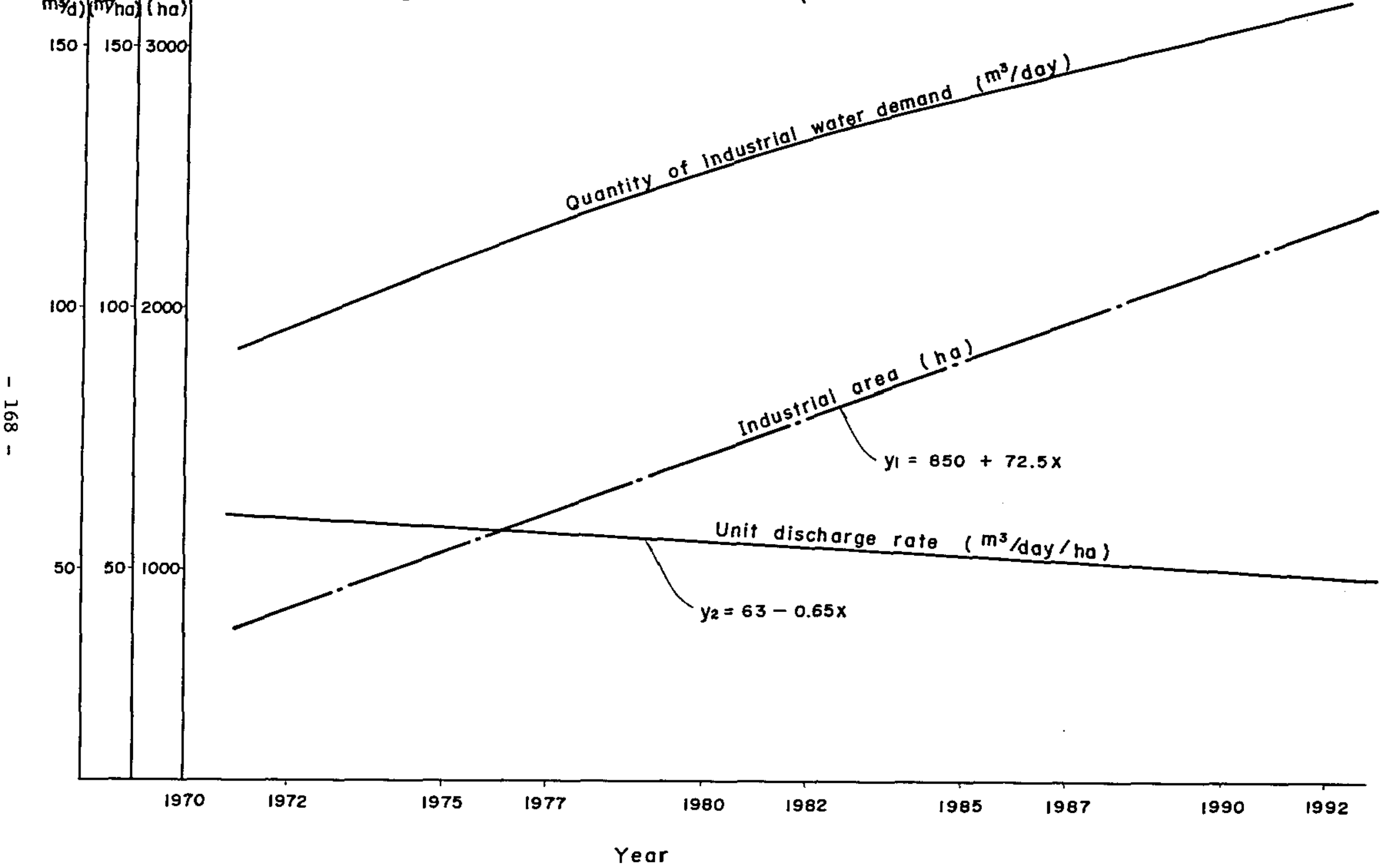
Variation of the industrial water consumption rate is usually not so wide as compared with the municipal water consumption rate. The daily maximum rate of the water will be 120% that of daily average one.

Assuming that the figures both the water requirement and the industrial area may vary linearly from the present ones, we may obtain Fig. 2 from which the overall daily average industrial water requirements in every five years are calculated as given in Table 10 below.

Table 10 Daily Average Industrial Water Requirement (cu m)

Year	1972	1977	1982	1987	1992
Industrial area (ha)	850	1,213	1,575	1,938	2,300
Unit water requirement (cu m/ha/d)	63	60	57	53	50
Water requirement (cu m/d)	53,600	72,800	89,800	102,700	115,000
Water requirement in Gresik (cu m/d)	42,500	42,500	42,500	42,500	42,500
Total	96,000	115,300	132,300	145,200	157,500

Fig. 2 Industrial Water Requirement Trends



4. Water Requirement for Dilution of River Water.

As discussed in a previous section, rivers in the city are have been suffering from the heavy water contamination which is emanating the offensive odours to the wide areas in the central part of the city, and the experiment indicated that the minimum dilution ratio by the fresh water to the river water is five or more. The required dilution water is then calculated for the Mas, Surabaja and Pegirian rivers by the following facts and assumptions:

a. catchment areas of the Pegirian and Mas rivers are 960 and 1,380 ha respectively,

b. the daily average per capita sewage flow may increase linearly from the present 80 l to 200 l by 1992,

c. as an infiltration, nearly 20% to the daily average sewage is to be added,

d. influence by the tide may be neglected,

e. average population density in the catchment are will increase linearly from the present 160 persons to 200 persons per ha by 1992, and

f. *sewage discharge ratio (a quantitative ratio of sewage reached* water way to sewage produced) will increase from the present 0.2 to 0.6 by 1992.

From the above-mentioned facts and assumptions we may estimate the future water requirement as given in the Table 11 below.

Table 11 Future Water Requirement for River Dilution (cu m per day)

Year	1972	1977	1982	1987	1992
*per capita sewage flow (l/d)	96	132	176	216	256
population density (person/ha)	160	170	180	190	200
population served	374,400	397,800	421,200	444,600	468,000
sewage produced (cu m/d)	35,940	54,100	74,100	96,000	119,800
discharge ratio	0.2	0.3	0.4	0.5	0.6
sewage reached to water ways	7,200	16,200	29,600	48,000	71,900
dilution water required	36,000	81,000	148,000	240,000	360,000

* The figures include an infiltration of 20% of the sewage quantity.

CHAPTER II

IRRIGATION WATER REQUIREMENT

1. General.

It is recognized that the irrigation area of the Surabaja river improvement project is the area of which the main irrigation water resources is Surabaja river water in the lower reaches of the Brantas river, especially, in the downstream area from Mlirip Sluice. The area consists of three irrigation systems, namely, the Grompol pump irrigation system (S-1: 277 hectares) *and the Gubeng irrigation system* (W-7, W-8: 2,917 hectares). The Grompol pump station is located at about 12 km downstream from Mlirip Sluice, Gunungsari dam about 40 km and Gubeng dam about 47 km, respectively.

These existing irrigation facilities have been serving for about 100 years after their construction without any fundamental improvement of their function. Only the maintenance work has been executed partly by the East Java Irrigation Service of the Provincial Government.

Most of the gates, sluices and turnouts have been constructed by the design of man power operation, so that it is very difficult to operate them against flood. As a result, it has been hard to ensure planning and practice of effective water management such as intake of water and water distribution to the canal system.

Besides, a lot of sand and silt transported from the upper reaches of the Brantas river entered easily into the irrigation canals and brought a harmful influence upon the capacity of the canals, for instance, narrowing of flow area, meandering of the route and eroding of side slope of the canals.

Basic objective of irrigation is to satisfy the water condition required for the target production of the cultivated crop by applying necessary amounts of water to optimum crop growth from time to time. Therefore, the determination of the water requirements is one of the basic components of the irrigation planning along with the arrangement of the water use facilities, canal system and farm conditions.

In this section, future irrigation water requirements of the project area in which the irrigation area is decreasing contrary to the development of the urban area, has been studied by using the data collected during the field survey and refering to the experimental data in Japan and abroad.

Especially, main crop in the project area is rainy season paddy, dry season paddy, and polowidjo (Soybeans, peanuts and vegetables). Since annual mean air temperature is 26°C and monthly range is only 1°C to 2°C, the cultivation of paddy is possible all the year round if the water condition is satisfied. Under the present condition, transplanting period of paddy is about four-month to five-month in both rainy and dry season.

The strength of irrigation is laid on the supplemental irrigation to the rainy season paddy. There are two kinds of paddy cultivation in dry season, the one is dry season paddy regulated and the other is dry season paddy non-regulated.

The former is planned to supply irrigation water by the Government (through Irrigation Service office), the latter is not guaranteed for irrigation water supply but expected by farmer surplus water from the upstream area and rare rainfall during the season.

As the subject in this section is to take a large view of future irrigation water demand, all the crop cultivated in the project area has been considered as the crop to be irrigated in the calculation.

Generally, unit water requirement of paddy is different by the growth stage. Under the condition that paddy at different growth stages is being cultivated in a irrigation block, estimation of water requirement of the area in every calendar month would be very difficult unless the cropping ratio or area of the different growth paddy has been already known.

So the following procedure has been taken into the study.

a. The study to find the transplanted area by month in each irrigation block has been examined on the basis of the monthly planted area of paddy in the last 7 years which have been collected and evaluated in the field survey.

b. The areal average growing ratio of the paddy has been calculated by irrigation block and calendar month.

c. Normal unit water requirement of paddy has been decided corresponding to the relative growth of paddy.

d. Monthly irrigation water requirement of each irrigation block has been calculated by multiplying planted area by normal unit water requirement.

e. Water shortage or surplus of distributed irrigation water has been checked up comparing the calculated values with the recorded ones by the Irrigation Service, Surabaja.

f. Anticipated irrigation acreage in the Surabaja city area has been studied on the basis of decreasing trend of them in the last 8 years.

g. Future irrigation water requirement has been estimated using the calculated water requirement and anticipated irrigation acreage in the Surabaja city area.

2. Present Planted Area under Crop in Each Irrigation Block and in the Brantas Delta.

The planted area of crops every month for the 7-crop-year (October, 1964 to September, 1970) has been collected together with the Indonesian counterparts, from the data of "Daftar Pertanaman" which is the reporting table prevailing under Irrigation Service on planted area and irrigation water distribution at intervals of 10-day.

In both rainy and dry season, it seems that starting time of the paddy cultivation was not fixed clearly. It may be dependent on the Status of the Surabaja river discharge as the water resources and harvest condition of the previous crop in the field. In addition to the above it would be a result of governmental arrangement on fairness of irrigation so that the priority of water intake might be rotated yearly from one irrigation block to the others.

In this study, it has been decided that the average value of the 7-year records is the basic acreage of calculation of irrigation water. Average value of crop-growing area in each irrigation block has been given in Table 1-1 to 1-9 in the chapter IRRIGATION WATER REQUIREMENT in Part 4.

And also the value in the Brantas Delta which is adjacent to the project area has been shown in Table 1-10 in the chapter above mentioned for reference.

3. Calculation of Monthly Growing Ratio and Average Plant Height of Crop in Each Irrigation Block.

In the project area, there are some paddy varieties in both rainy and dry season whose growing period is different each other. Although PB-variety which is encouraged by the government is going to prevail over the project area, local variety is still cultivated partly.

In calculating irrigation water requirement, areal average growing ratio of the paddy in each irrigation block each month has to be given. On the other hand, if the aereal average plant height of the paddy corresponding with areal average growing ratio is known by month by irrigation block, flood damage of the area in rainy season can be estimated on the basis of those data.

To cope with these requirements, areal average growing ratio and plant height of paddy in each irrigation block have been assumed on the basis of the following consideration and means. The calculation has been shown in Table 2-1 thru 2-24, and that of the Brantas Delta in Table 2-25 thru 2-28 in the Chapter Irrigation Water Requirement in Part 4.

a. Planted area of paddy by month by each irrigation block is equal to the figures of Table 1-1 thru 1-10 in the above-mentioned chapter in Part 4.

b. It is assumed that growing period of paddy will be 4-month, 5-month, 6-month and 7-month.

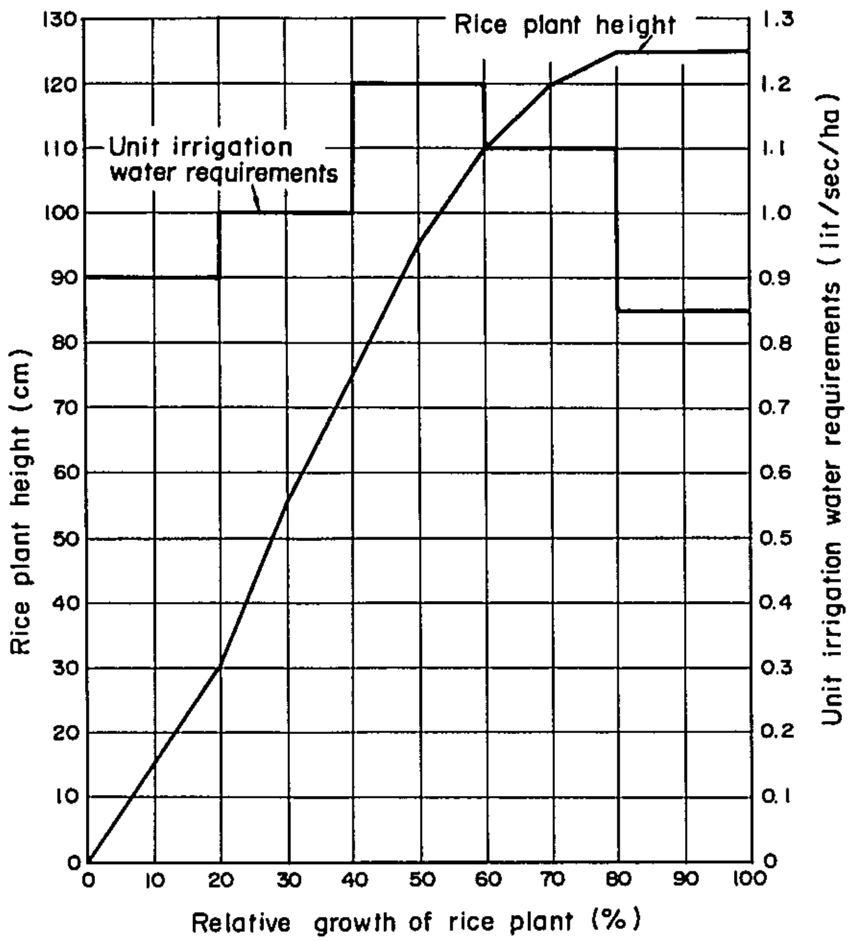
c. Difference of acreage between consecutive two calendar month total of transplanted area and harvested area in the latter calendar month.

d. Transplanted acreage of paddy having a proper growing period has been assumed and the total growing acreage of several kind of paddy in one calendar month has been checked so as to be equal to the total of growing acreage of 7-year average as shown in the above-mentioned Table 1-1 thru 1-10, respectively.

e. Assuming that seedling of one month old was used to be transplanted, a growing ratio of every paddy has been calculated. In order to find areal average growing ratio, weighted calculation has been executed.

f. Data on plant height corresponding to growing stage of cultivating paddy in the project area were collected from the Surabaja City Agricultural Office and East Java Provincial Agricultural Office. From them one typical elongation curve of paddy plant corresponding to relative growth has been derived as shown in Fig. 3.

Fig. 3 Rice Plant Height and Unit Irrigation Water Requirements vs. Relative Growth of Rice Plant



g. Using the curve decided above, areal average plant height has been given monthly at the last line of the above-mentioned Table 2-1 thru 2-27.

4. Field Delivery Water Requirement of Paddy.

Direct measurement method and calculation method by some empirical formula have been used to determine unit water requirement of crop. These methods, however, have been left for further detail survey and study since the object of this study was aimed at finding the future irrigation water demand in its broader aspects.

For the time being, the actual irrigation water distribution standard by the Irrigation Service Office and the figures which were proposed in the Brantas Delta Irrigation Rehabilitation Project have been studied in relation to the growing ratio of paddy (relative growth of paddy) and adjusted figures have been applied to the project area, because such important factors as climatic, land, water, and crop cultivating conditions of the project area are quite similar to the Brantas Delta area.

Adopted figures in the calculation are as follows;

a. Puddling water requirement (for land preparation)

Crop	Seed bed (mm)	Puddy field (mm)
Rainy season paddy	300	250
Dry season paddy	200	150

b. Field delivery unit water requirement of paddy (evapotranspiration and percolation)

Relative Growth (%)			Field delivery unit water requirement of paddy (lit/sec/ha)
1	to	20	0.90
21	to	40	1.00
41	to	60	1.20
61	to	80	1.10
81	to	100	0.85

Above figures are also shown in Fig. 3.

5. Field Delivery Water Requirement of Polowidjo.

Cultivation of polowidjo in the project area is limited to very small area. Main crop of polowidjo is tubers, pulses and vegetables. Field delivery water requirement for these has been decided at 0.5 lit/sec/ha referring to the figures of Brantas Delta Irrigation Rehabilitation Project.

6. Calculation of Field Delivery Water Requirement and Diversion Water Requirement in Each Irrigation Block.

Seed bed water requirement including the one for puddling water for bed preparation has been calculated for each irrigation block as shown in Table 3-1 thru 3-5 in the chapter of irrigation water requirement in Part 4.

Puddling water requirement of paddy field for each irrigation block has been calculated as shown in Table 3-6 thru 3-10 in above-mentioned chapter in Part 4.

Field delivery water requirement of paddy field for each irrigation block has been calculated as shown in Table 3-11 thru 3-15, and that of polowidjo in Table 3-16 in the above-mentioned chapter in Part 4.

Field delivery water requirement for crop by each irrigation block has been tabulated by summing up of each crop water requirement. In the calculation effective rainfall during the cropping season has been neglected and conservative figures of water requirement have been obtained.

Diversion water requirement at the intake site of each irrigation block has been calculated by assuming that overall losses of canal system are 30 percent in consideration of the total length and capacity of the canals. The result has been shown in Table 3-17 thru 3-25 in the above-mentioned chapter in Part 4.

Table 12 gives the whole irrigation water requirements of the project area and Table 13 shows the whole cropping area in the project area.

According to the above calculation, the maximum diversion water requirement of the project area was found at 12.24 cu m/sec in March. Fortunately, March is belonging to rainy season, so it is expected that struggle among beneficiary of water use will not become so serious.

Table 12 Diversion Water Requirements of Each Irrigation Block

cu m/sec

	W-1	W-2	W-3	W-4	W-5	W-6	W-7	W-8	Total	S-1	Total
Oct.	0.14	1.56	0.06	0.13	0.04	0.07	0.53	0.86	3.39	0.01	3.40
Nov.	0.12	0.98	0.04	0.10	0.27	0.03	0.31	0.31	2.16	0.01	2.17
Dec.	0.28	0.80	0.08	0.16	0.78	0.27	1.27	1.23	4.87	0.06	4.93
Jan.	0.45	1.23	0.07	0.15	0.73	1.37	1.92	2.80	8.72	0.33	4.05
Feb.	0.56	2.28	0.10	0.23	0.68	2.78	2.24	2.43	11.30	0.49	11.79
Mar.	0.51	2.90	0.09	0.22	0.44	2.36	1.84	3.52	11.88	0.36	12.24
Apr.	0.44	2.23	0.05	0.17	0.35	2.01	1.09	1.92	8.26	0.27	8.53
May	0.31	1.49	0.04	0.11	0.09	1.23	0.59	0.88	4.74	0.15	4.89
Jun.	0.34	1.24	0.07	0.15	0.12	0.30	0.47	0.65	3.34	0.05	3.39
Jul.	0.35	1.16	0.09	0.18	0.15	0.15	0.87	1.40	4.35	0.06	4.41
Aug.	0.29	1.80	0.10	0.20	0.11	0.16	1.24	1.72	5.62	0.05	5.67
Sep.	0.22	2.04	0.10	0.21	0.07	0.09	0.95	1.26	4.94	0.03	4.97

Table 13 Crop-Growing Area of Each Irrigation Block

	W-1	W-2	W-3	W-4	W-5	W-6	W-7	W-8	Total	S-1	Total
Oct.	104	1,002	40	88	34	54	426	607	2,355	16	2,371
Nov.	66	785	29	66	108	19	186	141	1,400	5	1,405
Dec.	118	613	38	82	325	104	456	446	2,182	20	2,202
Jan.	209	579	41	80	367	517	800	1,172	3,765	118	3,883
Feb.	263	971	47	105	397	1,186	1,041	1,283	5,293	223	5,516
Mar.	293	1,359	50	123	361	1,287	1,107	1,796	6,376	227	6,603
Apr.	274	1,436	41	106	225	1,282	899	1,579	5,842	223	6,065
May	224	1,180	26	78	64	1,007	468	699	3,746	126	3,872
Jun.	102	906	36	82	63	230	228	311	1,957	62	2,019
Jul.	194	678	48	103	83	75	437	691	2,309	80	2,389
Aug.	192	921	56	109	80	88	625	892	2,963	73	3,036
Sep.	146	1,096	60	120	61	61	603	904	3,051	44	3,095

7. Status of Irrigation Water Shortage under Present Condition.

The records of distributed water for irrigation in the Wonokromo area and diversion water requirement of the Wonokromo area has been compared, and the balance has been calculated as shown in Table 14. The calculation was performed monthly in 1964 and 1970 crop year. Also discharge of the Surabaja river was referred in the same Table.

Table 14 Balance Calculation of Irrigation Water

Month	Discharge of Surabaja river cu m/sec (A)	Distributed water cu m/sec (B)	Irrigation water req't cu m/sec (C)	Balance cu m/sec (D) = (B) - (C)
1964 crop year				
1964 Oct.	119.7	4.71	3.39	1.32
Nov.	75.2	7.41	2.16	5.25
Dec.	37.3	9.87	4.87	5.00
1965 Jan.	62.9	11.57	8.72	2.85
Feb.	127.6	11.94	11.30	0.64
Mar.	73.9	11.05	11.88	-0.83
Apr.	48.0	10.38	8.26	2.12
May	23.9	8.02	4.74	3.28
Jun.	24.0	5.99	3.34	2.65
Jul.	10.0	3.88	4.35	-0.47
Aug.	9.2	2.60	5.62	-3.02
Sep.	6.9	1.76	4.94	-3.18
1970 crop year				
1970 Oct.	4.7	4.33	3.39	0.94
Nov.	32.9	6.65	2.16	4.49
Dec.	36.2	5.54	4.87	0.67
1971 Jan.	69.8	6.46	8.72	-2.26
Feb.	109.6	5.59	11.30	-5.71
Mar.	86.9	6.31	11.88	-5.57
Apr.	69.5	6.80	8.26	-1.46
May	68.0	6.75	4.74	2.01
Jun.	50.7	6.65	3.34	3.31
Jul.	24.2	6.65	4.35	2.30
Aug.	9.0	5.79	5.62	0.17
Sep.	10.5	5.10	4.94	0.16

As can be seen from the above table, the Surabaja river discharge in rainy season (from November to April) was sufficiently over irrigation water demand in 1964 as well as 1970.

It seems that the distributed water to the irrigation area from October 1964 to May 1965 well met irrigation water requirement excluding March. However, considerable amounts of water shortage were found in rainy season especially from January to April 1971.

This would be a result of the facts that such main irrigation facilities as Gunungsari dam, Gubeng dam, and related canal system have been deteriorated extremely by the reason of overage and the operation of them has become troublesome, moreover the capacity of canals has been decreased remarkably.

When the records of intake of water are looked over year after year, the tendency of deterioration can be seen obviously. The records have been arranged as shown in Table 4-1 thru 4-8 in the chapter of irrigation water requirement in Part 4.

On the other hand, water requirement in dry season is lower than that in rainy season. Shortage of water intake in dry season would be caused mainly by decrease of Surabaja river discharge rather than difficulty of intake resulting from the deteriorated facilities. It means that ajustment among beneficiary of water use is important and necessary in dry season unless the Surabaja river water is increased by the regulation of water resources in upper basin of the Brantas river.

8. Anticipated Irrigation Acreage in the Surabaja City Area.

Transition of irrigation area in the Surabaja city area has been investigated and arranged as shown in Table 15.

Table 15 Transition of Irrigation Acreage in Surabaja City (as of January)

unit; ha

Irrigation Block	1964	1965	1966	1967	1968	1969	1970	1971
W-1 Simowau	404	404	387	387	387	387	387	387
W-2 Kebonagung	1,520	1,520	1,511	1,511	1,511	1,511	1,511	1,511
W-3 Djambangan	62	62	62	62	62	62	62	62
W-4 Karah	129	129	129	129	129	129	129	129
W-5 Rowowijung	430	430	430	430	430	430	430	430
W-6 Gunungsari	1,319	1,319	1,319	1,319	1,319	1,319	1,293	1,293
W-7 Kalibokor	1,143	1,143	1,143	1,129	1,129	1,129	1,109	1,109
W-8 Djeblokan	1,824	1,824	1,824	1,824	1,808	1,808	1,808	1,808
Total	6,831	6,831	6,805	6,791	6,775	6,775	6,729	6,729
Difference		0	-26	-14	-16	0	-46	0

Source: "Daftar Pertanaman" East Java Irrigation Service, Section Wonokromo

To estimate future irrigation acreage of the Surabaja city area, approximation equation has been obtained using the least square method for the above figures.

When equation is assumed to be linear, it is;

$$Y = 6,854.5 - 15.8\ t \qquad (1)$$

where Y: irrigation acreage (ha) after t-year
t: elapsed year from 1963 of calendar year
(t = 1 ...... 1964)

When equation is assumed to be exponential, it is;

$$Y = 7,000 - 5.035\ e^{0.073t} \qquad (2)$$

where Y: irrigation acreage (ha) after t-year
t: elapsed year from 1963 of calendar year
(t = 1 ...... 1964)

According to the above two expressions, obtained figures have been compared with actual records as shown in Table 16, and illustrated in Fig. 4.

Table 16 Comparison of Figures

Year	t	Actual Record	Eq.(1)	Error	Eq.(2)	Error
1964	1	6,831	6,839	-8	6,835	-4
1965	2	6,831	6,823	8	6,822	9
1966	3	6,805	6,807	-2	6,809	-4
1967	4	6,791	6,791	0	6,794	-3
1968	5	6,775	6,775	0	6,779	-4
1969	6	6,775	6,760	15	6,762	13
1970	7	6,729	6,744	-15	6,744	15
1971	8	6,729	6,728	1	6,724	5

Note $\sigma_1 = 8.45 \quad \sigma_2 = 8.33$

As can be seen from the standard deviation of σ_1=8.45 and σ_2=8.33 in Table 16, fitness of obtained equations to the actual record is found at same degree. If data more than 5 years can be got for the calculation, it will be clear which expression is more suitable to the on-going tendency of decrease.

From the equation obtained above, future irrigation acreage of the Surabaja city area has been assumed as shown in Table 17.

Fig.4 Perspective Irrigation Area in Surabaja City Area

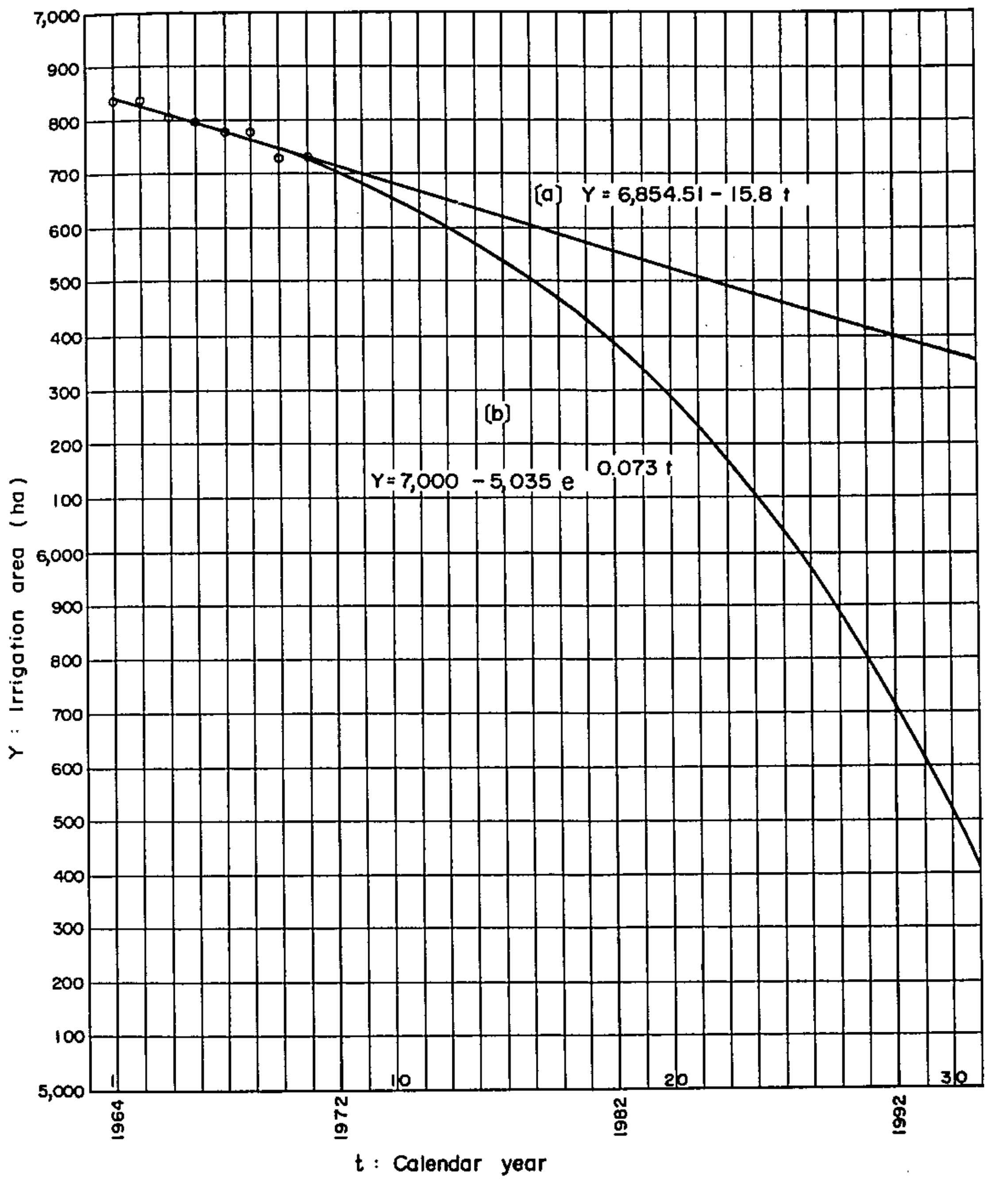


Table 17 Future Irrigation Acreage of Surabaja City

N	Calendar Year	Irrigation Acreage: Straight Line Expression ha	Irrigation Acreage: Exponential Curve Expression ha	t	N	Calendar Year	Irrigation Acreage: Straight Line Expression ha	Irrigation Acreage: Exponential Curve Expression ha	t
1	1973	6,696	6,681	10	26	1998	6,301	5,022	35
2	1974	6,681	6,657	11	27	1999	6,286	4,872	36
3	1975	6,665	6,631	12	28	2000	6,270	4,711	37
4	1976	6,649	6,603	13	29	2001	6,254	4,538	38
5	1977	6,633	6,573	14	30	2002	6,238	4,352	39
6	1978	6,617	6,547	15	31	2003	6,222	4,151	40
7	1979	6,602	6,506	16	32	2004	6,207	3,934	41
8	1980	6,586	6,468	17	33	2005	6,175	3,703	42
9	1981	6,570	6,428	18	34	2006	6,159	3,453	43
10	1982	6,554	6,385	19	35	2007	6,143	3,184	44
11	1983	6,538	6,338	20	36	2008	6,128	2,852	45
12	1984	6,523	6,291	21	37	2009	6,112	2,585	46
13	1985	6,507	6,234	22	38	2010	6,096	2,253	47
14	1986	6,491	6,176	23	39	2011	6,080	1,891	48
15	1987	6,475	6,113	24	40	2012	6,064	1,504	49
16	1988	6,459	6,046	25	41	2013	6,049	1,088	50
17	1989	6,444	5,975	26	42	2014	6,033	230	51
18	1990	6,428	5,897	27	43	2015	6,017	159	52
19	1991	6,412	5,813	28	44	2016	6,001	0	53
20	1992	6,396	5,724	29	45	2017	5,985	0	54
21	1993	6,380	5,628	30	46	2018	5,970	0	55
22	1994	6,365	5,523	31	47	2019	5,954	0	56
23	1995	6,349	5,411	32	48	2020	5,938	0	57
24	1996	6,333	5,292	33	49	2021	5,922	0	58
25	1997	6,317	5,161	34	50	2022	5,906	0	59

Nowadays Surabaja city is developing rapidly as a center of politics and economy, it is expected that conversion of farm land to some other purpose would be increased largely in accordance with expansion of public and private investment.

On the other hand the Surabaja city planning team has assumed that the farm land would be diminished year by year and converted completely to urban area by 1992 excluding green belt zone along the boundary of Surabaja city. It is considered that this assumption may not be realized unless powerful political efforts by the government be paid for the performance.

Actually considerable efforts by the government can be expected for the development of Surabaja city, then it is judged that decreasing tendency is not always keeping a form of a straight line expression preferably of a exponential curve expression.

From this consideration, it can be said at least that real irrigation acreage of the Surabaja city area will be a figure between the straight line and exponential curve derived above.

9. Estimated Future Demand of Irrigation Water in the Surabaja Project Area.

Future demand of irrigation water of the Surabaja project area has been calculated from the estimated irrigation water requirement under the existing condition and estimated future irrigation acreage.

For a conservative calculation, the following has been taken into consideration;

a. Future irrigation acreage of the Surabaja city area is dependent upon the straight line expression.

b. Future irrigation acreage of the Grompol irrigation block is the same as present figure because its location is far from the city center and agriculture will not be changed.

Table 18 gives the result of the calculation of future irrigation water demand at intervals of 5 years. The maximum demand of 12.24 cu m/sec in March at present will be reduced to 10.81 cu m/sec in March, 2022.

Table 18 Estimated Future Demand of Irrigation Water in the Surabaja Project Area

Calendar year	Diversion Water Requirements											Future Irrigation Water Demand									
	S-1 Area Present thru 2022	Surabaja city Area (Wonokromo)																			
		Present	1977	1982	1987	1992	1997	2002	2007	2012	2022	Present	1977	1982	1987	1992	1997	2002	2007	2012	2022
%	100	100	98.6	97.4	96.2	95.0	93.9	92.7	91.5	90.3	88.0										
Oct.	0.01	3.39	3.34	3.30	3.26	3.22	3.18	3.14	3.10	3.06	2.98	3.40	3.35	3.31	3.27	3.23	3.19	3.15	3.11	3.07	2.99
Nov.	0.01	2.16	2.13	2.10	2.08	2.05	2.03	2.00	1.98	1.95	1.90	2.17	2.14	2.11	2.09	2.06	2.04	2.01	1.99	1.96	1.91
Dec.	0.06	4.87	4.80	4.74	4.68	4.63	4.57	4.51	4.46	4.40	4.29	4.93	4.86	4.80	4.74	4.69	4.63	4.57	4.52	4.46	4.35
Jan.	0.33	8.72	8.60	8.49	8.39	8.28	8.19	8.08	7.98	7.87	7.67	9.05	8.93	8.82	8.72	8.61	8.52	8.41	8.31	8.20	8.00
Feb.	0.49	11.30	11.14	11.01	10.87	10.74	10.61	10.48	10.34	10.20	9.94	11.79	11.63	11.50	11.36	11.23	11.10	10.97	10.83	10.69	10.43
Mar.	0.36	11.88	11.71	11.57	11.43	11.29	11.16	11.01	10.87	10.73	10.45	12.24	12.07	11.93	11.79	11.65	11.52	11.37	11.23	11.09	10.81
Apr.	0.27	8.26	8.11	8.05	7.95	7.85	7.76	7.66	7.56	7.46	7.27	8.53	8.41	8.32	8.22	8.12	8.03	7.93	7.83	7.73	7.54
May	0.15	4.74	4.67	4.62	4.56	4.50	4.45	4.39	4.34	4.28	4.17	4.89	4.82	4.77	4.71	4.65	4.60	4.54	4.49	4.43	4.32
Jun.	0.05	3.34	3.29	3.25	3.21	3.17	3.14	3.10	3.06	3.02	2.94	3.39	3.34	3.30	3.26	3.22	3.19	3.15	3.11	3.07	2.99
Jul.	0.06	4.35	4.29	4.24	4.18	4.13	4.08	4.03	3.98	3.93	3.83	4.41	4.35	4.30	4.24	4.19	4.14	4.09	4.04	3.99	3.89
Aug.	0.05	5.62	5.54	5.47	5.41	5.34	5.28	5.21	5.14	5.07	4.95	5.67	5.59	5.52	5.46	5.39	5.33	5.26	5.19	5.12	5.00
Sep.	0.03	4.94	4.87	4.81	4.75	4.69	4.64	4.58	4.52	4.46	4.35	4.97	4.90	4.84	4.78	4.72	4.67	4.61	4.55	4.49	4.38

Note: The figures on %-line were obtained from the comparison between the present irrigation acreage and the estimated future irrigation acreage by the straight line expression.

CHAPTER III

OVERALL WATER REQUIREMENT

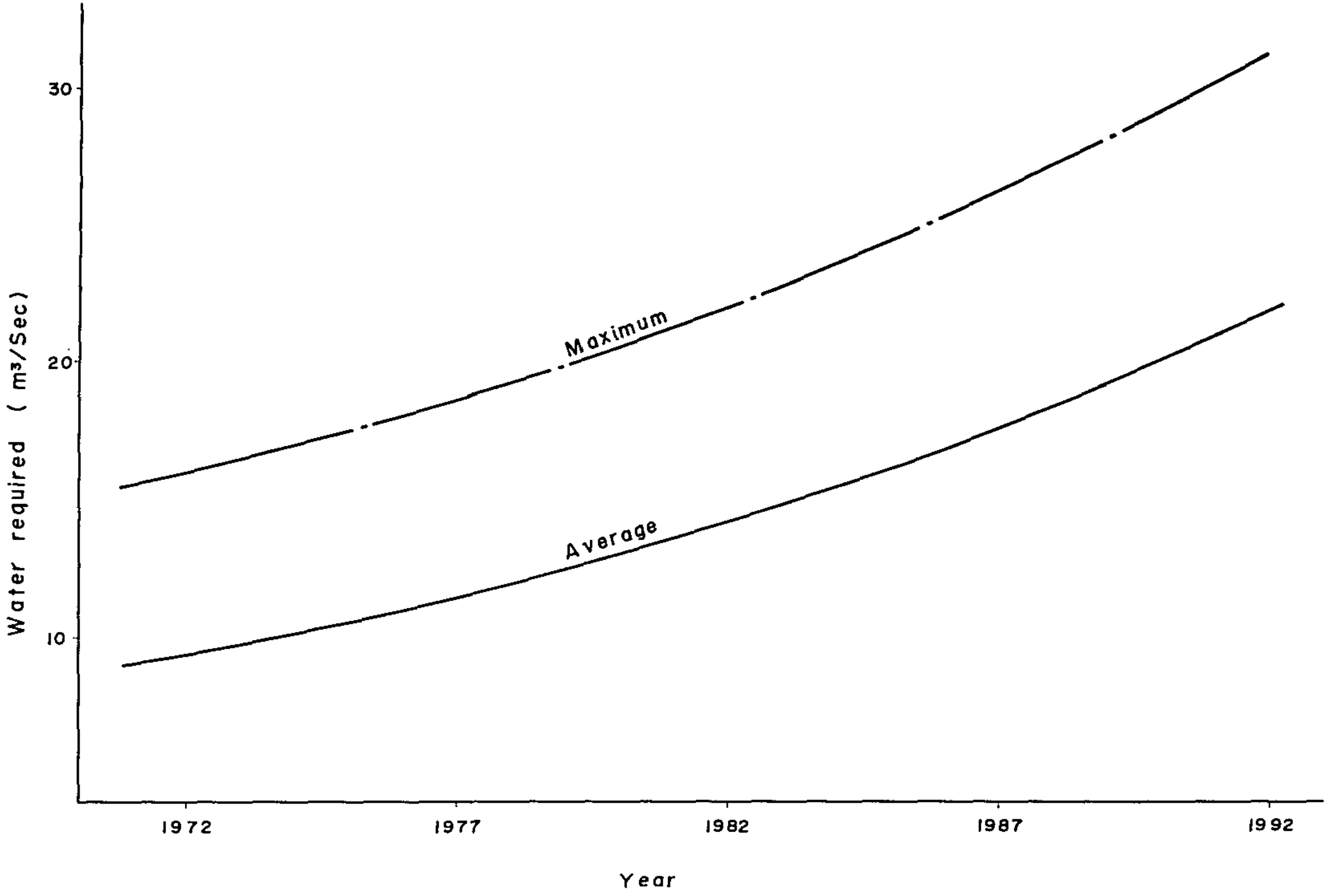
The future water requirement in every five years in the Surabaja area have been estimated in previous chapters. The estimated water requirements for different uses may gradually increase except the irrigation use as given in the following table, and illustrated in Fig. 5.

Table 19 Water Requirements Estimation (cu m per second)

Year	1972		1977		1982		1987		1992	
water use	Ave.	Max.	Ave.	Max.	Ave.	Max.	Ave.	Max.	Ave.	Max.
Municipal	1.60	1.90	3.00	3.75	4.86	6.08	7.12	8.90	9.81	12.26
Industrial	1.11	1.33	1.33	1.60	1.53	1.84	1.68	2.02	1.82	2.18
River Dilution	0.42	0.53	0.94	1.18	1.71	2.14	2.78	3.48	4.17	5.13
Irrigation	6.29	12.24	6.20	12.07	6.13	11.93	6.05	11.79	5.98	11.65
Total	9.42	16.00	11.47	18.60	14.23	21.99	17.68	26.19	21.78	31.22

Note: Figures for maximum water requirement are calculated by multiplying the average values by the factors, 1.25 for Municipal and River Dilution and 1.2 for Industrial water.

Fig. 5 Estimation of Future Water Requirement



APPENDIX A

TERMS OF REFERENCE

FOR

THE SURABAJA RIVER IMPROVEMENT PROJECT

Directorate General of Water Resources Development

Ministry of Public Works and Power

Republic of Indonesia

I. Back Ground Information of the Project. (omitted)

1. General.

2. Present aspect of the project.

II. Scope of Work.

To meet the objectives described the services of experienced Engineering consultant are required who will assist the Directorate River and Swampy Area Development.

These services will include the following:

1. Study for improvement of the Surabaja river, including Wonokromo canal which is purposed for flood control, drainage, etc. For this purpose the consultant work must be based on the following;
   - allocation of design discharges.
   - preliminary design of river channels.

2. Study for rehabilitation of drainage and sewerage systems of the Surabaja city and its hinterland.
   For this purpose the consultant work must be based on the following terms:
   - allocation of drainage areas and examination of drainage discharges.
   - examination of the capacity of pumping stations.

3. Examine the present conditions of Surabaja river, including drainage and sewerage systems of the Surabaja city, and to suggest for necessary rehabilitation and improvement work.

4. Examine the present water demand for municipal & industrial supply and irrigation, and to suggest for necessary improvement works.

5. Examine the economic feasibility for rehabilitation and improvement project, including:

   a. Assessment of production in damages as a result of the project.

   b. Quantification of other benefits from improvements to drainage and sewerage systems.

c. Determination of investment cost, broken down into local and foreign exchange components and taking into account alternative modes of execution of work.

d. Assessment of operation and maintenance.

e. The economical evaluation should include land acquisition.

f. Evaluation of the basis of the mentioned above and other relevant factors of economic justification of the project.

III. Expertise. (omitted)

APPENDIX B

QUESTIONNAIRES SUBMITTED BY THE STUDY TEAM

1. Questionnaire Submitted to DPUT.

(1) Questions in general.

1) The reason why water level is kept high by Wonokromo sluice and Djagir barrage.

2) The reason why water level is kept high by Gubeng sluice.

3) Whether the locks of Wonokromo sluice and Gubeng sluice are necessary or not.

4) Whether the Marmojo river can be left as it is, or not.

5) What is the purpose of the Gedeg river and its sluice (Gedeg sluice)? Whether the gate is allowed shut during flood, or not.

6) Whether the Kedungsoro river is connected to the Brantas river, or not.

7) Do you have any plan to construct some storage ponds in landside of the sea-dike which extends from the mouth of the Pegirian river to the Wonokromo canal?

8) Whether the Gunungsari canal unites with the Lamong river, or not.

9) Whether the syphon of the Watudakon river has the effective function at present.

10) Authorities to manage the Surabaja river, the Mas river, the Pegirian river and the Wonokromo canal.

11) Authorities to control the sluices and the barrage on the rivers described above.

12) Names of laws and/or regulations concerning rivers and/or river structures, and competent authorities concerned.

13) We want to confirm that the word "sewerage" mentioned in the scope of work in the terms of reference for the Surabaja river improvement project does not contain sewage disposal, but means only town drainage of storm water.

14) What is the extent that the word "hinterland" means in the scope of work in the terms of reference for the Surabaja river improvement project?

15) We want to know the short history of the improvement works of the Surabaja river, the Porong river and relevant rivers.

16) Have you ever experienced any inundation in Surabaja City and its hinterland caused by over-topping or breaking of levee of the Surabaja river?

(2) Request for major data.

1) Research papers on rainfall and runoff in Java Island, if any.

2) Aero-photograph of the Surabaja river basin within the extent shown in the following topographic map No. (scale: 1/50,000)

			54/XL-C	54/XL-D & 55/XL-C
52/XL1-B	53/XL1-A	53/XL1-B	54/XL1-A	54/XL1-B & 55/XL1-A
	53/XL1-C	53/XL1-D		

3) The typical cross-section of Surabaja river showing the elevation of both agricultural land located on each bank of the river and mean water level through a year in the Surabaja river.

4) Data on groundwater table observed on agricultural land located along both banks of the river.

5) Both agricultural development plan. formulated and maps of drainage system and irrigation system for the agricultural land located on the left side of the Surabaja river.

6) Soil map, land use map, geological map and land classification map for the area located on both banks of the Surabaja river.

(3) Request for data in detail. (omitted)

2. Questionnaire Submitted to the City Master Plan Team of Surabaja.

(1) Questions in general.

1) The situation of the Master Plan Team on the organization chart of Municipality of Surabaja and the function and authority of the team. The year of establishment of the team.

2) Organization chart of the team including the relation to the other organizations such as Propinsi Djawa Timur and the central government.

3) The area which is dealt with by the team.

4) Do you have any booklet which describes the history of the improvement works of the Surabaja river and the Mas river which include Gunungsari dam, Wonokromo canal, Wonokromo sluice, Djagir sluice, Gubeng dam, and Pegirian gate. If you have not such booklet, short description about the above.

5) What is the purpose of damming up river water by Wonokromo and Djagir sluices?

6) What is the purpose of damming up river water by Gubeng dam?

7) The locks of Wonokromo sluice and Gubeng sluice seem to be unnecessary. How do you think about this?

8) What is the purpose of boezem? Is there any trouble with the boezem?

9) Are there any actual damages in rice fields during rainy season? If there are, what is the kind of damage and what is the amount of damage?

(2) City planning.

In the city plan of Surabaja, the whole city area is designated to several different zones -, in which population of approximately 4.8 million is expected to inhabit by 1990.

In respect to the city planning we have the following questions:

1) What kind facilities or housings will be provided in the zones such as block of low density, forest for hunting and recreational area?

2) Do you have any population plan for the zonings?
Or, do you have population density plan to support the population distribution of 4.8 million in 1990?

3) What is area (ha) of each zone for industry, residence, commerce, agriculture and public use?

4) Time and cost schedule of construction of future highway?

5) In the industrial area, what kind of factories will be settled, and what are sizes and numbers?
This information, if available, will be helpful to predict future industrial water demand.

6) Present population densities in the central part of the city is considered to be unusually high. Are these densities based upon the result of investigation or merely guessed?

7) Future population of the city.

According to the "Master Plan Assainering" the future populations of the city are predicted as 3.04 million for the year 1980 and 4.8 million for 1990 respectively.

We understand that those figures are projected from year 1966, population of 1,895,056, assuming that the annual population growth rate is 4 per cent.

However, the census reports after the year 1966 indicate remarkable variations, especially between 1967 and 1968, sudden decrease of more than one million was observed.

This might be caused due to the social problems and will affect to the population estimate greatly. Therefore some considerations are to be given to the facts.

For the computation of future municipal water demand we have to catch up as accurate future population as possible, and the following question has been arisen:

Do you have any revised population estimate to supersede the former one?

(3) Present conditions of urban drainage system.

Regarding the operational and maintenance problems of the drainage system, we have the following opinions and questions:

1) From our inspection it seems that their capacities are, after rehabilitated, enough to lift the inflowing runoff mainly because the rainfall is depressed in the tributary areas, thus making the runoff coefficient low. This fact was confirmed by inspection carried out during heavy rainfall. Eventhough having heavy rainfall in their tributary area the water level in the pump well had not been risen. If there is any pumping station which is suffering from frequant floodings, please tell us the condition.

2) For flooding areas, what kind of measures do you intend to take as emergency relief works?
If the function of pumping station is suspended, which pumping station do you think is the most important one, from the damage and public health view point?

(4) Agriculture.

1) What is the base of your prospect that the agricultural field will be changed completely into the town area in 1990?
How is the decrease trend of agricultural field in Kotamadya Surabaja for last 10 years?

2) As for the agriculture in Kotamadyo Surabaja, what opinion do you have? Whether the agriculture should be encouraged or not?

3) As for the existing irrigation facilities, do you have the idea of utilizing them in future as the conveyance system of the city water required?

4) As for the regulation on throwing execration of human being or other dirty material down into irrigation canal, do you have a program to execute?

5) As for the drainage system of existing paddy field, what idea or program do you have?

3. Questionnaire Submitted to Surabaja City.

1) Is there any booklet which describes the history of the improvement works of the Surabaja river and the Mas river which include Gunungsari Dam, Wonokromo Canal, Wonokromo Sluice, Djagir Barrage, Gubeng Dam, Pegirian Gate?
If there are no such booklet, please let us know the history, whatever it may be short.

2) Is there any trouble caused by local rainfall in the urban area?
If there are some troubles, what are the kind and conditions of such troubles?

3) We find Gubeng Dam always damming up the river water.
What are the purpose of this dam?
Is there any hindrance against the urban drainage due to the damming?

4) Do you think that irrigation water which is taken from Gubeng Dam will decrease with the urban development?
If it is considered to have a decreasing trend, how do you think about the rough trend of decrease?

5) We hear that there are several drainage districts which rely on pumping stations and these pumps have been used for many years to discharge excess surface water. Is there any trouble with these pumping stations? If there are actually some troubles, what are the causes for them?

For instance:

a. The capacity of pumps themselves is too small, or they have some defects. We hear that the present capacity of existing pumps has been decreased to about 1/3 of the original one when those pumps were installed. How has it been confirmed?

b. Carring capacity of drainage canals is too small.

c. Conditions of ditches or conditions of flow into main drainage canals are bad.

d. There are some inflows from other outlets of drain to the river while the pumps are operated.

e. Others.

6) Considerably much river water is discharged through Djagir Barrage into Wonokromo canal at present in the rainy season. Why don't they make a part of the discharge flow into the Mas river and the Pegirian river as flushing water?

7) Cannot they make it flow into the Mas river and the Pegirian river because there might be some obstacles to it on the course of channel?

8) Some grasses are seen to grow around the entrance of Pegirian gates and this gives no evidance of flowing through the gates. Aren't these gates used at present?

9) Why does the downstream channel of Pegirian river change its direction far to the east?

10) How many gates are provided on the Pegirian river? What is the function of these gates and how are these gates operated?

11) What is the longitudinal slope of the Pegirian river? Are there on this channel any places where the river water is easily stagnated owing to certain changes of the longitudinal slope?

12) What is the extent of tidal effect on the Pegirian river?

13) What is the purpose of Ngemplak pumping station?
When and how is this station operated?
This station is considered as the one which provides the Pegirian river with flushing water for purification.
Cannot we make the Mas' water flow into the Pegirian by gravity?

APPENDIX C

MEMORANDUMS

MEMORANDUM No.1

DISCUSSION BETWEEN THE STUDY TEAM AND DPUT CONCERNING THE SURABAJA RIVER IMPROVEMENT PROJECT

I. The First Day.

Date: Thirsday, December 23, 1971, from 9.30 to 13.30
Place: Conference Room of DGWRD.
Attendants: Shown below

IR. J. Soedarjoko	DR. S. Sato
IR. Koesdarjono	MR. K. Soejima
IR. M. Gajo	DR. S. Nakagawa
IR. Sunarjo	MR. S. Ohira
IR. Aziz	MR. K. Ohno
DRS. Firman	MR. N. Jitsuhiro
IR. M. Majangkoro	MR. T. Nakaoka
DRS. Attamimi	
MR. Kasama	
MR. Konoike	
MR. Kitamura	
MR. Ilham B.E.	
DRS. Hudadi	
MR. Hendro	

1. Scope of work of the study team.

The study team proposed to DPUT a scope of work which was prepared by the team in accordance with the Terms of Reference for the Surabaja River Improvement Project. This proposed one is as follows:

SCOPE OF WORK PROPOSED BY THE JAPANESE FEASIBILITY STUDY TEAM FOR THE SURABAJA RIVER IMPROVEMENT PROJECT

1. Collection of data.

(1) Collection of data on hydrology, hydraulics, geology, and topography.

(2) Collection of data on economic statistics.

(3) Collection of data on construction equipment, materials, and execution of works.

2. Examine the present conditions of the Surabaja River including Wonokromo canal and the Kali Mas together with sluices, locks, and major gates.

3. Examine the present conditions of the major drainage canals together with gates and the major irrigation canals.

4. Examine the present conditions of the major pumping stations and their relevant canals.

5. Examine the present conditions of Morokrembangan Boezem and its relevant canals.

6. Allocation of design discharges for every river channel of the Kali Surabaja, Wonokromo canal, and the Kali Mas.

7. Preliminary design of river channels for flood control.

8. Study of whether to rehabilitate or improve major pumping stations. Preliminary design of them, if necessary.

9. Study of whether to rehabilitate or improve sluices, locks, and other structures. Preliminary design of them, if necessary.

10. Study of whether to rehabilitate or improve Morokrembangan Boezem and preliminary design, if necessary.

11. Study of other basic measures to facilitate drainage of Surabaja city and its hinterland, if necessary.

12. Rough estimation of construction costs.

13. Rough estimation of maintenance costs.

14. Survey of damages caused by flood and inundation.

15. Survey of water demand for domestic, agricultural, industrial, and maintenance use.

16. Estimation of benefits which result from flood control and facilitating drainage. Estimation of other benefits, if possible.

17. Benefit-cost analysis of the project.

After some discussion, the proposed one was agreed and confirmed between the two parties as follows:

(1) The rehabilitation of irrigation itself is not included the present scope of study in accordance with the terms of reference. But since the team also recognized its importance, the team shall advice in its report concerning the prospective aspect of rehabilitation. And the study team will transmit the intention of the government of Indonesia to the government of Japan if the former requests to the latter another study mission from Japan for irrigation rehabilitation problem in the area concerned to the present project.

(2) As for the sea-dike, it was decided that some discussion would be made between the both parties in Djakarta after the team finished their reconnaissance, concerning whether the improvement of the sea-dike would be included in the scope of works in the present feasibility study or not.

(3) It was understood that the operation cost would be included in "rough estimation of maintenance cost" in the scope of work proposed by the team.

2. Holidays in the working office in Surabaja were decided as follows:

Every Sunday
Dec. 25 : Christmas day
Dec. 31 : not a holiday but practically a holiday
Jan. 1 : New Year Day
Jan. 27 : national holiday
Feb. 16 : national holiday

3. Counterparts in Tokyo.

DPUT eagerly requested to the government of Japan to accept the Indonesian counterparts in Japan when the planning work would be done in Tokyo.

The team replied that they would transmit the Indonesian intention to the Embassy of Japan so that this intention might be realized.

4. Survey map of Morokrembangan Boezem.

Dinas Pengairan Propinsi Djawa Timur disclosed that Morokrembangan Boezem have already been surveyed.

5. Discussion about the questions in general.

Questions in General, Appendix B, were read by the team and the following replies were made by DPUT concerning each question shown in Appendix B.

(1) The reasons why water level is kept high by Wonokromo sluice and Djagir barrage are as follows:

a. To take water for purification plant,
b. To keep ground water table high for domestic wells and prevention of salt intrusion,
c. To keep the Surabaja area cool by evaporation.

(2) The reasons why water level is kept high by Gubeng sluice are as follows:

a. To keep ground water table high for domestic wells and prevention of salt intrusion,
b. To keep the Surabaja area cool by evaporation,
c. To take water for irrigation.

(3) The locks of Wonokromo sluice and Gubeng sluice are not needed because there is no traffic of ship and raft nowadays, but the lock of Mlirip sluice is needed.

(4) After the team has finished his inspection on the Surabaja river and the Marmojo river, the two parties shall discuss whether the improvement works of the Marmojo river will be included in the present scope of study and the extent of the river to be improved shall be discussed at that time if the river has been decided to be included in the present study.

DPPDT disclosed that the surveying of the Marmojo river was going on and the data required for the planning would be given to the team during their stay in Indonesia.

(5) Gedeg sluice is often opened during the flood of the Brantas river for the purpose of reducing the flood discharge of the Brantas itself according to the water level of the Porong river.

(6) The Kedungsoro river is completely shut.

(7) It was disclosed that the municipality Surabaja has an idea of constructing a long storage pond inside the sea-dike, the purposes of which are as follows:

a. To supply fresh water to the city because the water taken from the Brantas river contains heavy silt and much cost is needed for purification,
b. Flood control,
c. To privide a recreation area by making a green belt outside of the pond with the spoil of dredging the pond,
d. To make cool by the evaporation from the surface of the pond.

Since this idea is a huge project, the two parties may discuss this idea as the case may be.

II. The Second Day.

Date: Friday Dec. 24, 1971, from 9.00 to 11.45 a.m.
Place: Conference room of Directorate of River and Swampy Area Development.
Attendants: Shown below

IR. J. Soedarjoko	DR. S. Sato
IR. Koesdarjono	MR. K. Soejima
IR. Majangkoro	DR. S. Nakagawa
IR. Gajo	MR. S. Ohira
MR. Ilham B.E.	MR. K. Ohno
DRS. Hudadi	MR. N. Jitsuhiro
MR. Konoike	
MR. Kasama	
MR. Kitamura	
MR. Aziz	

Questions in general.

(8) Gunungsari canal does not unite with the Lamong river, but has a syphon at its crossing. The canal is expected to have two functions as follows:

a. Irrigation canal,
b. Drainage canal in the rainy season. Therefore some facilities may be needed to diverge excess water to the Lamong river.

(9) The syphon of the Watudakon river still has effective function.

(10) Only the Pegirian river is managed by the municipality Surabaja. Other rivers are managed by Dinas Pengairan Propinsi Djawa Timur.

(11) Authorities to control the sluices and the barrage on the rivers described above are the same as in the above. That is, Pegirian sluice and Pegirian pumping station which has been already constructed are controled by the municipality Surabaja. Five other larger pumping station are also controled by the municipality. However, irrigation canals and main drainage canals are controled by Dinas Pengairan Propinsi Djawa Timur except the Pegirian river. The purposes of pumping stations and Pegirian sluice will be explained later at Surabaja.

(12) The government of Indonesia has AWR (Algemene Water Reglement) and PWR (Provincial Water Reglement), but these regulations are now not effective enough. So new regulation are under consideration. The team was requested to advise DPUT if the team had some opinion about these.

(13) Concerning the sewerage system, it was streassed by DPUT that the separate systems which separate domestic sewage and industrial waste water from storm water would be needed in principle. However, it was confirmed that the words "drainage and sewerage systems" described in the terms of reference for the Surabaja river improvement project do not contain sewage disposal but mainly mean drainage itself. Therefore, it was confirmed that the team had only to suggest the future aspect of the town drainage systems which are supposed to have a connection with the improvement works of the Surabaja river. And the study of the rehabilitation of main pumping stations is needed.

(14) The extent of "hinterland" described in the terms of reference has already been discussed on the first day.

7. Discussion about the additional questions in general.

(1) The counterparts decided to try to find at Surabaja any history of the improvement works of the Surabaja river and its relevant rivers.

(2) As for the levee of the Surabaja river and other relevant rivers, there are no experiences of inundation caused by overtopping of river water or break on the levee.
Before the war, the left side of Gunungsari canal had been an inundation area. But later, many houses were built in this area and levee was cut by inhabitants to make the storm water flow into Surabaja city. The area around Darmo pumping station and other areas are often inundated in the rainy season.

(3) As for the research papers on rainfall and runoff, a few data books were shown, and it was suggested that the Institute of Hydraulic Engineering, Bandung might have some.

(4) Some aero-photographs were lent to the team. The photographs other than the above are kept in Malang, which were to be asked by DRSAD for providing the team.

(5) Both the agricultural development plan formulated and the maps of drainage and irrigation systems for agricultural land located on the left side of the Surabaja river can be shown at Surabaja.

MEMORANDUM No.2

A meeting for discussion on the pending matters at the preveious meeting in Djakarta and the survey results obtained up to this time by the Japanese Survey Team was held from 9.00 a.m. to 3.30 p.m. on Feb. 24, 1972 in the meeting room of the Directorate of River and Swampy Area Development, DPUT. In the midst of this discussion, a meeting with Ir. Sujono, Director General of Water Resources Development was held in his room.

Attendants in the former meeting were as follows:

Ir. J. Soedarjoko	Dr. S. Sato
A. Ashari, BIE	Mr. K. Soejima
Ir. Koesdarjono	Dr. S. Nakagawa
Ir. Aziz	Mr. S. Ohira
Ir. M. Gajo	Mr. M. Matsui
Mr. M. Konoike	
Mr. K. Kasama	
Mr. Kitamura	
Mr. Tsukamoto	

As a result of the discussion, some of the following were agreed or others were left pending until next meeting.

1. After the team made report to the Directorate of Water Resources Development on the results of his field survey, it was agreed by the two parties that the study of improvement of the Marmojo river shall be included in the scope of work of the present study. It was also agreed by the two that the team will cooperate with the Brantas River Survey Team in studying the flood discharge distribution of the Brantas river to the Surabaja river, within the frame work of "hinterland" which is described in the Terms of Reference.

2. The survey results up to this time were reported on the Surabaja river and Wonokromo canal including important facilities such as dams and sluice. These matters shall be included in the scope of work of the present study.

3. The team disclosed his opinion that the Mas river should be dealt with at the following stage of drainage and sewerage systems of Surabaja city, because the team judged that the flood water of the Surabaja river should not be poured into the Mas river through Wonokromo sluice and the Mas river should be taken into consideration as a main drainage canal within the drainage and sewerage systems of the city. There was no objection against this opinion from the DPUT side.

4. The team reported that the inundations in some parts of the urban area could not be diminished unless the secondary and tertiary channels of drainage were rehabilitated or improved, however the pumping stations were strengthened in their capacity. There is an essential unbalance between them. So the team disclosed his opinion that the rehabilitation or improvement of the pumping stations should be inclusively dealt with at the following stage of study of drainage and sewerage systems of Surabaja city. There was no objection against this opinion.

5. The team reported that the carrying capacity of Gunungsari canal is less than 3.5 m^3/s, so that it is essentially difficult that the canal

accepts the flood runoff from Gunungsari Hill. The team disclosed his opinion that the rehabilitation or improvement of Gunungsari canal should be inclusively dealt with at the following stage of study of drainage and sewerage systems and irrigation rehabilitation, because the disposal of the flood runoff from the hill should be separated from the irrigation system and, therefore, this problem can not be solved independently of an integrated study on the urban drainage and sewerage systems and the irrigation systems. There was no objection against this opinion.

6. Concerning the rehabilitation or improvement of the existing irrigation system, the team showed an idea that the intakes of Djeblokan canal and Kali Bokor might be moved to the upstream of Gunungsari dam when Gubeng dam was desired to be removed in order to lower the water level upstream of the dam for the purpose of improvement of drainage of the city and improvement of water quality for irrigation. Furthermore, the team disclosed his opinion that the problems of main drainage canals in farm land should be studied at the following stage of study on irrigation systems and drainage and sewerage systems of the urban area, because the main drainage canals in farm land cannot be dealt with independently of the drainage and swerage systems of the urban area since those canals also serve at present as drainage of the urban area and, further, the urban area is so energetically expanding to the farm land year by year that the farm land is expected by the Team Master Plan of Surabaja city to vanish by the year 1990. There was no objection against this opinion.

7. The team suggested the benefits of lowering the normal water level upstream of Djagir dam and that this idea is desired to be studied at the following drainage and sewerage systems and irrigation systems.

8. The Directorate of River and Swampy Area Development and Dinas Pengairan of East Java insisted that the sea-dike was originally built in order to protect the city from the menace of intrusion of the sea, while the present condition of sea-dike is critical owing to frequent overtopping of the sea and devastation of dike body itself, consequently the rehabilitation of sea-dike should be included in the scope of work of the present study. The team reserved his opinion until the next discussion with Director of Dinas Pengairan which will held in Surabaja on 26th of this month.

9. The above-mentioned two agencies also insisted that the rehabilitation of Morokrembangan Boezem should be included in the scope of work of the present study, but the team reserved his opinion until the next meeting mentioned above.

MEMORANDUM No.3

A meeting for discussion on the problems which have been left pending at the meeting held in Djakarta on Feb. 24, 1972 was held in the office room of the Director of Dinas Pengairan, East Java province from 9.00 a.m. to 0.30 p.m. on Feb. 26, 1972. The attendants were Mr. A. Ashari B.I.E., Dr. S. Sato and Mr. S. Ohira.

1. After discussion on the critical status of the existing sea-dike which contains also extremely devastated flap gates, the two parties have agreed to take in the rehabilitation problem of sea-dike in the scope of work of the present study from the viewpoint of prevention of the city

from the menace of intrusion of the sea.

2. Concerning the rehabilitation problem of Morokrembangan Boezem, the two parties have agreed to take in this problem, at least rehabilitation of Mitre gates at the mouth of the Boezem, in the scope of work of the present study from the viewpoint of the menace of intrusion of the sea.

3. It was confirmed that the locks of Gunungsari dam and Wonokromo sluice are not needed any longer.

APPENDIX D

MINUTES OF THE MEETINGS BETWEEN THE JAPANESE STUDY TEAM AND INDONESIAN COUNTERPARTS/ OFFICIALS ON THE SURABAJA RIVER IMPROVEMENT PROJECT

The Japanese study team shown in ANNEX No.1 and the Indonesian counterparts/ officials shown in ANNEX No.2 had discussions on the draft final report on the Surabaja River Improvement Project on November 30 - December 5, 1972. Both the parties agreed that the following matters would be adopted in the final report.

1. Master Plan.

(1) The designation of the river system is shown in ANNEX No.3.

(2) At this stage, 50-year flood shall be adopted for the design of the Surabaja river.

(3) Twenty-year flood shall be adopted as the design discharge for the Marmojo river.

2. Construction.

(1) Engineers

The construction works shall be carried out under the Supervision of the Indonesian engineers who have good experiences in similar works to the proposed ones, especially the dam construction and dredging such as Gunungsari dam and Morokrembangan boezem.

The Indonesian engineers shall be assisted by consulting engineers.

(2) Organization and facilities

Organization and facilities required for the present project shall be mentioned in the final report.

(3) Equipment

Some of the special equipment which have been used in similar projects and available to the present project will be provided by DPUT.

The list of these equipment will be given later on and will be attached as ANNEX No.4. These equipment shall be in good working condition and delivered at the job sites.

Other special equipment needed to the project, which are not listed in ANNEX No.4 should be procured by the Project. Ordinary equipment required for construction works, such as concrete mixers etc., shall be prepared by the contractor.

(4) Gates

The construction of gates shall be carried out in cooperation with foreign experienced contractor.

With regard to the provision of gates, design and manufacturing shall be ordered to the foreign contractor. However, manufacturing of the gate body and installation of the gates shall be carried out by local manufacturer under the technical guidance of the said contractor.

(5) Construction schedule.

The period for the construction works shall be five and a half years including detail design. Tentative schedule is shown in ANNEX No.5.

3. Water Requirements.

The study of the water resources corresponding to the water demand of the present project area shall be made by Brantas River Basin Development Study Team of OTCA.

4. Engineering Services.

Engineering services for detail design shall be based on the surveying checked by the consulting engineers.

5. Benefit-Cost Analysis.

On the occasion of benefit-cost analysis, the five construction works such as the Surabaja-Wonokromo River Improvement Works, the Mas River Improvement Works, the Morokrembangan Boezem Improvement Works, the Sea Dike Improvement Works, and the Marmojo River Improvement Works shall be dealt with as an integrated one.

6. Recommendation.

The technical and economic feasibility for the areal improvement of urban drainage system and improvement of irrigation system should be studied as soon as possible after the completion of the present study. For this study about one year may be required.

After completion of the Surabaja River Improvement Project, an operation organization for water management and flood control should be established. The organization shall be studied at the final design stage of the present project.

Although 50-year flood was adopted as the design discharge of the Surabaja river, it may be necessary in the future to adopt more than 50-year flood, say 100-year flood, in accordance with the development of the basin and the municipality of Surabaja. On that occasion, at least the whole width between both dikes may be needed. Therefore, new construction on this area should not be permitted.

Jakarta, December 8, 1972.

Project Coordinator
of the Surabaja River
Improvement Project

Ir. Y. Sudaryoko

Leader
of the Japanese Feasibility
Study Team
for the Surabaja River
Improvement Project

Dr. S. Sato

ANNEX No.1

THE JAPANESE STUDY TEAM MEMBERS

WHO PARTICIPATED IN THE MEETINGS HELD ON NOV.30-DEC.5, 1972

Dr. S. Sato	- Leader
Mr. K. Soejima	- Adviser
Mr. S. Ohira	- Expert
Mr. M. Matsui	- "
Mr. J. Okazaki	- "
Mr. K. Ohno	- "
Mr. S. Sata	- "

ANNEX No.2

THE INDONESIAN COUNTERPARTS/ OFFICIALS
WHO PARTICIPATED IN THE MEETINGS

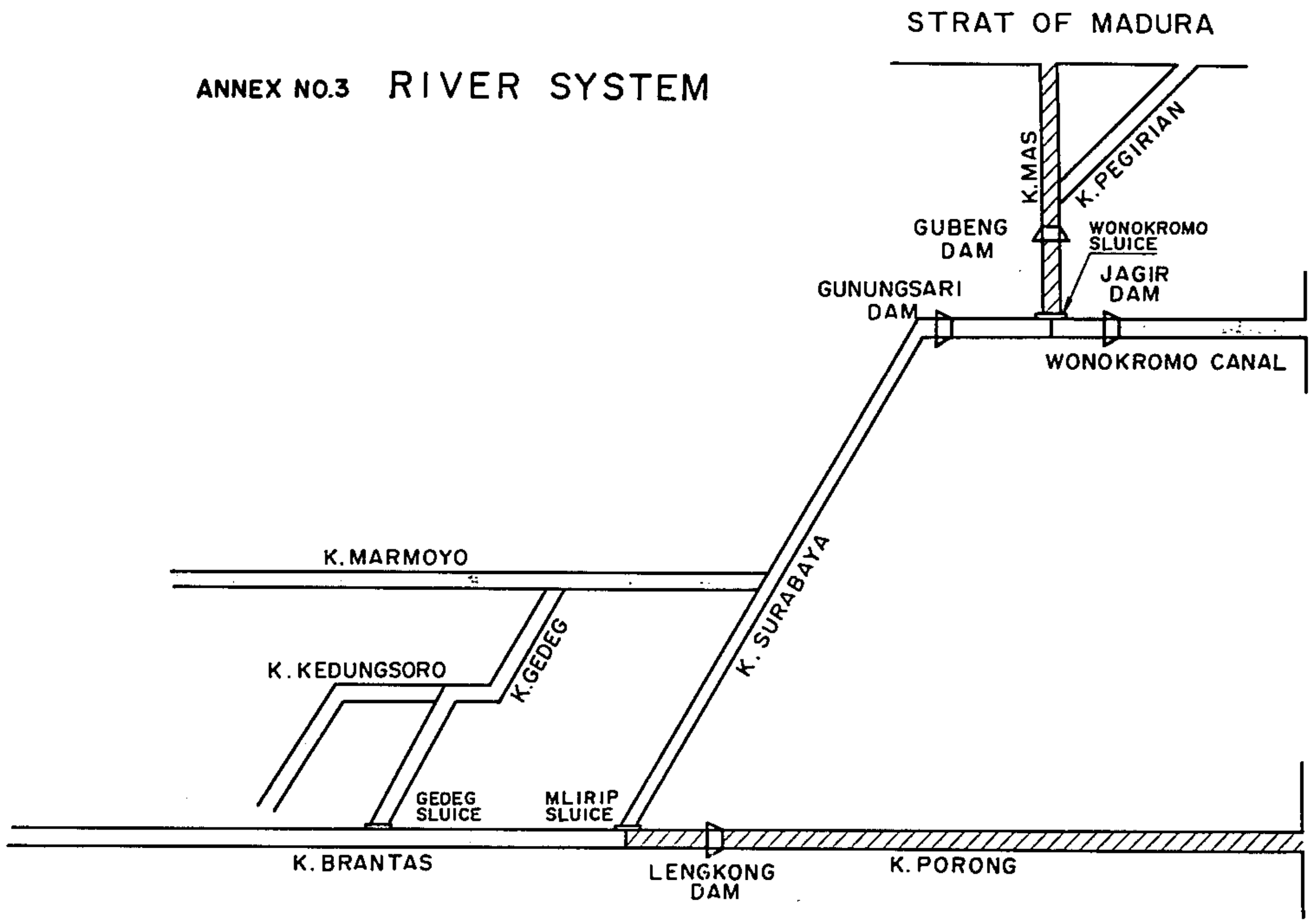
1. The counterparts who participated in the discussion on November 30 - December 4, 1972.

Mr. Y. Sudaryoko
Mr. A. Ashari
Mr. Kusdaryono
Mr. M. Mayangkoro
Mr. M. Y. Gayo
Mr. Sunaryo
Mr. Firman Sulaiman
Mr. Parmo
Mr. Suraji
Mr. Kuwat
Mr. Sukisno
Mr. A. Nuch
Mr. M. Husni
Mr. Prastowo
Mr. Sudiman
Mr. Rijuan
Mr. K. Kasama

2. The officials who participated in the meeting held on December 5, 1972.

Mr. Suyono Sosrodarsono	- Director General of WRD
Mr. Y. Sudaryoko	- Director of River and Swampy Area Development
Mr. Kusdaryono	- Chief of River Service
Mr. M. Y. Gayo	- Chief of River Section
Mr. K. Kasama	- Colombo Plan Expert

ANNEX NO.3 RIVER SYSTEM

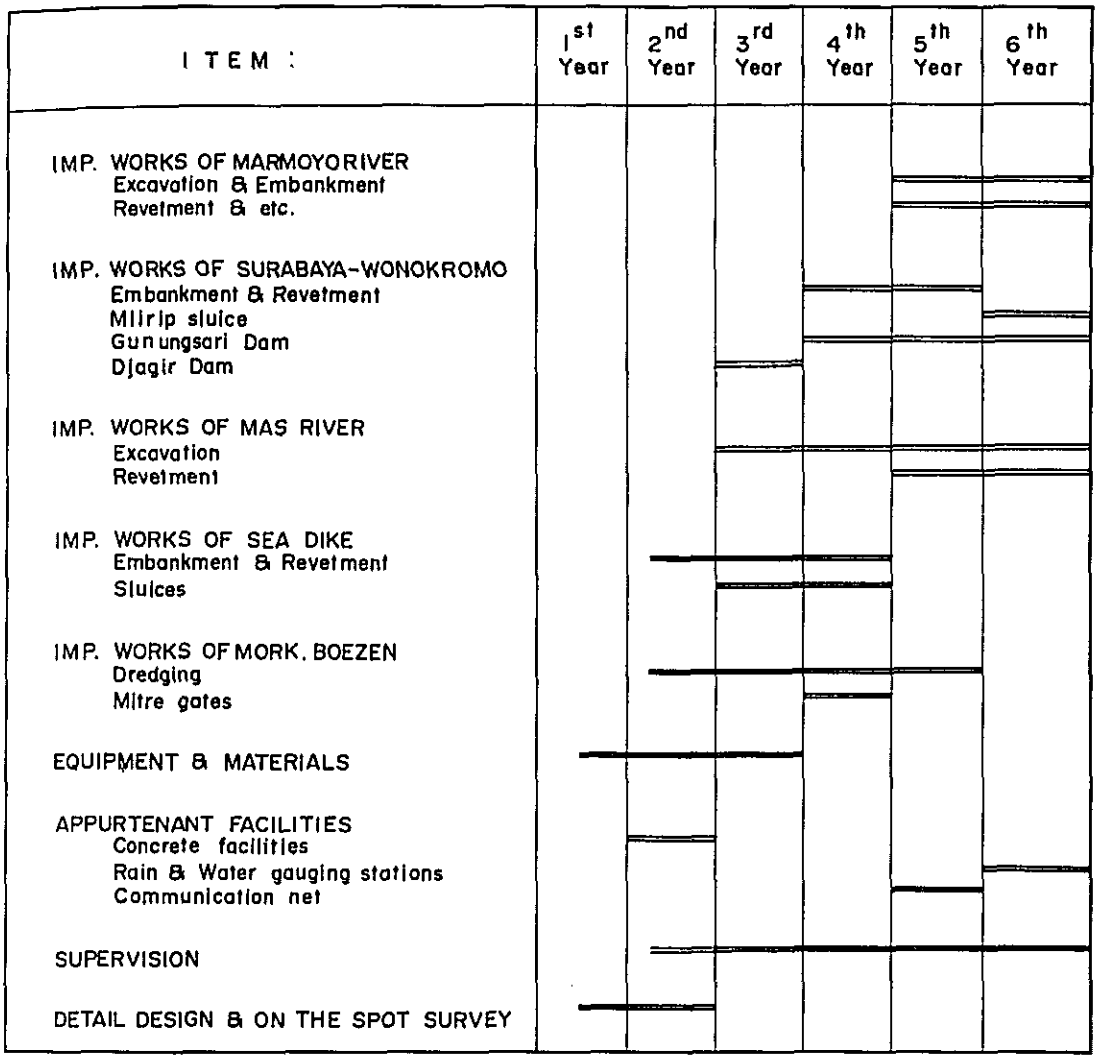


ANNEX No.4 LIST OF EQUIPMENT TO BE PROVIDED
FOR THE PRESENT PROJECT

No.	ITEM	TYPE	NUMBER:	USED		Residual Value(%)	NOTES
				START	FINISH		
1.	DREDGER	ELLICOT	2 UNITS	-	-	-	Prepared by DPUT
2.	SAND - PUMP	RDP-100B	1 UNIT	October 1972	November 1973	-	
3.	RADIO TRANSCEIVER	SSB-UHF	1 UNIT	-	-	-	
4.	CONCRETE VIBRATOR	HYOSHI EF - 38	6 UNITS	-	-	-	
5.	WHEEL LOADER	1,9M3 KIMCO-HOUGH J.465 C	2 UNITS	November 1972	January 1974	-	
6.	DUMP TRUCK	6TON ISUZU TXD 40D	21 UNITS	November 1972	January 1974	-	
7.	STAKE TRUCK	6TON ISUZU TXD 40D	5 UNITS	November 1972	January 1974	-	
8.	STAKE TRUCK CRANE	6TON W/3TON CRANE ISUZU TXD 40	2 UNITS	November 1972	January 1974	-	
9.	MOBILE - WORKSHOP	STELL BODY TYPE ISUZU TXD 50	1 UNIT	-	January 1974	-	
10	FUEL TANKER	6000 L ISUZU TXD 40	1 UNIT	November 1972	January 1974	-	
11	LUBRICATING CAR	ISUZU TXD 40	1 UNIT	November 1972	January 1974	-	
12	DRAGLINE	0,8M3 CRAWLER TYPE	3 UNITS	August 1972	January 1974	-	
13	AIR COMPRESSOR W/ CONCRETE VIBRATOR	AIRMAN FDR 176	2 UNITS	November 1972	January 1974	-	
14	VIBRATION PILE DRIVER	YAMADA KIKAICHI V3	12 UNITS	September '72	November 1973	-	
15	VIBRATING ROLLER	SAKAI SV - 9603	3 UNITS	-	January 1974	-	
16	BULLDOZER SWAMPY-LAND	6TON KOMATSU D 30F-12	5 UNITS	August 1972	January 1974	-	
17	MOTOR GRADER	3,6 M KOMATSU D637-5H	1 UNIT	August 1972	January 1974	-	
18	CONCRETE MIXER	250 LT W/ BATCHING EQUIPMENT	2 UNITS	-	November 1973	-	
19	BELT CONVEYOR	WIDTH 350 MM PORTABLE TYPE	4 UNITS	-	January 1974	-	
20	INSPECTION CAR	CANVAS TOP	7 UNITS	-	-	-	
21	INSPECTION CAR	STATION WAGON	3 UNITS	-	-	-	
22	DIESEL GENERATOR	30 KVA	1 UNIT	-	-	-	

ANNEX NO.5

CONSTRUCTION SCHEDULE OF THE
SURABAYA RIVER IMPROVEMENT PROJECT



APPENDIX E

LETTER OF MR. K. SOEJIMA AND DR. S. NAKAGAWA

Djakarta, Jan. 17, 1972

Ir. J. Soedarjoko
Director of River and Swampy
Area Development

Dear Sir,

We have joined the Feasibility Study Team for the Surabaja River Improvement Project as the advisers to the team and participated in the study during our stay in Indonesia for about one month.

On this occasion when we are going to leave Djakarta, we wish to express our deep appreciation for your respectful consideration to us since we have arrived at the Republic of Indonesia on Dec. 21, 1971.

Hearing from you the explanation of the large organization of the counterparts composed of 27 persons headed by you to work with the Japanese study team, we have been deeply impressed with intense eagerness of the Government of Indonesia for the Surabaja river improvement project. At Surabaja our investigation was started without any delay and is going on according to the schedule prepared beforehand, under the support of heartfelt advice of Ir. Koesdarjono, warm consideration of Mr. Ashari and generous cooperation of Ir. Majangkoro and other counterparts. We believe that the perfect cooperation between the Japanese team led by Dr. Sato and the Indonesian team led by Ir. Majangkoro will make it possible to collect sufficient data required to make a "Feasibility Study of the Surabaja River Improvement Project".

It is the great pleasure for us to report these facts to the Government of Japan and the Overseas Technical Agency, Japan. And at the same time, we surely transmit to the above authorities your eager request that the government of Japan could agree the Indonesian counterparts to visit Japan when the planning work for the Surabaja river project will be done in Tokyo.

We have recognized that the Surabaja river which flows through Surabaja city, the second largest city in Indonesia, has a very important function but now is in the condition which needs urgent rehabilitation and improvement as the Government of Indonesia has already pointed out. However, as we know, we should never forget that "River is ceaselessly living." Artificial hasty change in a river would often make it behave violently which might lead to the unexpected result.

We believe it is the same with the Surabaja river. The improvement and maintenance of the Surabaja river had been made continuously for centuries except the hours of hardship after the war. Therefore, the rehabilitation and improvement works of the Surabaja river should be carried out step by step under the careful consideration of the historic data.

And, as we know, we have already another survey team for the Brantas river. Therefore, we think we have to make a closer contact with the team and make a necessary adjustment between the two teams on the occasion of making the plan for the Surabaja river improvement works.

Furthermore, as for the irrigation system around the Surabaja river, we think it desirable to make another study what the system should be in connection with the urban planning of Surabaja city.

The other day, we had an opportunity to visit the hydraulic laboratory in the Solo River Development Project accompanied by Dr. Sato at the request of you and the Embassy of Japan. On the way to the laboratory, we saw the people eagerly engaging in the river improvement works as well as in the agricultural cultivation. Then we have realized the importance of the river improvement works and the basin development concerned. We want to transmit our deep impression to the Government of Japan and the Overseas Technical Cooperation Agency, Japan when we have returned to Japan.

Ir. Koesdarjono, Mr. Gajo and Mr. Konoike came to the laboratory in spite of being busy in Djakarta and guided us by themselves to give us important information about the prospected function of the laboratory. We have realized the importance of the laboratory, which has made us to make up our minds to make every effort so that some Japanese experts for hydraulic model test may be sent to the project and to assist you in arranging the instruments required to conduct the hydraulic tests.

Although our stay in Indonesia has been rather short, we are deeply impressed with the diligence of Indonesian people as well as the enthusiasm of the younger government officials who are earnest in developing your country. We have been convinced that the future of your country must be glorious if the further effort goes on.

We thank you again for the kind acceptance since our arrival at Djakarta and hoping your good health and the prosperity of your country, we remain.

Yours faithfully

Ken Soejima

Dr. Shoichiro Nakagawa

Advisers to the Feasibility Study Team for the Surabaja River Improvement Project.

VOLUME II

STUDY AND DATA

PART 4

STUDY

CHAPTER I

MEASUREMENT OF DISCHARGES

1. Method of Measurement.

For the purpose of measurement of roughness and examination of discharge measurement by surface-floats, current-meter measurement was carried out according to the following way.

(1) Sounding of the cross section.

Sounding was made at intervals of 2 or 5 m on the width of the measurement cross section with bomboo rod graduated in centimeters. The sections of sounding were spaced nearly twice the ones of velocity measurement.

(2) Velocity measurement.

Velocities at points on each section were measured with on electrical current meter (screw-type meter) from a boat. Although it is desirable to take as many points for the measurement as possible in order to obtain an adequately accurate mean velocity on the section, it was also important to finish the measurement in a minimum time. Hence, the three-point method was taken to determine the mean velocity (v_m).

$$v_m = \frac{1}{4}(v_{0.2} + 2v_{0.6} + v_{0.8})$$

where $v_{0.2}$, $v_{0.6}$ and $v_{0.8}$ denote the velocities at 0.2, 0.6 and 0.8 of the depth respectively. The cross section was devided into several sections so that the interval between them was less than 10 meters, and the mean velocity of the section was defined to represent that of a segment-area which stretches over both sides of the section. Total discharge of the cross section was calculated as the sum of the discharge of the segment-areas.

(3) Measurement of surface slope.

Surface slope was measured at the site where Manning's coefficient of roughness was to be measured. Water level was surveyed with level at two cross sections which were selected at intervals of 100 meters respectively upstream and downstream of the cross section. Leveling was made for both going and returning, and the surface slope was taken on the average of going and returning if the results of them were close enough.

2. Results of Measurements.

Discharge measurements were made at the following sites which are shown in Fig. 1.

a. Mlirip and Perning on the Surabaja river.

b. Djetis on the Marmojo river.

c. Kedungberuk on the Wonokromo river.

d. Sindhunegara on the Mas river.

Results of discharge measurements at each site is shown in Table 1-1 to 1-6. In the Table 1-1 to 1-3 for Mlirip and Perning station are also given the results obtained by DPPDT almost at the same time using surface-floats of 10-centimeter long banana stem.

Fig. I Location of Discharge Measurement Site and Suspended Load and Bed Material Sampling Site

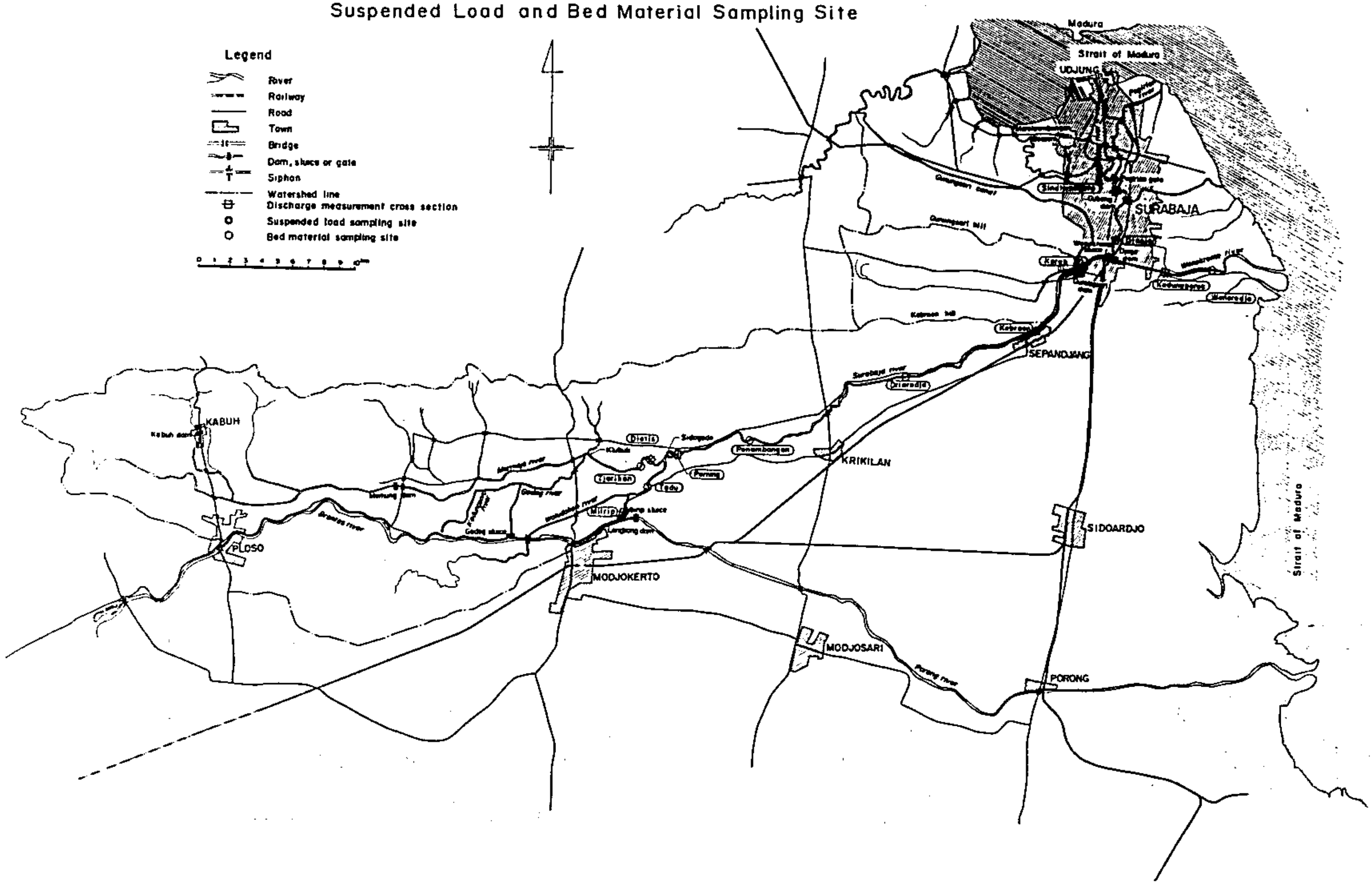


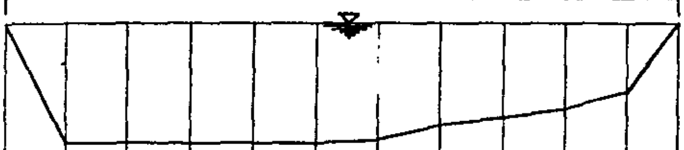
Table 1-1

MEASUREMENT OF DISCHARGE

River	The Surabaja river	Place	Mlirip
Location	about 550m downstream of Mlirip sluice (lower line of discharge measurement site of DPPDT)		
Date	Jan. 24 -72	Time	10:00 ~ 12:00

Lateral profile ;

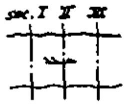
54.0 m



Sounding	Distance (m)		5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	4.0
	Depth (m)	0.0	1.97	1.96	1.99	1.97	1.98	1.90	1.67	1.55	1.40	1.10	0.0
Area (m²)	① ai		9.84	14.78	19.78	18.63	15.43	8.45					
	② Total		86.90 m²										
Velocity measurement	Distance (m)		5.0	5.0	10.0	10.0	10.0	10.0	4.0				
Velocity (m/s)	$V_{0.2}$		1.25	1.30	1.25	1.15	1.15	1.35					
	$V_{0.6}$		1.15	1.30	1.05	1.05	1.06	1.30					
	$V_{0.8}$		0.95	1.08	0.85	0.95	0.98	1.25					
	③ V_{mi}		1.13	1.25	1.05	1.05	1.06	1.30					
Discharge (m³/s)	q_i = ①×③		11.12	18.48	20.77	19.56	16.36	10.99					
	④ Total		97.28 m³/s										
Wetted perimeter (m)	P_i		5.38	5.0	5.0	5.0	5.0	5.0	5.0	5.0'	5.0	5.0	4.15
	Total		54.53 m										

Surface slope : $I = 0.128^m / 200^m = 1/1{,}562$ (by leveling)

Notes ; Result obtained by DPPDT (surface floats)

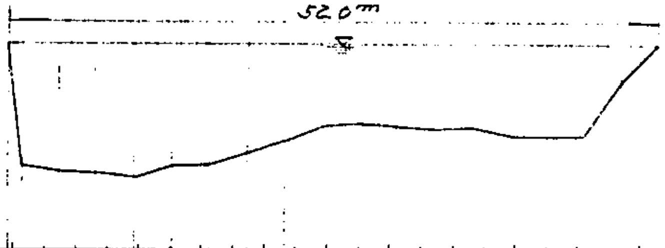


	Average area of sec. I, II, III	Average traveling time for 100 m	Mean velocity C = 0.8	Discharge
F_1	20.70 m²	119 sec.	0.67 m/s	13.91 m³/s
F_2	28.76	129	0.62	17.83
F_3	31.46	151	0.53	16.67
Total	80.92			48.41

MEASUREMENT OF DISCHARGE

River	The Surabaja river	Place	Mlirip
Location	about 500m downstream of Mlirip sluice (center line of discharge measurement site of DPPDT)		
Date	Feb. 11 -72	Time	9:40 ~ 11:45

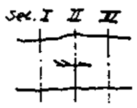
Lateral profile ;



Sounding	Distance (m)		1.0, 3.0, 3.0, 3.0, 3.0, 3.0, 3.0, 3.0, 3.0, 3.0, 3.0, 3.0, 3.0, 3.0, 3.0, 3.0, 3.0, 3.0
	Depth (m)		0.0, 2.02, 2.12, 2.12, 2.27, 2.02, 2.02, 1.82, 1.62, 1.36, 1.31, 1.36, 1.41, 1.36, 1.52, 1.52, 1.5[illegible], 0.61, 0.0
	Area (m²)	① a_i	21.76 | 15.15 | 12.04 | 10.10 | 10.67 | 12.09
		② Total	81.81 m^2

Velocity measurement	Distance (m)	7.0	7.5	7.5	7.5	7.5	7.5	7.5
Velocity (m/s)	$V_{0.2}$		0.88	1.00	0.92	0.95	1.05	1.06
	$V_{0.6}$		0.74	0.80	0.75	0.75	0.70	0.82
	$V_{0.8}$		0.60	0.70	0.56	0.60	0.50	0.65
	③ V_{mi}		0.74	0.83	0.75	0.76	0.74	0.74
Discharge (m^3/s)	q_i = ①×③		16.10	12.57	9.03	7.68	7.89	10.14
	④ Total		63.41 m^3/s					

Notes ; Result obtained by DPPDT (surface float)



	Average area of sec. I, II, III	Average traveling time for 100m	Mean velocity C=0.8	Discharge
F_1	15.56 m^2	125 sec.	0.64 m/s	9.96 m^3/s
F_2	20.18	126	0.64	12.81
F_3	26.29	129	0.63	16.25
Total	62.03			39.02

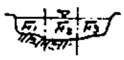
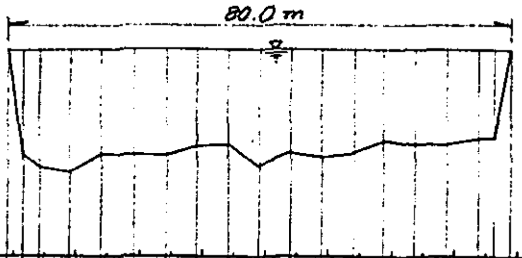


Table 1-3

MEASUREMENT OF DISCHARGE

River	The Surabaja river	Place	Perning
Location	about 100 m downstream of Perning bridge (center line of discharge measurement site of DPPDT)		
Date	Jan. 25 -'72	Time	10:00 ~ 12:0

Lateral profile ;



Sounding	Distance (m)		2.5, 2.5, 5.0, 5.0, 5.0, 5.0, 5.0, 5.0, 5.0, 5.0, 5.0, 5.0, 5.0, 5.0, 5.0, 5.0, 2.5, 2.5
	Depth (m)		0.0, 1.92, 1.97, 2.08, 1.72, 1.73, 1.76, 1.60, 1.56, 1.93, 1.70, 1.77, 1.88, 1.50, 1.58, 1.52, 1.46, 1.62, 0.0
	Area (m²)	① ai	16.50, 17.92, 17.14, 16.54, 17.76, 16.58, 15.45, 12.83
		② Total	130.72 m²
Velocity measurement	Distance (m)		5.0, 10.0, 10.0, 10.0, 10.0, 10.0, 10.0, 10.0, 5.0
	Velocity (m/s)	$V_{0.2}$	0.96, 1.13, 1.08, 1.05, 1.17, 1.04, 0.93, 0.98
		$V_{0.6}$	0.76, 0.97, 0.93, 0.85, 0.98, 0.85, 0.78, 0.90
		$V_{0.8}$	0.72, 0.78, 0.85, 0.80, 1.18, 0.78, 0.62, 0.73
		③ V_{mi}	0.80, 0.96, 0.95, 0.91, 1.08, 0.88, 0.78, 0.88
Discharge (m³/s)		q_i = ①×③	13.20, 17.20, 16.28, 15.05, 19.18, 14.59, 12.05, 11.29
		④ Total	118.84 m³/s
Wetted Perimeter (m)		P_i	3.02, 2.51, 5.00, 5.01, 5.00, 5.00, 5.00, 5.00, 5.02, 5.01, 5.00, 5.00, 5.00, 5.00, 5.00, 5.00, 2.50, 2.88
		Total	80.97 m

Surface slope : $I = 0.093^m / 119.25^m = 1/2,150$ (by leveling

Notes ; Result obtained by DPPDT (surface floats)

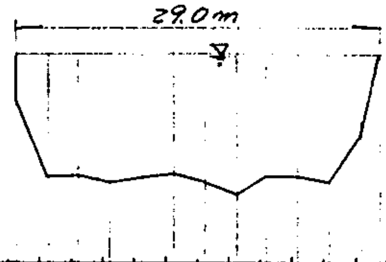
	Average area of sec. I, II, III	Average traveling time for 100 m	Mean velocity c × a B	Discharge
F_1	50.71 m²	96 sec.	0.83 m/s	42.09 m³/s
F_2	58.13	90	0.89	51.74
F_3	16.20	87	0.92	14.90
Total	125.04			108.73

Table 1-4

MEASUREMENT OF DISCHARGE

River	The Marmojo river	Place	Djetis
Location	under railway bridge at about 1.2 Km upstream of the confluence with the Surabaja river		
Date	Feb. 11 - '72	Time	13:00 ~ 15:00

Lateral profile ;



Sounding	Distance (m)		2.5 2.5 2.5 2.5 2.5 2.5 2.5 2.5 2.5 2.5 2.5 1.5
Sounding	Depth (m)		0.76 2.02 2.02 2.12 2.02 1.97 2.12 2.32 2.02 2.02 2.12 1.31 0.0
Sounding	Area (m²)	① ai	8.53 10.34 | 10.10 | 10.97 | 10.22 | 5.26
Sounding	Area (m²)	② Total	55.42 m²
Velocity measurement	Distance (m)		2.5 | 5.0 | 5.0 | 5.0 | 5.0 | 5.0 | 1.5
Velocity measurement	Velocity (m/s)	$V_{0.2}$	0.71 1.07 1.05 1.16 1.17 0.45
Velocity measurement	Velocity (m/s)	$V_{0.6}$	0.64 0.80 0.90 1.03 1.05 0.53
Velocity measurement	Velocity (m/s)	$V_{0.8}$	0.55 0.63 0.83 0.95 0.95 0.55
Velocity measurement	Velocity (m/s)	③ V_{mi}	0.64 0.82 0.92 1.04 1.06 0.52
Discharge (m^3/s)		q_i = ①×③	5.46 8.48 | 9.30 | 11.40 | 10.85 2.74
Discharge (m^3/s)		④ Total	48.23 m³/s
Wetted Perimeter (m)		P_i	3.56 2.50 2.50 2.50 2.50 2.50 2.51 2.52 2.50 2.50 2.63 1.98
Wetted Perimeter (m)		Total	30.70 m

Surface slope : $I = 0.046^{m} / 171.7^{m} = 1/3{,}730$ (by leveling)

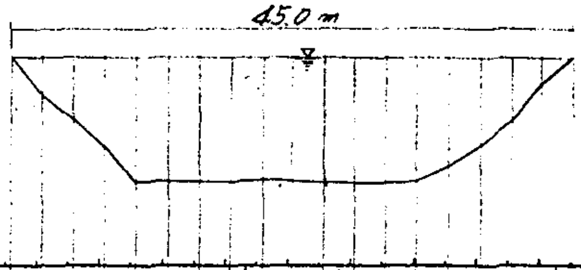
Notes ;

Table 1-5

MEASUREMENT OF DISCHARGE

River	The Wonokromo river	Place	Kedungberuk
Location	about 3.2 Km downstream of Djagir dam		
Date	Feb. 1 -'72	Time	11:00 ~ 12:00

Lateral profile ;



Sounding	Distance (m)	2.5 2.5 2.5 2.5 2.5 2.5 2.5 2.5 2.5 2.5 2.5 2.5 2.5 2.5 2.5 2.5 2.5 2.5
	Depth (m)	0.0 1.30 2.09 2.94 4.10 4.08 4.06 4.07 4.00 4.07 4.13 4.12 4.06 4.05 3.60 2.72 2.06 0.86 0.0
	Area (m²) ① ai	12.18 19.10 20.35 20.18 20.56 20.36 17.70 10.98
	Area (m²) ② Total	141.41 m
Velocity measurement	Distance (m)	5.0 5.0 5.0 5.0 5.0 5.0 5.0 5.0 5.0
	Velocity (m/s) $V_{0.2}$	0.14 0.15 0.20 0.32 0.32 0.40 0.28 0.17
	Velocity (m/s) $V_{0.6}$	0.07 0.07 0.19 0.30 0.30 0.30 0.20 0.15
	Velocity (m/s) $V_{0.8}$	0.05 0.00 0.16 0.20 0.27 0.27 0.18 0.15
	Velocity (m/s) ③ V_{mi}	0.08 0.08 0.19 0.28 0.30 0.32 0.22 0.16
Dis-charge (m³/s)	q_i = ①×③	0.97 1.53 3.87 5.65 6.17 6.52 3.89 1.76
	④ Total	30.36 m³/s
Wetted perimeter (m)	P_i	2.81 2.62 2.64 2.75 2.50 2.50 2.50 2.50 2.50 2.50 2.50 2.50 2.50 2.54 2.59 2.64 2.77 2.64
	Total	46.50 m

Surface slope : $I = 0.046^m/195.9^m = 1/4259$ (by leveling)

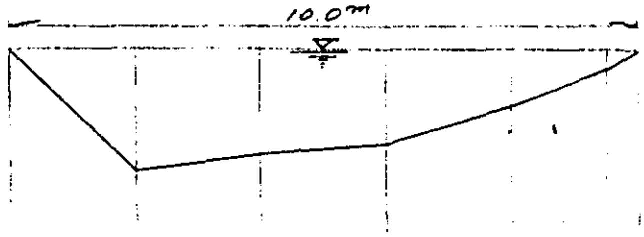
Notes ;

Table 1-6

MEASUREMENT OF DISCHARGE

River	The Mas river	Place	Sindhunegara bridge
Location	about 50 m downstream of Sindhunegara bridge		
Date	Jan. 31 - 72	Time	10:00 ~ 12:00

Lateral profile ;



Sounding	Distance (m)	2.0	2.0	2.0	2.0	1.5	0.5	
	Depth (m)	0.00	1.00	0.86	0.72	0.46	0.16	0.0
	Area (m²) ① a_i	1.00	1.86	1.64	1.24	0.50		
	② Total	6.24 m²						
Velocity measurement	Distance (m)	1.0	2.0	2.0	2.0	2.0	1.0	
Velocity (m/s)	$V_{0.2}$							
	$V_{0.6}$	0.30	0.35	0.12	0.25	0.25		
	$V_{0.8}$							
	③ V_{mi}	0.30	0.35	0.30*	0.25	0.25		
Discharge (m³/s)	q_i = ①×③	0.30	0.65	0.49	0.31	0.13		
	④ Total	1.88 m³/s						
Wetted perimeter (m)	P_i	2.24	2.00	2.00	2.02	2.02	0.53	
	Total	10.81 m						

Surface slope : $I = 0.026^m / 160.5^m = 1/6{,}173$

Notes ;

※ As $V_{0.6}$-value measured by currentmeter at 5.0 m from the left was extremely small, it was adjusted with the measurement by surface floats as follows.

Distance from the left	Traveling time for 50 m	Velocity (V_s)	$V_{0.6}/V_s$
1.0 m	121 sec.	0.413 m/s	0.73
3.0	107	0.467	0.75
5.0	117	0.427	—
7.0	137	0.365	0.69
9.0	135	0.370	0.68

$(V_{0.6}/V_s)_{mean} \doteqdot 0.7$

$V_{0.6}$ at 5.0 m $= 0.7 \times 0.427 = 0.30$ m/s

CHAPTER II

MEASUREMENT OF SUSPENDED LOAD AND BED MATERIAL

1. Suspended Load.

Suspended-load observation shall contain measurements of velocity distribution and water temperature as well as concentration of suspended load over the whole cross section for measurement. Sampling of suspended load were performed on three sections at the right, center and left part of the cross section with suspended load sampler shown in Fig. 1. The sampler makes it possible to take several samples of different depths at a time. Results of suspended-load sampling and its analysis are shown in Table-1 and Fig. 2 and result of discharge measurement in Table-2. We entrusted the analysis of samples to Fakultas Teknik Kimia, Institut Teknologi Surabaja (Faculty of Chemistry, Surabaja Institute of Technology) on the following terms:

a. Concentration of suspended solid (mg/l),

b. Concentration of suspended solid passing through the sieve of 0.074 mm in diameter (mg/l), and

c. Concentration of suspended solid remaining on the sieve of 0.074 mm in diameter (mg/l).

2. Bed Material.

Bed materials were sampled at the following sites of which locations are shown in Fig. 1 in Chapter I, Part 4.

a. Tadu, Perning, Penambangan, Driaredja, Kebraon and Karah of the Surabaja river,

b. Tjarikan of the Marmojo river,

c. Kedungberuk and Wonoredjo of the Wonokromo river, and

d. Dinojo and Sindhunegara of the Mas river.

At each site, samples were taken at three points of right, center and left of the channel. If two or three samples among these three showed similar appearance, they were mixed and made one sample for sieve analysis and specific gravity test.

The results of test are shown in Table 3-1 to 3-6. We entrusted the analysis of bed materials to Fakultas Teknik Sipil, Institut Teknologi Surabaja (Faculty of Civil Engineering, Surabaja Institute of Technology) on the following terms:

a. Sieve analysis and

b. Specific gravity test.

Fig. 1 Suspended Load Sampler

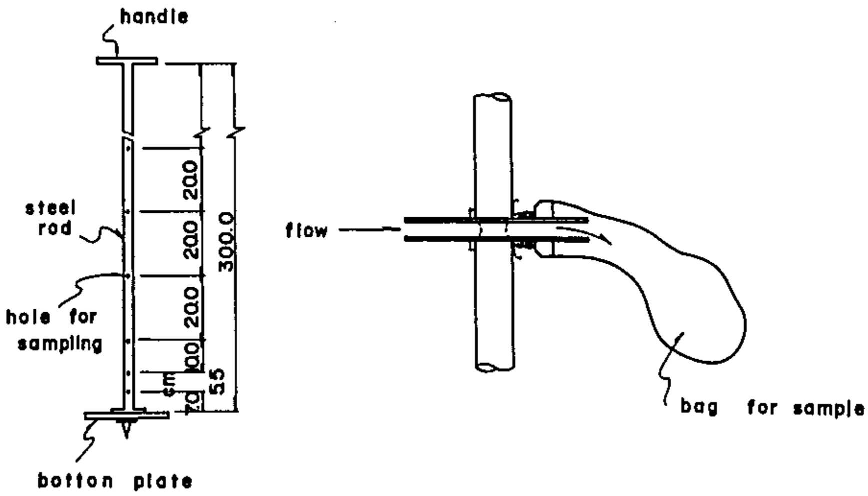


Fig. 2 Distribution of Concentration of Suspended Load

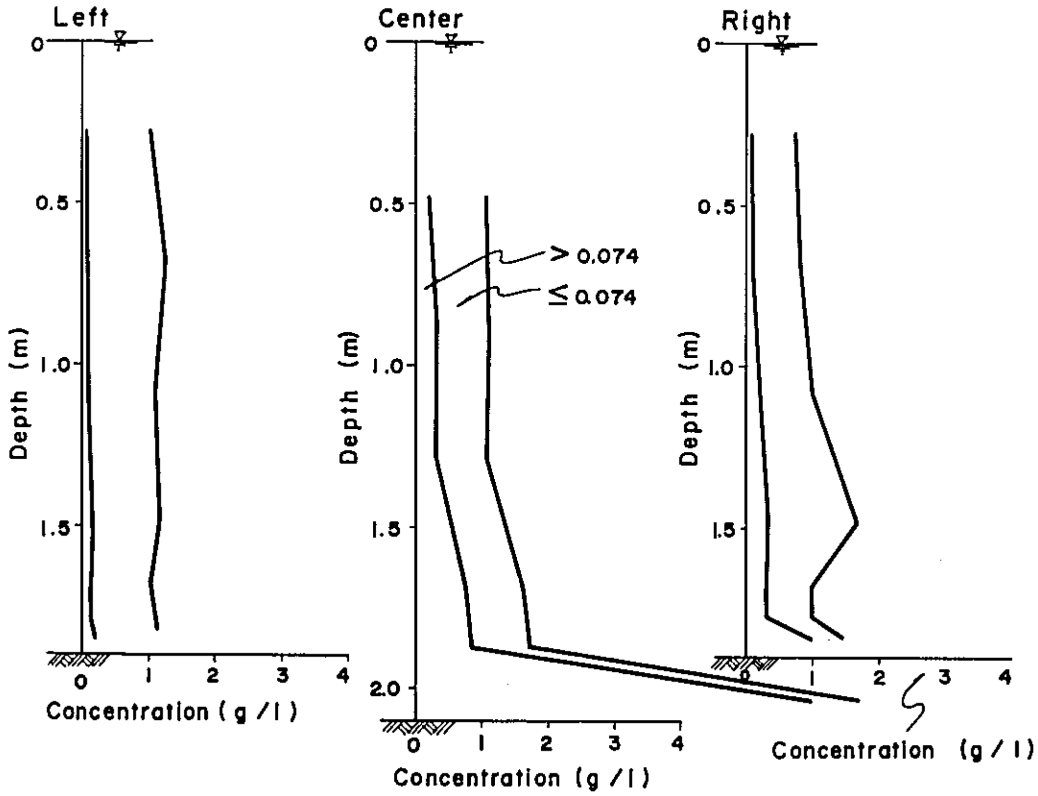
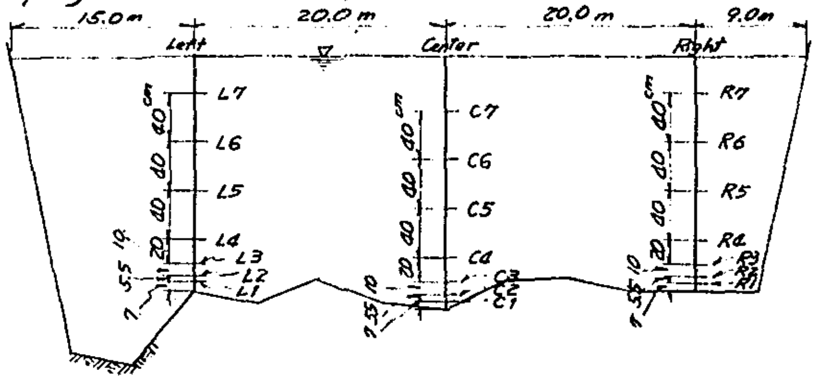


Table 1

SUSPENDED LOAD SAMPLING

River	The Surabaja river	Place	Perning
Location	about 150m upstream of Perning bridge		
Date	Feb. 9 - '72	Time	17:50

Sampling Position of the Suspended Load ;



Water Temperature ;

at the bottom at the center : 29.3 °C
at 30 cm from the surface at the center : 29.3 °C
at the bottom at Right-side : 29.3 °C

Result of Analysis on the Concentration of Suspended load ;

Sample			Concentration of suspended load (mg/ℓ)		
No	Code	Volume (ℓ)	On screen 0.074 mm	Through screen 0.074 mm	Total
1	L1	2.01	212	888	1100
2	L2	2.07	168	1001	1169
3	L3	2.24	142	898	1040
4	L4	2.28	157	1006	1163
5	L5	2.20	107	998	1105
6	L6	1.90	88	1190	1278
7	L7	2.26	72	986	1058
8	C1	1.90	5871	867	6738
9	C3	2.18	863	851	1714
10	C4	2.30	761	842	1603
11	C5	2.24	255	855	1110
12	C6	2.28	322	774	1096
13	C7	2.30	223	843	1066
14	R1	1.58	934	550	1484
15	R2	2.00	317	696	1013
16	R3	2.14	319	688	1007
17	R4	1.95	354	1352	1706
18	R5	2.20	235	766	1001
19	R6	2.08	97	737	834
20	R7	2.03	56	674	730

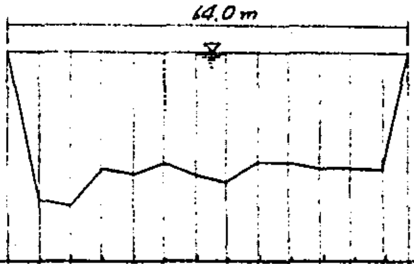
※ C2 was failed to sample

Table 2

MEASUREMENT OF DISCHARGE

River	The Surabaja river	Place	Perning
Location	about 150 m upstream of Perning bridge		
Date	Feb. 9 -'72	Time	15:00 ~ 18:00

Lateral profile ;



Sounding

Distance (m)	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	4.0	
Depth (m)	0.0	2.42	2.52	1.92	2.02	1.82	2.02	2.12	1.82	1.82	1.72	1.92	1.92	0.0

Area (m^2)							
① ai	18.42	20.94	19.20	20.20	18.44	14.40	8.44
② Total	120.24 m^2						

Velocity measurement

Distance (m)	5.0	10.0	10.0	10.0	10.0	10.0	5.0 / 4.0
Velocity (m/s) $V_{0.2}$	0.70	1.02	1.20	1.10	1.10	1.07	0.83
$V_{0.6}$	0.75	0.85	1.00	0.92	0.97	0.80	0.80
$V_{0.8}$	0.85	0.50	0.95	0.82	0.70	0.70	0.70
③ V_{mi}	0.76	0.80	1.04	0.94	0.94	0.84	0.78

Discharge (m^3/s)

q_i = ① × ③	14.00	16.74	19.99	19.00	17.33	12.10	6.74
④ Total	105.90 m^3/s						

Notes ;

Table 3-1

TEST OF BED MATERIAL

Name of river	The Harmojo river , The Surabaja river

Terms on Sampling Material

Date	Sampling site		Visual condition of material	Code of sample
Feb. 2-'72	Tjarikan on the Mormojo river, (about 3.0 km upstream of the confluence with the Surabaja river)	L	gray , fine uniform grain	Tjar.
		C	gray , fine uniform grain	
		R	gray , fine uniform grain	
Feb. 2'72	Tadu on the Surabaja river; (about 3.2 km downstream of Mlirip sluice)	L	blackish, uniform grain	Tadu
		C	blackish, containing coarse grains	
		R	blackish, containing white grains	

※ L, C, R represent left-side, center, right-side of the river

Result of Sieve Analysis and Specific Gravity Test

Code of sample	Percentage of weight passing through the seive												Weight of sample (g)	Specific gravity (t/m³)
	5 mm	4 mm	2 mm	1.41 mm	1.00 mm	0.42 mm	0.21 mm	0.105 mm	0.074 mm	0.063 mm	<0.063 mm	Loss		
Tjar.	100.00	99.61	97.15	94.81	92.75	89.62	47.71	5.28	0.76	0.73	0.20	0.24	630.5	2.587
Tadu	98.38	97.40	94.56	92.56	88.80	33.82	11.12	2.16	1.65	1.13	0.14	0.84	1077.0	2.497

Grading Curve

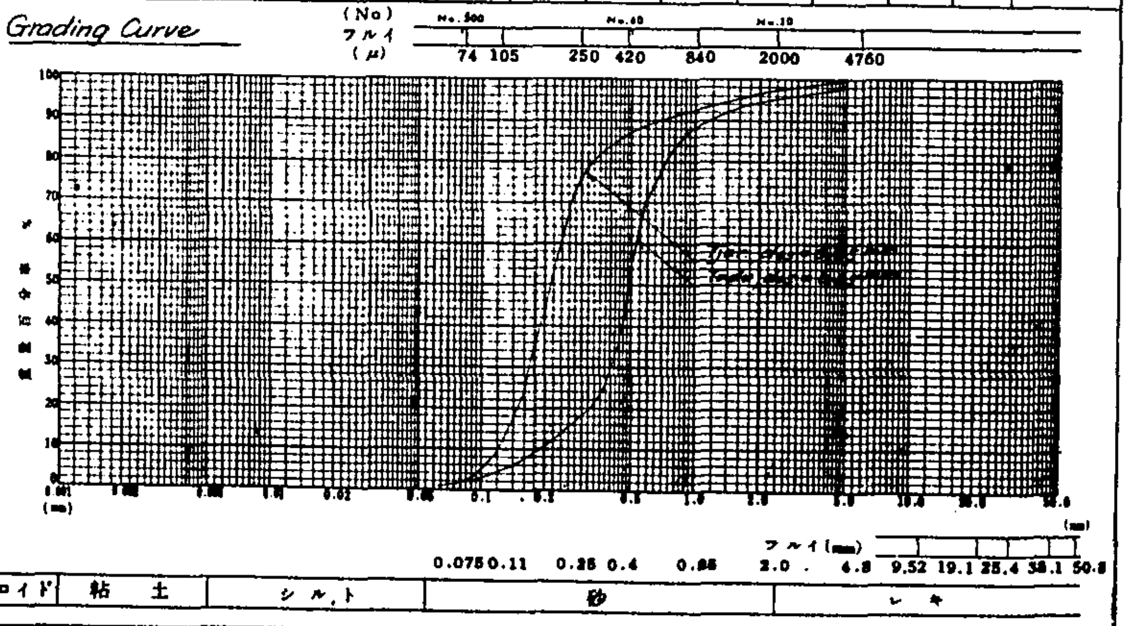


Table 3-2

TEST OF BED MATERIAL

Name of river	The Surabaja river

Terms on Sampling Material

Date	Sampling site		Visual condition of material	Code of sample
Jan. 25 -'72	Perning ; (about 100 m downstream of Perning bridge)	L	containing many coarse grains	Pern. (L)
		C	blackish, uniform grain	Pern. (CR)
		R	blackish, uniform grain	
Feb. 2 -'72	Penambangan ; (about 5.3 km downstream of Parning station)	L	containing many coarse grains and shell	Pena. (L)
		C	blackish, uniform grain	Pena. (CR)
		R	blackish, uniform grain	

※ L, C, R represent left-side, center, right-side of the river

Result of Sieve Analysis and Specific Gravity Test

Code of sample	Percentage of weight passing through the seive												Weight of sample (g)	Specific gravity (t/m³)
	5 mm	4 mm	2 mm	1.41 mm	1.00 mm	0.42 mm	0.21 mm	0.105 mm	0.074 mm	0.063 mm	<0.063 mm	Loss		
Pern. (L)	97.30	94.69	85.36	77.02	69.83	39.66	12.02	1.23	0.98	0.90	0.08	0.82	611.5	
Pern. (CR)	99.84	99.68	99.60	99.28	99.04	32.85	14.82	1.90	1.00	0.87	0.12	0	619.5	
Pena. (L)	93.32	89.08	78.16	68.05	58.30	16.97	5.34	1.37	1.01	0.95	0.05	0.72	554.0	
Pena (CR)	98.98	98.69	98.21	97.77	97.12	22.39	5.55	0.40	0.11	0.08	0.04	0.04	684.5	2.520

Grading Curve

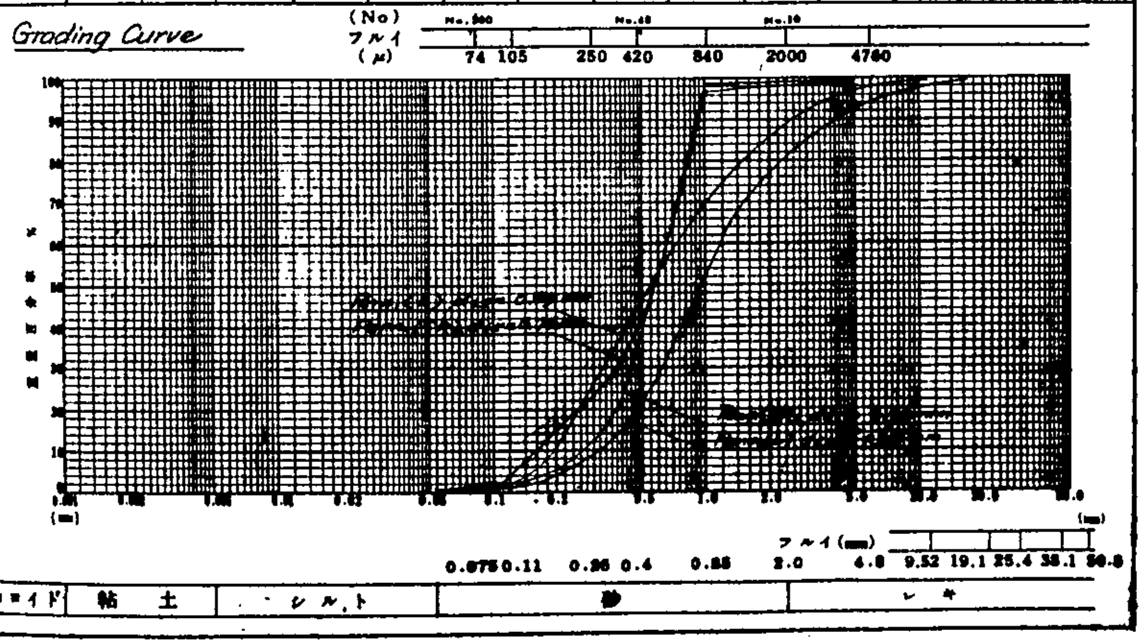


Table 3-3

TEST OF BED MATERIAL

Name of river	The Surabaja river

Terms on Sampling Material

Date	Sampling site		Visual condition of material	Code of sample
Feb. 2-'72	Driaredja ; (about 12.0 Km downstream of Penambangan site)	L	blackish, containing coarse grain	Dria.
		C	blackish, containing brown silt	
		R	blackish	
Feb. 2-'72	Kebraon ; (about 4.0 Km upstream of Gunungsari dam)	L	blackish, uniform grain	Kebr.
		C	blackish, containing fine black grains	
		R	blackish, uniform grain	

※ L, C, R represent left-side, center, right-side of the river

Result of Sieve Analysis and Specific Gravity Test

Code of sample	Percentage of weight passing through the seive												Weight of sample (g)	Specific gravity (t/m³)
	5 mm	4 mm	2 mm	1.41 mm	1.00 mm	0.42 mm	0.21 mm	0.105 mm	0.074 mm	0.063 mm	<0.063 mm	Loss		
Dria.	99.93	99.83	99.68	99.58	99.47	36.77	10.74	0.88	0.08	0.01	0.07	0.04	710.5	
Kebr.	99.96	99.87	99.78	99.74	99.69	66.29	17.90	1.97	0.15	0.06	0.09	0.13	574.5	2.647

Grading Curve

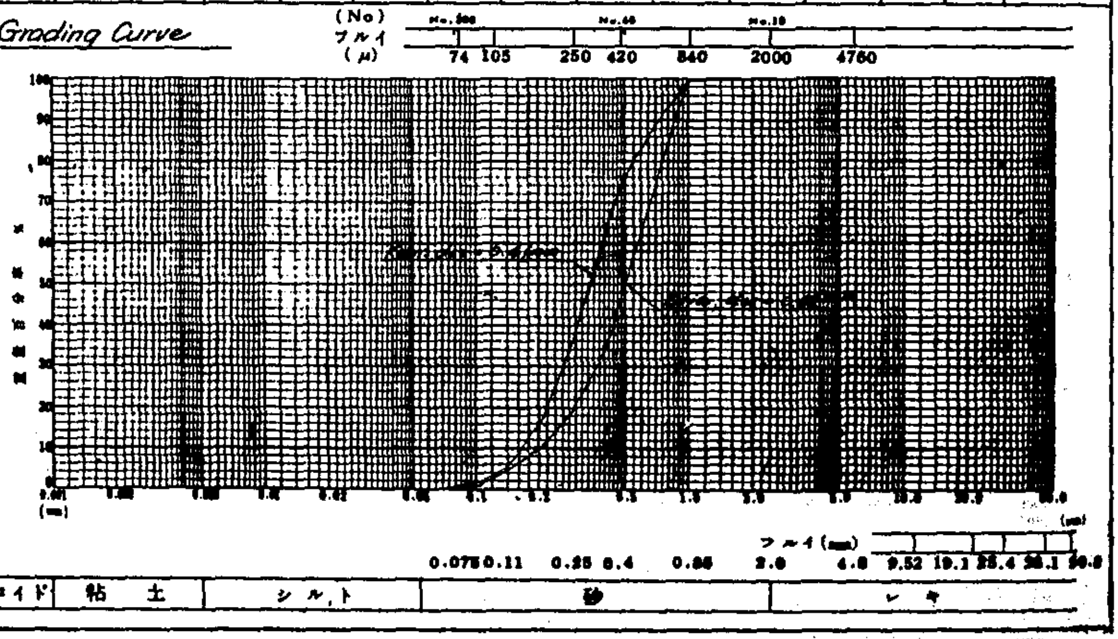


Table 3-4

TEST OF BED MATERIAL

Name of river	The Surabaja river

Terms on Sampling Material

Date	Sampling site		Visual condition of material	Code of sample
Mar. 3-'72	Karah (1) ; (about 850 m upstream of Gunungsari dam)	L	gray, containing broun silt	Kara.1(L)
		C	fine grain, containing brown silt	Kara.1(C)
		R	fine grain, containing brown silt	Kara.1(R)
Mar. 3-'72	Karah (2) ; (about 100 m upstream of Gunungsari dam)	L	gray, silty	Kara.2(L)
		C	blackish, fine grain	Kara.2(C)
		R	gray containing brown silt	Kara.2(R)

※ L, C, R represent left-side, center, right-side of the river

Result of Sieve Analysis and Specific Gravity Test

Code of sample	Percentage of weight passing through the seive												Weight of sample (g)	Specific gravity (t/m³)
	5 mm	4 mm	2 mm	1.41 mm	1.00 mm	0.42 mm	0.21 mm	0.105 mm	0.074 mm	0.063 mm	<0.063 mm	Loss		
Kara.1(L)	100.00	99.95	99.06	98.56	97.87	73.25	22.87	5.76	1.88	1.00	0.81	0.19	399.0	2.720
Kara.1(C)	100.00	100.00	100.00	100.00	99.96	99.65	31.30	5.07	2.83	2.70	0.45	0.45	558.5	2.720
Kara.1(R)	100.00	100.00	82.98	66.64	55.10	30.15	10.87	1.79	1.04	0.75	0.09	0.66	264.5	2.056
Kara.2(L)	99.83	99.75	99.37	99.24	98.99	57.11	16.49	2.54	0.78	0.65	0.34	0.29	594.5	2.816
Kara.2(C)	100.00	100.00	100.00	100.00	100.00	98.59	65.99	8.49	2.04	1.47	1.25	0.21	480.0	2.770
Kara.2(R)	100.00	100.00	100.00	100.00	100.00	99.34	56.15	12.04	3.17	1.85	1.32	0.53	378.0	2.426

Grading Curve

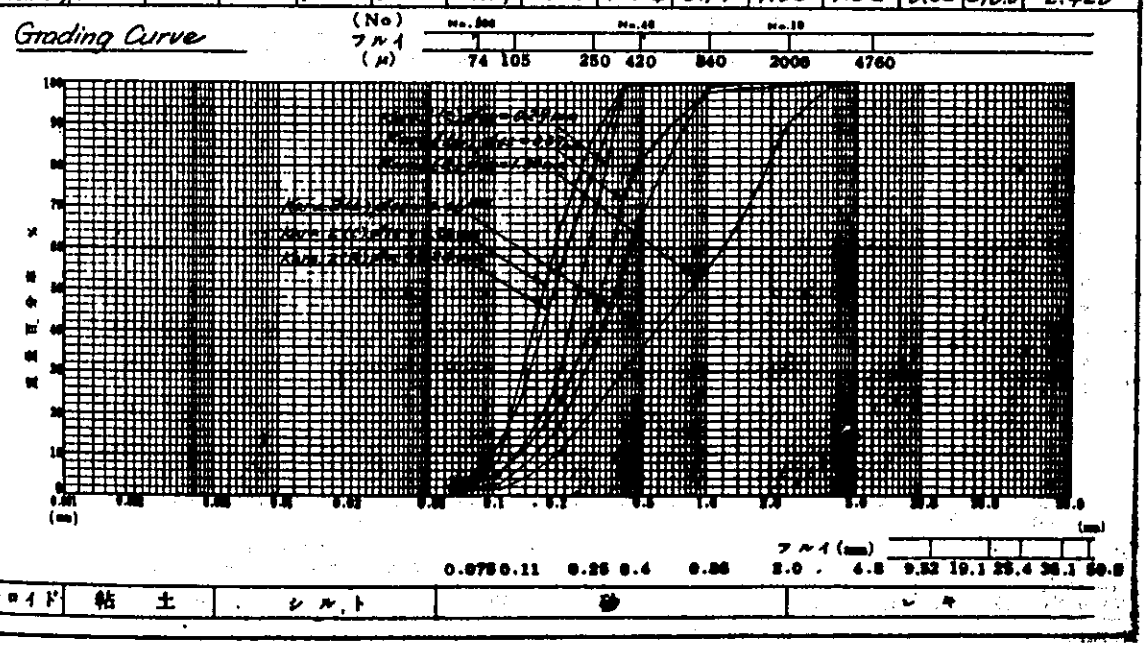


Table 3-5

TEST OF BED MATERIAL

Name of river	The Wonokromo river

Terms on Sampling Material

Date	Sampling site		Visual condition of material	Code of sample
Feb. 1-'72	Kedungberuk ; (about 3.2km downstream of Djagir dam)	L	gray	Kedu.(LR)
		C	black	Kedu.(C)
		R	gray	
Feb. 1-'72	Wonoredjo ; (about 6.5Km downstream of Djagir dam)	L	gray, containing many coarse grains	Wono.(L)
		C	blackish, uniform grain	Wono.(C)
		R	gray, fine grain	Wono.(R)

※ L, C, R represent left-side, center, right-side of the river

Result of Sieve Analysis and Specific Gravity Test

Code of sample	Percentage of weight passing through the seive: 5 mm	4 mm	2 mm	1.41 mm	1.00 mm	0.42 mm	0.21 mm	0.105 mm	0.074 mm	0.053 mm	<0.053 mm	Loss	Weight of sample (g)	Specific gravity (t/m³)
Kedu.(LR)	99.25	97.09	82.12	67.70	55.41	31.94	24.15	13.91	2.87	1.38	0.56	0.09	266.25	2.180
Kedu.(C)	99.58	99.15	98.20	97.27	96.31	60.43	20.20	2.01	0.59	0.38	0.11	0.32	235.5	
Wono.(L)	99.18	98.77	98.16	97.71	97.38	90.79	34.62	6.62	0.53	0.45	0.16	0.12	611.0	
Wono.(C)	99.96	99.91	99.91	99.87	99.82	99.69	98.05	12.21	0.40	0.31	0.18	0.13	565.0	2.573
Wono.(R)	99.79	99.50	98.24	96.45	95.45	94.16	92.66	14.31	1.24	0.24	0.79	0.07	350.0	

Grading Curve

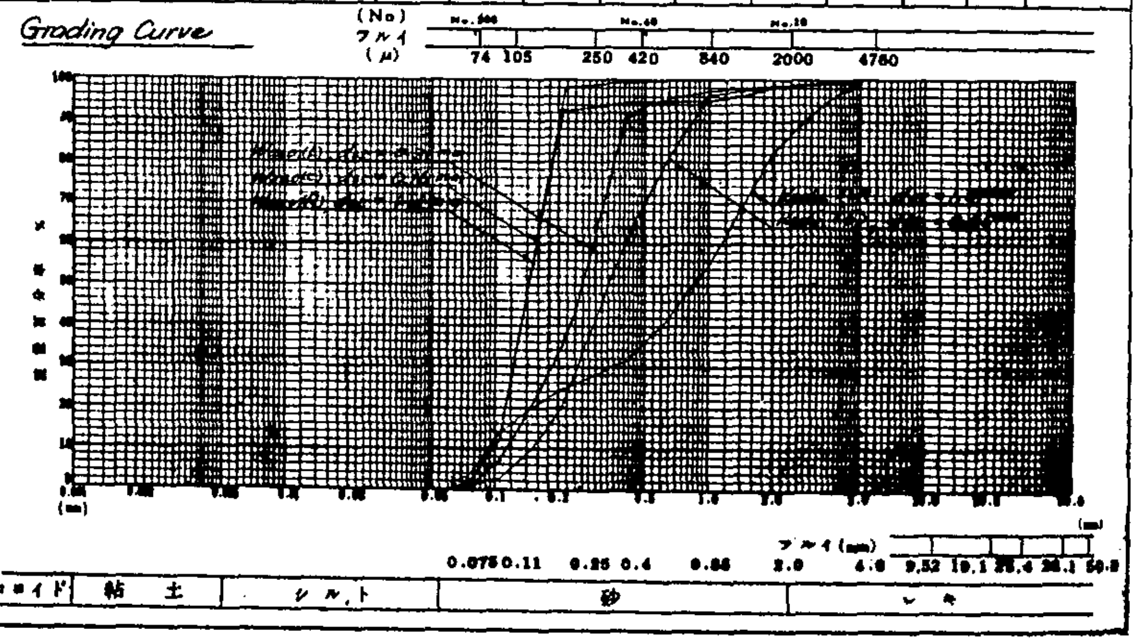


Table 3-6

TEST OF BED MATERIAL

Name of river	The Mas river

Terms on Sampling Material

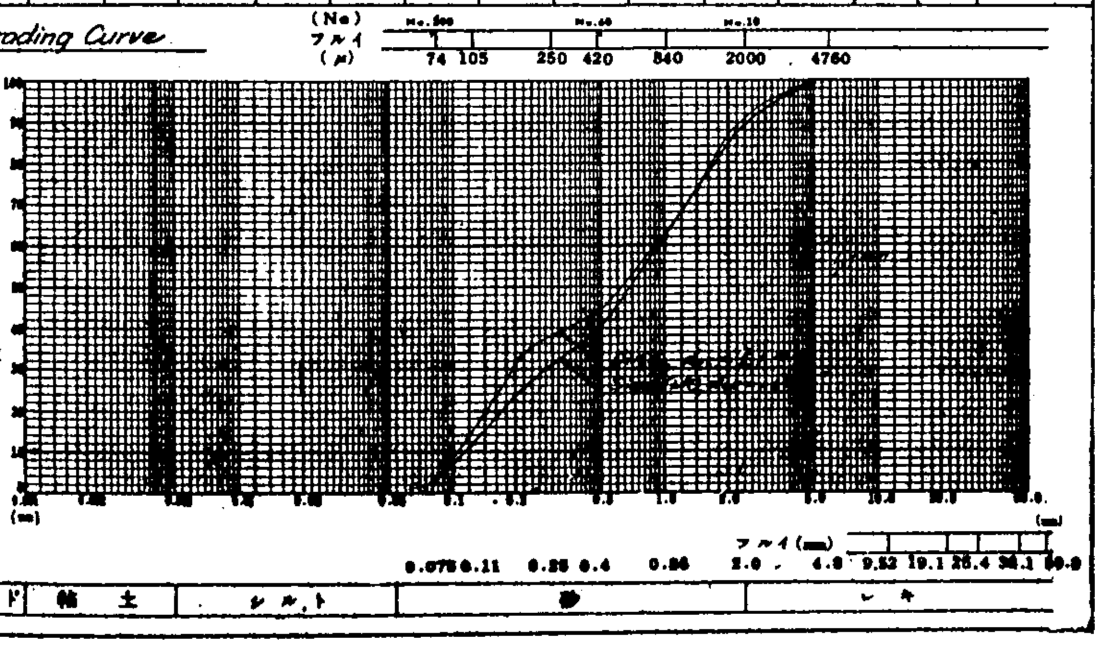
Date	Sampling site		Visual condition of material	Code of sample
Jan. 31-'72	Dinojo ; (about 1.2 km downstream of Wonokromo sluice)	L	blackish, silty	Dino.
		C	brown, silty	Dino.
		R	brown, silty	Dino.
Jan. 31-'72	Sindhunegara ; (about 50m downstream of Sindhunegara bridge)	L	gray, silty	Sind.(LR)
		C	gray, silty, containing many waste things	Sind.(c)
		R	gray, silty	Sind.(LR)

※ L, C, R represent left-side, center, right-side of the river

Result of Sieve Analysis and Specific Gravity Test

Code of sample	Percentage of weight passing through the seive												Weight of sample (g)	Specific gravity (t/m³)
	5 mm	4 mm	2 mm	1.41 mm	1.00 mm	0.42 mm	0.21 mm	0.105 mm	0.074 mm	0.053 mm	<0.053 mm	Loss		
Dino.	99.50	97.85	87.02	72.39	62.20	41.75	32.64	9.90	1.73	0.72	0.57	0.14	348.5	2.163
Sind(LR)	99.69	97.69	85.54	73.38	61.77	36.69	24.23	8.08	1.31	0.48	0.15	0.0.	325.0	
Sind.(c)	is not analyzed because of containing too many wastes such as paper, nylon, wood chips etc.													

Grading Curve



CHAPTER III

MANNING'S COEFFICIENT OF ROUGHNESS

1. Values Calculated from Discharge Measurement.

Manning's coefficients of roughness were calculated from the results of measurement using the following equation.

i) Equation;

$$n = \frac{A}{Q} R^{2/3} I^{1/2} \qquad (1)$$

where Q: discharge (m^3/s),
A: cross sectional area (m^2),
R: hydraulic mean depth (m) = A/P,
P: wetted perimeter (m),
I: surface slope.

ii) Calculation;

Table 1

Place		Mlirip	Pening	Djetis	Kedungberuk	Sindhunegara
River		Surabaja	Surabaja	Marmojo	Wonokromo	Mas
Measured	$Q(m^3/s)$	97.28	118.84	48.23	30.36	1.88
	$A(m^2)$	86.90	130.72	55.42	141.41	6.24
	P(m)	54.53	80.97	30.70	46.50	10.81
	I	1/1,562	1/2,150	1/3,730	1/4,259	1/6,173
$I^{1/2}$		1/39.52	1/46.37	1/61.07	1/65.25	1/78.57
$R=\frac{A}{P}$		1.594	1.614	1.805	3.041	0.577
$R^{2/3}$		1.365	1.376	1.482	2.099	0.693
n		0.031	0.032	0.028	0.150	0.029

Value of n calculated from the data at Kedungberuk is extremely large. This may be due to the fact that Kedungberuk is located in a reach affected by tide.

2. Values Calculated from Bed Materials.

According to logarithmic velocity distribution law based on the theory of turbulent flow and Manning's mean velocity formula, the relation between Manning's coefficient of roughness (n) and roughness height (k) is generally expressed by

$$n = \phi\left(\frac{R}{k}\right) k^{1/6} \qquad (2)$$

where $\phi(R/k)$ is a function of R/k,

Roughness height (k) of the river bed is generally represented by d_{65}, the grain size of which 65% of the bed material by weight is finer. Grain size (d_{65}) at a site was determined from the results of sieve analysis of the bed materials averaging samples except extreme one.

The values of $\phi(R/k)$ were calculated as shown in Table 2 by applying the measured values of n and d_{65} to the equation (2), where the grain sizes

Table 2

Place	Mlirip	Perning	Djetis	Sindhunegara
$d_{65}(\times10^{-3}m)$	0.55	0.75	0.25	1.10
$d_{65}^{1/6}$	0.286	0.301	0.251	0.321
n	0.031	0.032	0.028	0.029
$\phi(R/k)$	0.108	0.106	0.112	0.090

at Tadu and Tjarikan were used respectively in the calculation of n at Mlirip and Djetis, because the former sites are very near to the latter. The values of $\phi(R/k)$ seem to be constant except at Sindhunegara. Hence we adopted an average value of the first three. Then we get

$$n = 0.109\ d_{65}^{1/6} \qquad (3)$$

d_{65} = grain size of which 65% of the bed material by weight is finer, in m.

Using the equation (3), we calculated Manning's coefficients of roughness at Penambangan, Driaredja, Kebraon, Karah, and Wonoredjo where bed materials had been sampled and analysed. The results are shown in Table 3.

Table 3

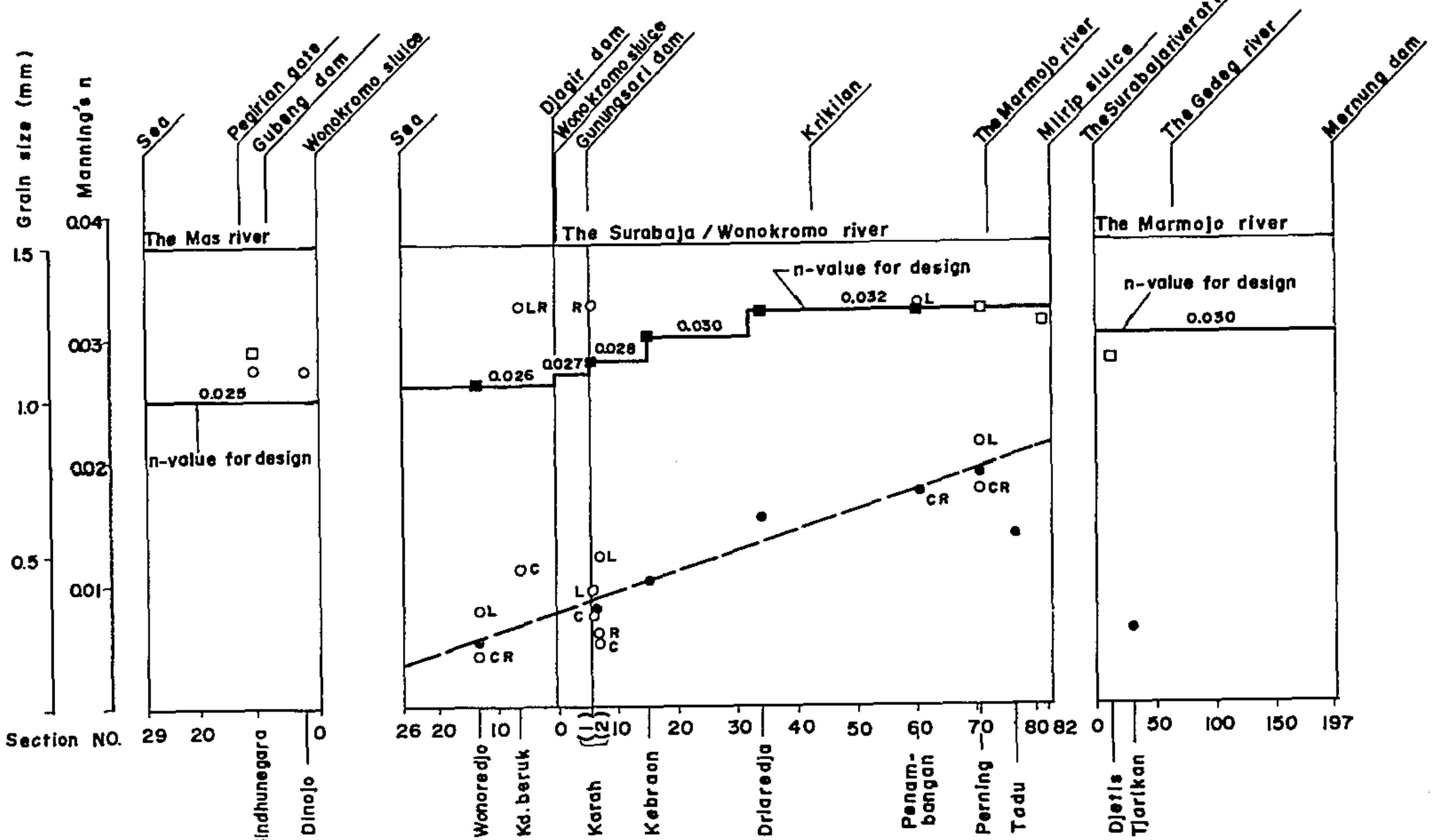
Place	$d_{65}(\times10^{-3}m)$	$d_{65}^{1/6}$	n
Penambangan	0.70	0.298	0.032
Driaredja	0.62	0.292	0.032
Kebraon	0.41	0.272	0.030
Karah	0.32	0.262	0.028
Wonoredjo	0.21	0.243	0.026

3. Values for Design.

Fig. 1 shows the distribution of grain size of the bed material and Manning's coefficient of roughness calculated in previous sections along the river course. Manning's n for design was laid down as shown in the following table considering the result of analysis and the condition of river channel.

Name of river	Reaches	n
Marmojo	Mernung dam - Sidogede	0.030
Surabaja/Wonokromo	Mlirip sluice - Section No.32	0.032
	Section No.32 - Section No.15	0.030
	Section No.15 - Gunungsari dam	0.028
	Gunungsari dam - Djagir dam	0.027
	Djagir dam - River mouth	0.026
Mas	Wonokromo sluice - River mouth	0.025

Fig. I Manning's Coefficient of Roughness for Design



Legend;
○ : Grain size (d_{65})
● : Grain size used for calculation of Manning's n
□ : Manning's n calculated from discharge measurement
■ : Manning's n calculated from bed material

CHAPTER IV

STAGE-DISCHARGE CURVES

1. Stage-Discharge Curves at Mlirip and Perning.

The stage-discharge curves, given in Fig. 1, at Mlirip and Perning stream-gaging stations were found at Daerah Modjokerto Office, though it is not clear when and by whom they were drawn. In this figure, the results of the discharge measurements which were conducted by us during the period of field survey are also plotted with black circles. This indicates that these curves hold good.

Next, we calculated the water stages of the Surabaja river for different discharges by using the equation of nonuniform flow and the relationships between water stages and discharges at Perning have been compared with the stage-discharge curve in Fig. 1, where calculated points are plotted with black triangles. This also indicates that the curve holds goods.

2. Stage-Discharge Curve at a Section Close Downstream of Gedeg Sluice.

(1) Discharge through Gedeg sluice.

The structure of Gedeg sluice is shown in Fig. 2. According to the records of operation, all of the stop logs were removed during the period from 18.00 on 27th of February, 1959 to 21.00 on 3rd of March, 1959. Since the method of operation of the stop logs has not been clarified with regard to the period except full open, the water-level records during the period of full open were used for the calculation of discharge through the sluice.

Since the flow in this case is taken as submerged one, the discharge equation shown below was used.

$$Q = CBH_2\sqrt{2g(H_1 - H_2)}$$

where C is a coefficient (assumed at 0.92 in this case), B in m is the total width of openings, g in m/s^2 is the acceleration of gravity, and H_1 and H_2 in m are as shown in Fig. 3. Calculation was carried out for the three cases shown in Table 1.

Table 1

Case	1	2	3
Date	18.00 Feb. 27	18.00 Mar. 3	12.00 Mar. 1
Water level upstream (m)	2.90	3.38	3.20
Water level downstream (m)	3.28	3.70	3.51
H_1 (m)	1.281	0.801	0.981
H_2 (m)	0.901	0.481	0.671
B (m)	27.54	27.54	27.54
$H_1 - H_2$ (m)	0.38	0.32	0.31
Q (m^3/s)	62.30	30.52	41.91

Fig. I Stage - Discharge Curves at Mlirip and Perning

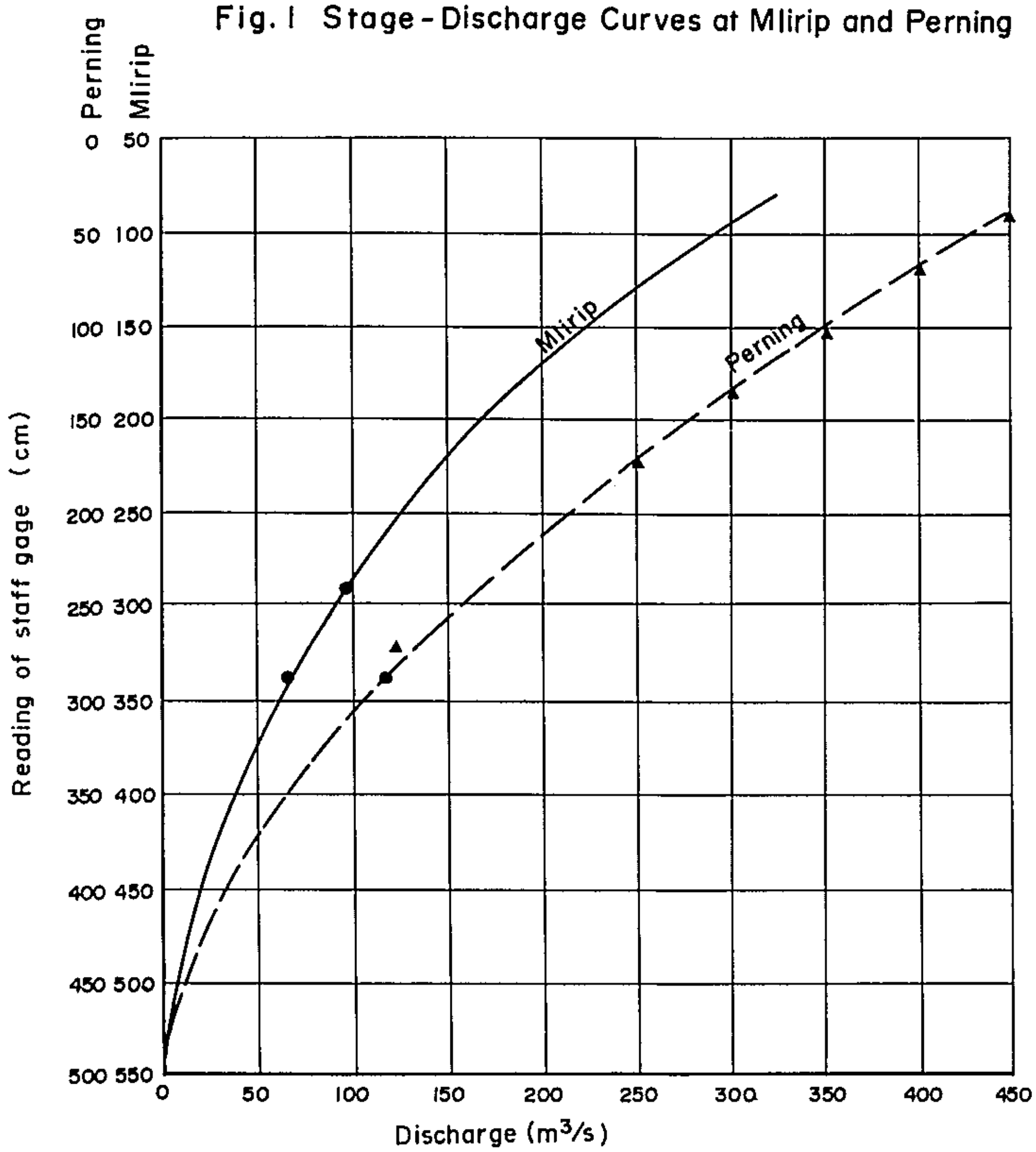


Fig.2 Gedeg Sluice

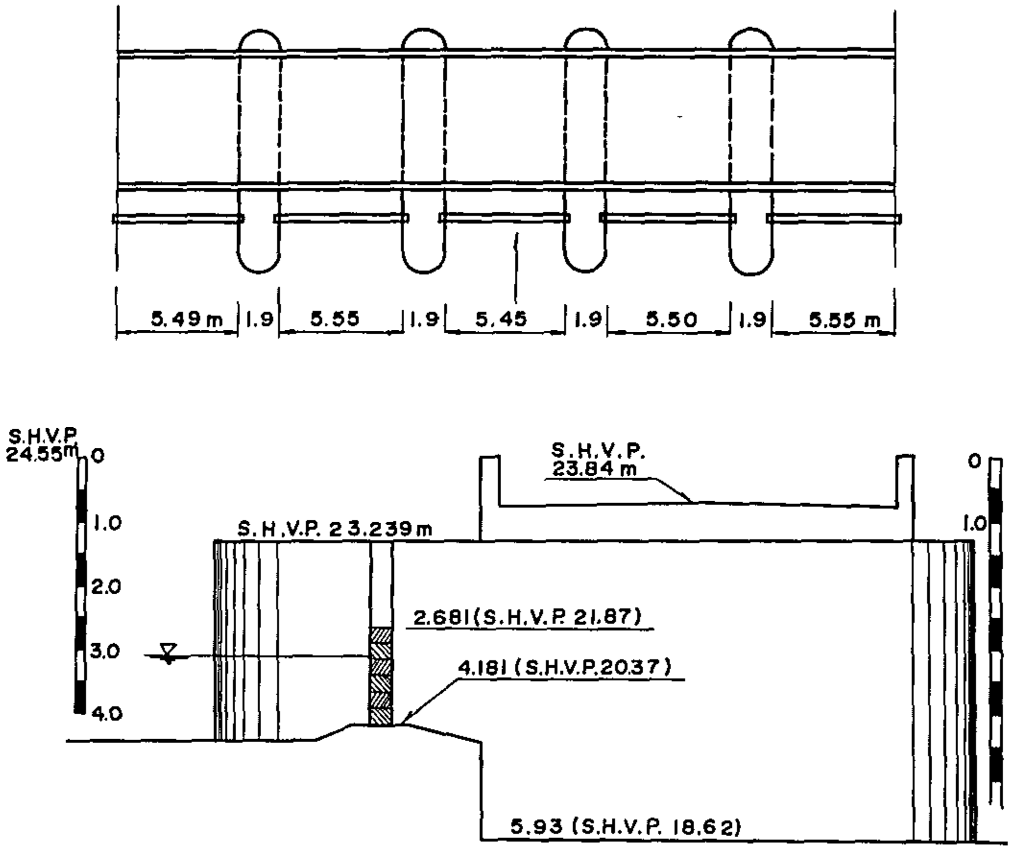
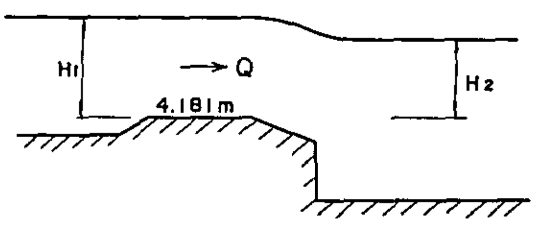


Fig.3



(2) Stage-discharge curve at a section close downstream of Gedeg sluice.

The relationship between water stage and discharge of the Gedeg river was obtained at a section shown in Fig. 4 which was made averaging the sections from No. 35 to No. 31 of the river. In this calculation, Manning's formula

$$Q = \frac{1}{n} AR^{2/3}I^{1/2}$$

was used, where n was assumed at 0.03 refering to the previous study on coefficient of roughness and the average slope I was assumed at 1/1,700 using the elevation difference, 2m, between the two dike crowns of section No. 36 (close downstream of Gedeg sluice) and section No. 55 (confluence of the Gedeg river and the Kedungsoro river) and the distance between them, 3,400 m. The results are shown in Table 2.

Table 2

Water stage of staff gage (m)	Water stage (m, SHVP)	Depth (m)	Water area A (m^2)	Mean depth R (m)	Q (m^3/s)
5.1	19.45	1.0	3.5	0.625	2.1
4.1	20.45	2.0	25.0	1.033	20.7
3.6	20.95	2.5	36.875	1.417	37.6
3.1	21.45	3.0	53.375	1.495	56.4
2.6	21.95	3.5	70.875	1.865	86.8
2.1	22.45	4.0	89.375	2.222	130.2

The above relationships between water stage and discharge are shown in Fig. 5 with circles altogether with the relationships obtained from discharge through the sluice with triangles.

Fig. 4

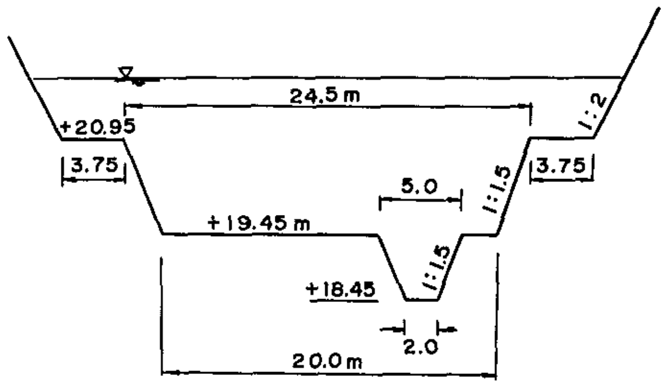
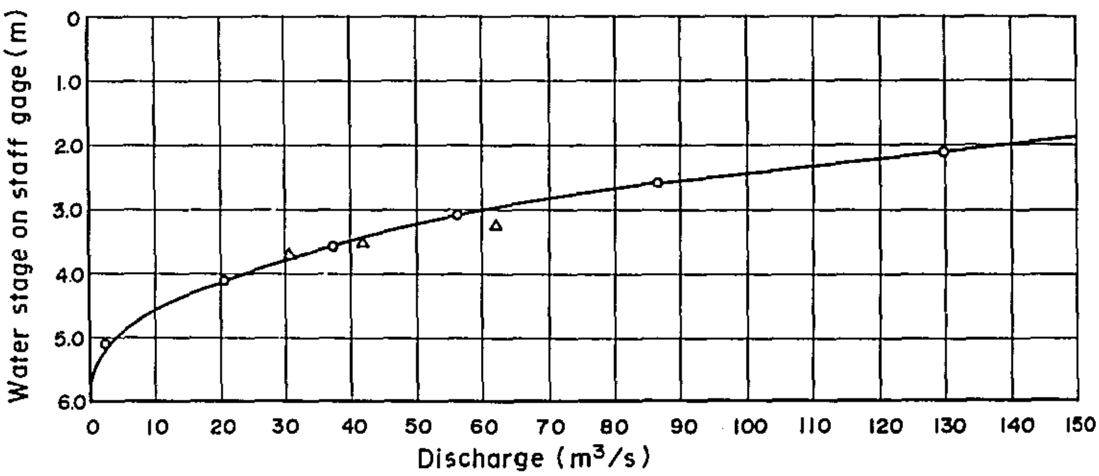


Fig. 5 Stage - Discharge Curve of the Gedeg River, Close Downstream of Gedeg Sluice



CHAPTER V

DISCHARGE CURVES OF DAMS ON THE SURABAJA/WONOKROMO RIVER

1. Kindsvader-Carter-Tracy's Method of Calculation for Flow through Constriction (see Fig. 1)

$$Q = CA_3\sqrt{2g(\Delta h - h_f + \alpha_1\frac{V_1^2}{2g})}$$

where

Q = discharge (m^3/sec),
C = over-all coefficient of discharge,
$= C'K_FK_rK_WK_\phi K_yK_xK_eK_tK_j$,
A_3= water area at section 3 (m^2),
g = acceleration of gravity (m/sec^2),
Δh= $h_1 - h_3$ (m),
h_f= frictional loss (m),
α_1= energy coefficient at section 1,
α = energy coefficient,
$= \frac{\int v^3 dA}{V^3 A} = 1.03 - 1.36$,
v = velocity of water passing through at point on a water area A,
V = mean velocity,
V_1= velocity at section 1,
C' = coefficient of discharge (standard value)
= a function of m and L/b,
L = length of the abutment in the direction of the thread of the stream,
m = contraction ratio $= 1 - K_b/K_B = 1 - b/B$ for a rectangular approach section of width B and a rectangular contracted section of width b,
K_B= conveyance of the uncontracted approach section 1 at normal discharge,
K_b= conveyance of the contracted section 3 which has the same normal depth and roughness characteristics as the approach section,
K = conveyance $= Q/\sqrt{S}$
$= AR^{2/3}/n$ when Manning formula is used,
S = slope,
R = hydraulic depth (m),
n = coefficient of roughness,
K_F= a function of Froude number ($Q/A_3\sqrt{gd_3}$),
d_3= water depth at section 3 (m),
K_r= coefficient of entrance rounding,
K_W= coefficient of wing walls or chamfers,
K_ϕ= coefficient of angularity of constriction,
K_y= coefficient of side depths at abutment,
K_x= coefficient of side slope of abutment,
K_e= coefficient of eccentricity of constriction,
K_t= coefficient of submergence of bridge,

K_j = coefficient of bridge piles and piers,
B = width of approach section (m),
b = width of contracted section (m),
$j = A_j/A_3$
A_j = area of piers (m^2),
$h_1^*/\Delta h$ = backwater ratio, a function of Manning's n and contraction ratio m,
k_a = adjustment factor for backwater ratio, a function of contraction ratio m and the ratio C/C_{basic},
C_{basic} = discharge coefficient for the basic type of a vertical-faced constriction with square abutments,
A_B, P_B, R_B = water area, wetted perimeter, and hydraulic depth at approach section,
A_b, P_b, R_b = water area, wetted perimeter, and hydraulic depth at contracted section.

Fig.1 Constriction

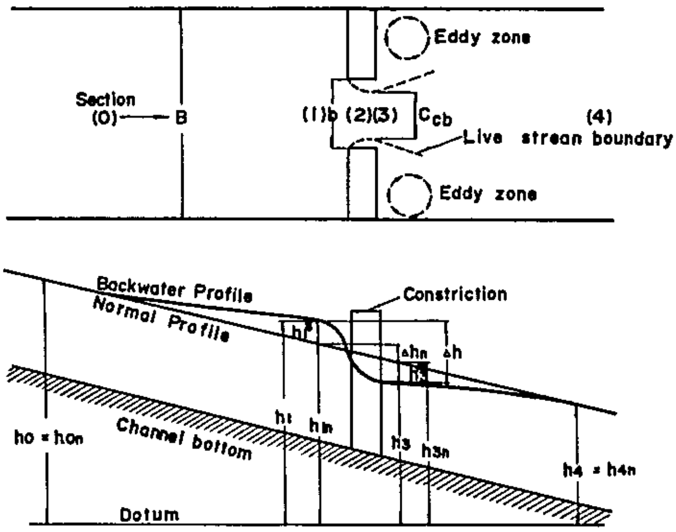
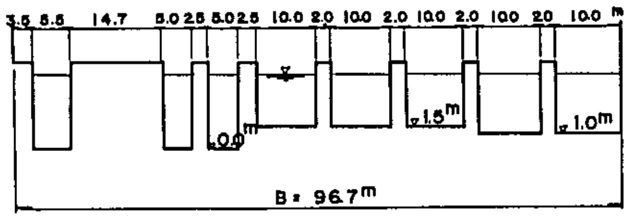


Fig. 2 Gunungsari dam



2. Gunungsari Dam and Djagir Dam.

The backwater of a constriction is to be expressed by a function of discharge and water level of the tailwater when the shape of the constriction is given. The backwater can be computed by Kindsvader-Carter-Tracy's method. An example of the computation is shown below for a case of a discharge 200 m^3/s passing through Gunungsari dam (Fig. 2). Though the actual computation is carried out by a trial and error method, only the last step is shown here for the convenience of comprehension.

a. When $Q = 200\ m^3/s$, the water level of the tailwater was 3 m , SHVP, that is, refering to Fig. 1

$$k_{3n} = 3.0\ m.$$

b. Refering to Figs. 1 and 2 and the structural drawings of the dam,

$$B = 96.7\ m,\ b = 65.5\ m,\ L = 8.5\ m$$
$$A_B = 96.7 \times 3 = 290.1\ m^2$$
$$P_B = 96.7 + (2 \times 3) = 102.7\ m$$
$$R_B = 2.825\ m,\ R_B^{2/3} = 1.998$$
$$\Sigma A_b = 131.5\ m^2$$
$$\Sigma P_b = 100.5\ m$$
$$R_b = 1.308\ m,\ R_B^{2/3} = 1.196$$

c. Taking n as 0.027,

$$K_B = (1/n) A_B R_B^{2/3} = 21,467$$
$$K_b = (1/n) \Sigma A_b R_b^{2/3} = 5,825$$
$$\therefore\ m = I - K_b/K_B = 0.729$$

d. $L/b = 0.130$ and $m = 0.729$, hence $C' = 0.69$ from the Kindsvader-Carter-Tracy's empirical diagram (hereafter refered as KCT diagram).

e. $K_r = 1$ from KCT diagram.

f. $j = A_j/A = 0.630$. $j > 0.1$, hence $K_j = 1$ from KCT diagram.

g. $n = 0.027$ and $m = 0.729$, hence $h_I^*/\triangle h = 0.78$ from KCT diagram.

h. $C_{basic} = C$ and $m = 0.729$, hence $k_a = 1$ from KCT diagram.

i. $\triangle h_n = h_f = I_l$ when $I = Q^2 n^2/(A_B R_B^{2/3})^2$ and l = distance between section 1 and 3. If we assume the position of section 1 to be around one span upstream from section 3, $l = L$ + one span $= 8.5 + 10 = 18.5$ m. Hence $\triangle h_n = 0.001$ m.

j. We assume $\triangle h = 0.236$ m.

k. $h_I^*/\triangle h = 0.78$, hence $h_I^* = 0.184$ m.

l. $h_3^* = \triangle h - h_I^* - \triangle h_n = 0.236 - 0.184 - 0.001 = 0.051$ m.

m. d_m denotes the mean depth at the contracted section.

$$d_m = \Sigma A_b / \Sigma b = 131.5/65.5 = 2.008 \ m$$
$$d_3' = d_m - h_3^* = 2.008 - 0.051 = 1.957 \ m$$
$$F_3 = \frac{Q}{bd_3'\sqrt{gd_3'}} = 0.356$$

Hence, according to KCT diagram, $K_F = 0.96$.

n. $C = C'K_rK_jK_Fk_a = 0.69 \times 1 \times 1 \times 0.96 \times 1 = 0.66$

o. $V_3 = Q/A_3 = Q/bd_3' = 1.560$ m/s
$d_1 = d_g' + \Delta h - \Delta h_n = 2.192$ m
$A_1 = Bd_1 = 212.0 \ m^2$
$V_1 = Q/A_1 = 0.943$ m/s

$$\therefore \ \Delta h = \frac{V_3^2}{2gC^2} - \alpha\frac{V_1^2}{2g} + h_f = 0.236 \ m$$

This value agrees with the assumed one. Therefore, the case of $Q = 200 \ m^3/s$ has been settled. By similar way, the backwater of contraction for several water levels of the tailwater were computed for Djagir dam as well as Gunungsari dam. Those are shown in Table 1, Figs. 3 and 4.

Table 1

Tailwater stage (m)	Discharge (m^3/s)	Δh (m)	h_I^* (m)	h_3^* (m)	Δh_n (m)	$h_I^* + \Delta h_n$ (m)
Gunungsari dam						
3.0	200	0.236	0.184	0.051	0.001	0.185
	300	0.517	0.403	0.111	0.003	0.406
	400	0.881	0.687	0.188	0.006	0.693
4.0	300	0.229	0.174	0.054	0.001	0.175
	400	0.416	0.316	0.098	0.002	0.318
	500	0.647	0.492	0.152	0.003	0.495
5.0	300	0.134	0.101	0.032	0.001	0.102
	400	0.226	0.170	0.055	0.001	0.171
	500	0.364	0.273	0.089	0.002	0.275
Djagir dam						
1.0	200	0.384	0.200	0.130	0.054	0.254
	250	0.585	0.304	0.197	0.084	0.388
	300	0.744	0.387	0.235	0.122	0.509
2.0	200	0.238	0.138	0.075	0.025	0.163
	300	0.502	0.291	0.155	0.056	0.347
	400	0.869	0.504	0.266	0.099	0.603
3.0	300	0.365	0.223	0.111	0.031	0.254
	400	0.632	0.386	0.191	0.055	0.441
	500	0.985	0.601	0.298	0.086	0.687

Fig.3 Discharge Curve of Gunungsari Dam

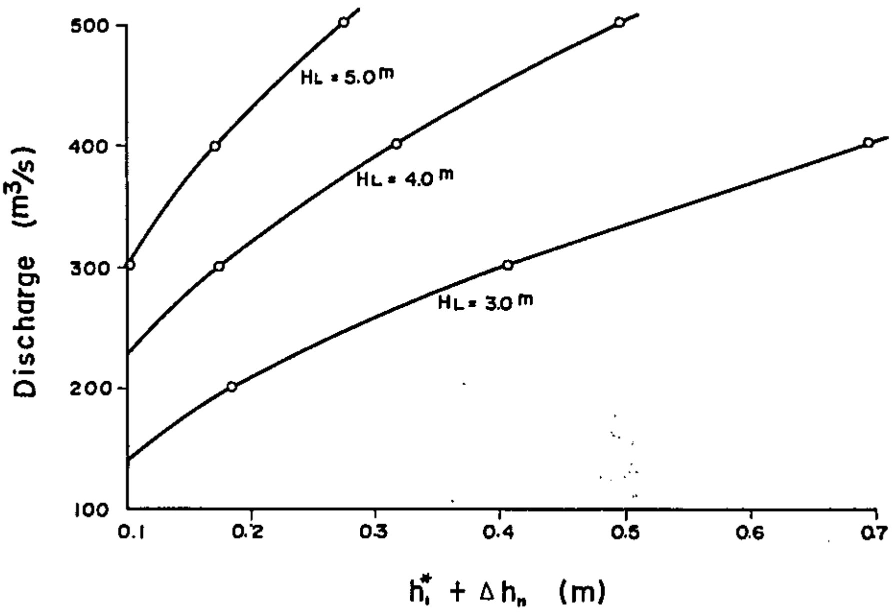
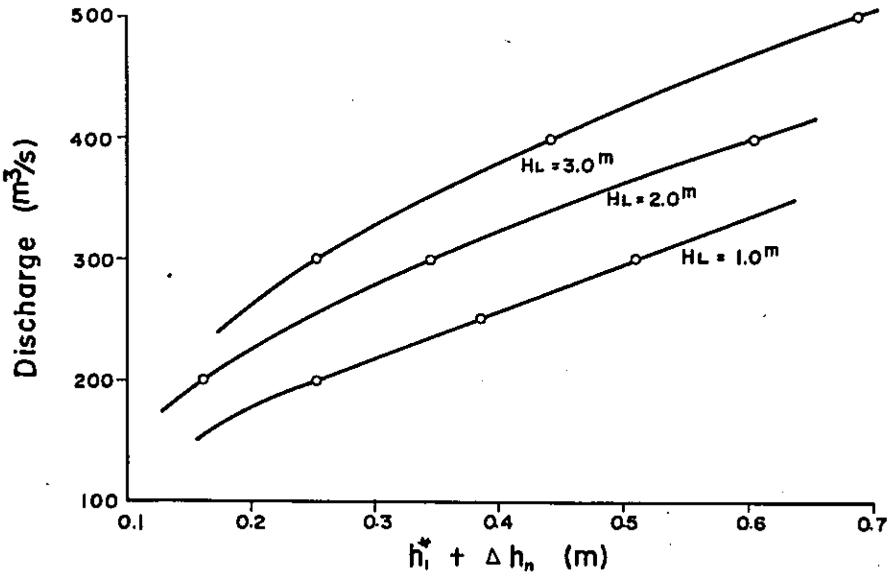


Fig.4 Discharge Curve of Djagir Dam



CHAPTER VI

TIDE LEVEL OUTSIDE OF MOROKREMBANGAN GATE

1. Comparison of Tide Level of Surabaja Harbor with That of Boezem.

Hourly records of tide level at Surabaja harbor and Boezem were collected from Administrator Pelabuhan Surabaja and DPPDT.

The contents of these records are as follows.

a. Records of tide level at Surabaja harbor.

Period : from Jan. 1966 to Mar. 1972
Lack of records : from Mar. to May in 1967, Jan. to Dec. in 1969
Datum of staff gage : LWS (m)
Conversion of LWS into SHVP : T(LWS) - 2.70m = T'(SHVP).
Time of observation : every hour
Most of the records were taken only in the daytime of weekdav.

b. Records of tide level at Boezem (sea-side).

Period	from Dec. 20, 1964 to Feb. 1972
Lack of data	from Jan. 1 to 10 in 1965, Aug. 22 to Sept. 20 in 1970
Datum of staff gage	SHVP (m)
Time of observation	every hour

The records at Surabaja harbor are not so many because of lacking, but the records at Boezem (sea-side) are enough for analysis. Hence, it is better if the records at Boezem (sea-side) are available in place of those at the harbor.

In order to know differences between the both, we picked up 3 periods of records observed at the same time, which are shown in Fig. 1.

This shows no significant difference between the both curves. Accordingly, we used the records at Boezem (sea-side) for the study of design tide level.

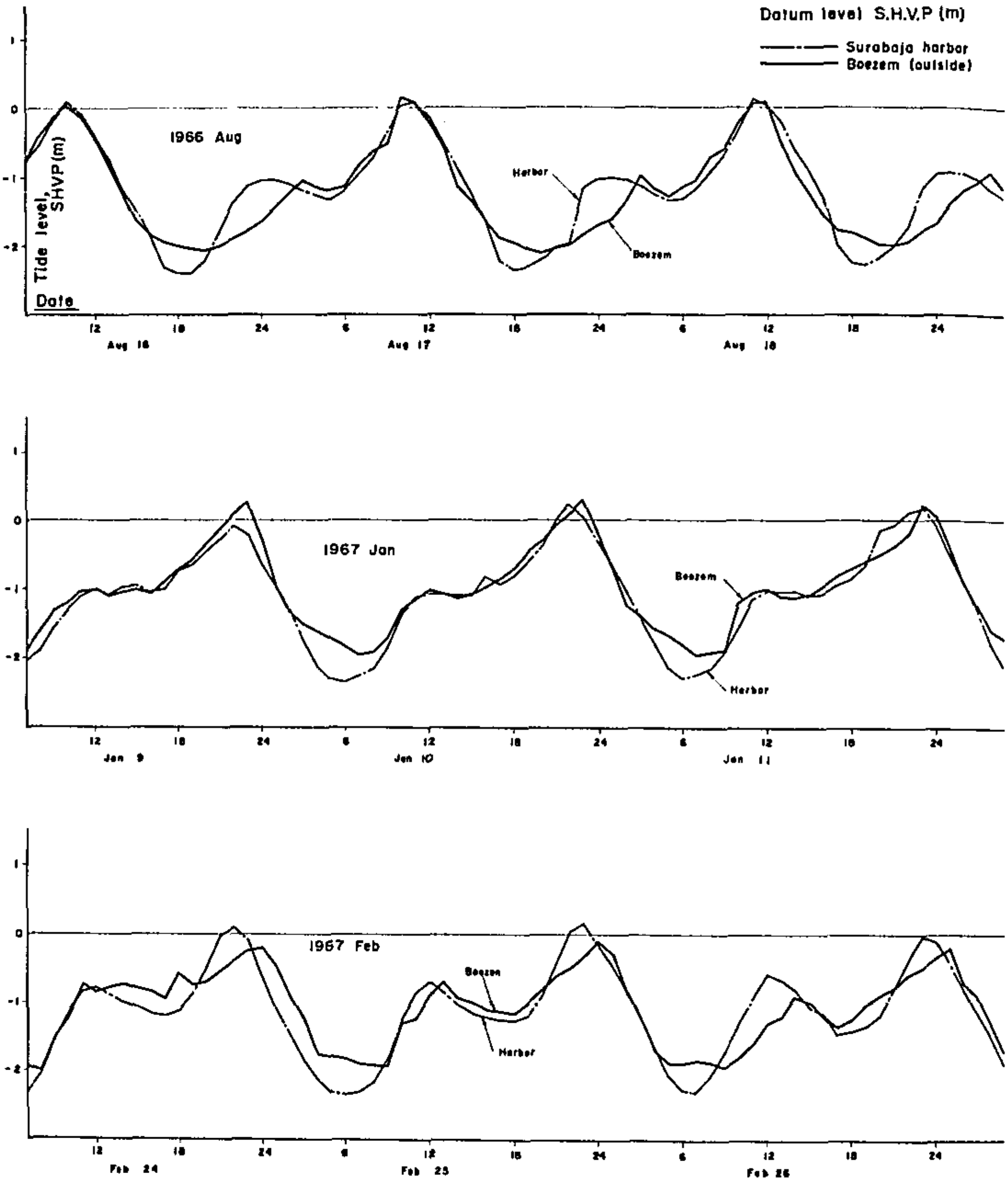
2. High Water Level, Mean Water Level, and Low Water Level.

(1) High water level.

From the hourly records of tides at Boezem (sea-side), we picked up monthly highest tide levels which are shown in table 1.

Further, we picked up annual highest tide levels in the years 1967,

Fig. 1 Comparison of Tide Levels between Surabaja harbor And Boezen



1968, 1969, and 1971 which have no lack of records, and took an average of these four values. This was used as High Water Level.

High Water Level = +0.373 m (SHVP)

Table 1 Monthly Highest Tide Levels (m, SHVP)

month	1965	1966	1967	1968	1969	1970	1971	1972
Jan.		0.31	0.31	0.35	0.30	0.15	0.25	0.25
Feb.	0.20	0.10	0.19	-0.10	0.10	-0.09	0.20	0.25
Mar.	-0.20	-0.25	-0.02	-0.14	-0.09	-0.10	0.20	
Apr.	-0.15	0.05	0.25	0.20	0.15	0.05	0.35	
May	0.23	0.25	0.15	0.25	0.17	0.40	0.43	
June	0.44	0.25	0.17	0.37	0.30	0.36	0.30	
July	0.35	0.15	0.10	0.40	0.35		0.35	
Aug.	0.12		0.06	0.10	0.00	0.35	0.10	
Sept.	-0.15		-0.20	-0.02	-0.20	0.42	0.20	
Oct.	-0.20	-0.12	-0.10	0.25	-0.10	0.20	0.20	
Nov.	0.10	0.30	0.25	0.36	0.25	0.35	0.40	
Dec.	0.24	0.28	0.30	0.30	0.35	0.25	0.38	

Note: Average of annual highest tide levels +0.373

(2) Mean water level.

A relationship between LWS and SHVP shown below is reported to be made based on tide records at Surabaja harbor during the period from 1913 to 1930.

	LWS(m)	SHVP(m)
HHW	+3.22	+0.52
	+2.70	± 0
GHW	+2.13	-0.57
GW	+1.48	-1.22
GLW	+0.88	-1.82
LWS	± 0	-2.70
LLW	-0.02	-2.72

where HHW = the highest tide level in the past records,
GHW = the most frequent high tide level,
GW = mean tide level,
GLW = the most frequent low tide level, and
LLW = the lowest tide level in the past records.

It is seen in the above table that mean water level at this harbor is -1.22 m, SHVP.

(3) Low water level.

Table 2 shows monthly lowest tide levels collected by a similar way to the case of High Water Level. Annual lowest tide levels were picked up for the four years of 1967, 1968, 1969 and 1971 which have no lack of records and an average of them was take. This was used as Low Water Level.

LWL = -2.083 m, SHVP

Table 2 Monthly Lowest Tide Levels (m, SHVP)

month	1965	1966	1967	1968	1969	1970	1971	1972
Jan.		-1.99	-2.08	-2.01	-2.03	-2.00	-1.97	-1.97
Feb.	-1.99	-1.99	-2.05	-1.95	-1.97	-2.03	-1.96	-1.96
Mar.	-1.96	-1.99	-2.02	-1.95	-1.98	-1.98	-1.91	
Apr.	-1.86	-1.99	-2.05	-1.95	-1.91	-1.95	-1.91	
May	-1.90	-1.98	-2.04	-1.97	-1.90	-1.96	-1.91	
June	-1.96	-1.99	-2.07	-1.97	-2.00	-2.00	-1.88	
July	-1.99	-2.06	-2.20	-1.97	-2.02		-1.92	
Aug.	-1.99		-2.12	-1.97	-2.07	-2.02	-1.98	
Sept.	-1.97		-2.15	-2.02	-2.08	-2.03	-1.99	
Oct.	-1.96	-2.08	-2.05	-2.06	-2.02	-1.99	-1.96	
Nov.	-1.98	-2.00	-2.07	-2.00	-2.05	-1.99	-1.91	
Dec.	-1.99	-2.07	-2.09	-2.01	-2.01	-1.99	-1.93	

Note: Average of monthly lowest tide levels = -2.083

CHAPTER VII

CALCULATION OF NONUNIFORM FLOW AND UNSTEADY FLOW

1. Nonuniform Flow.

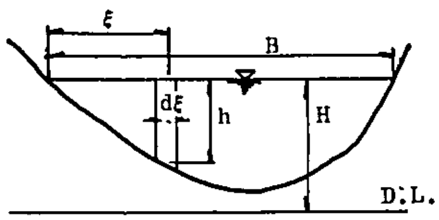
$$H_i = H_{i-1} + \frac{DQ^2}{2g}\left(\frac{1}{A_{i-1}^2} - \frac{1}{A_i^2}\right) + \frac{Q^2X}{2}\left(\frac{N_{i-1}^2}{R_{i-1}^{4/3}\,A_{i-1}^2} + \frac{N_i^2}{R_i^{4/3}\,A_i^2}\right)$$

Where, according to Ida,

$$D = \alpha\frac{A^2\int_0^B \frac{h^3}{n^3}\,d\xi}{\left(\int_0^B \frac{h^{5/3}}{n}\,d\xi\right)^3}$$

$$N = \frac{\int_0^B h^{5/3}\,d\xi}{\int_0^B \frac{h^{5/3}}{n}\,d\xi}$$

$$R = \left(\frac{1}{A}\int_0^B h^{5/3}d\xi\right)^{3/2}.$$



Notation

H = Elevation of water level (m),
g = Acceleration of gravity (m/sec^2),
Q = Discharge (m^3/sec),
A = Water area (m^2),
X = Small distance between two cross-sections (m),
D = Coefficient for correction,
N = Equivalent coefficient of roughness for the whole cross-section,
R = Equivalent hydraulic depth for the whole cross-section (m),
α = Correction coefficient for vertical distribution of velocity,
n = Manning's coefficient of roughness for every part of cross-section.

Suffix denotes number of a cross-section, from downstream to upstream.

2. Unsteady Flow.

(1) Equation of motion.

Equation of motion of unsteady flow is written as follows:

$$\frac{1}{g}\frac{\partial v}{\partial t} + \frac{\partial v}{g\partial x} + \frac{\partial H}{\partial x} + \frac{n^2|v|v}{R^{4/3}} = 0$$

In finite difference,

$$\frac{1}{g}\frac{v_{\tau+1,i} - v_{\tau,i}}{\Delta t} + \frac{v_{\tau,i}}{g}\cdot\frac{v_{\tau,i+1} - v_{\tau,i}}{\Delta x} + \frac{H_{\tau,i+1} - H_{\tau,i}}{\Delta x} + \frac{1}{2}\left[\frac{n^2|v_{\tau,i}|v_{\tau+1,i}}{R_{\tau,i}^{4/3}} + \frac{n^2|v_{\tau,i+1}|v_{\tau+1,i+1}}{R_{\tau,i+1}^{4/3}}\right] = 0$$

(2) Equation of continuity.

Equation of continuity of unsteady flow is written as follows:

$$\frac{\partial H}{\partial t} + \frac{1}{B} \cdot \frac{\partial (Av)}{\partial x} = 0$$

In finite difference,

$$\frac{H_{\tau+1,i+1} - H_{\tau,i+1}}{\triangle t} + \frac{1}{B} \cdot \frac{A_{\tau,i+1} v_{\tau,i+1} - A_{\tau,i} v_{\tau,i}}{\triangle x} = 0$$

Notation

v = Mean velocity (m/sec),
g = Acceleration of gravity (m/sec^2),
H = Elevation of water level (m),
n = Manning's coefficient of roughness,
τ and $\tau+1$ = Two successive times. The interval of them is $\triangle t$.
i and i+1 = Numbers of two successive cross sections. The distance between them is $\triangle x$ (m).
R = Hydraulic mean depth (m),
B = River width (m),
A = Cross-sectional area (m^2).

CHAPTER VIII

RAINFALL

1. Distribution of Rain-Gage Stations.

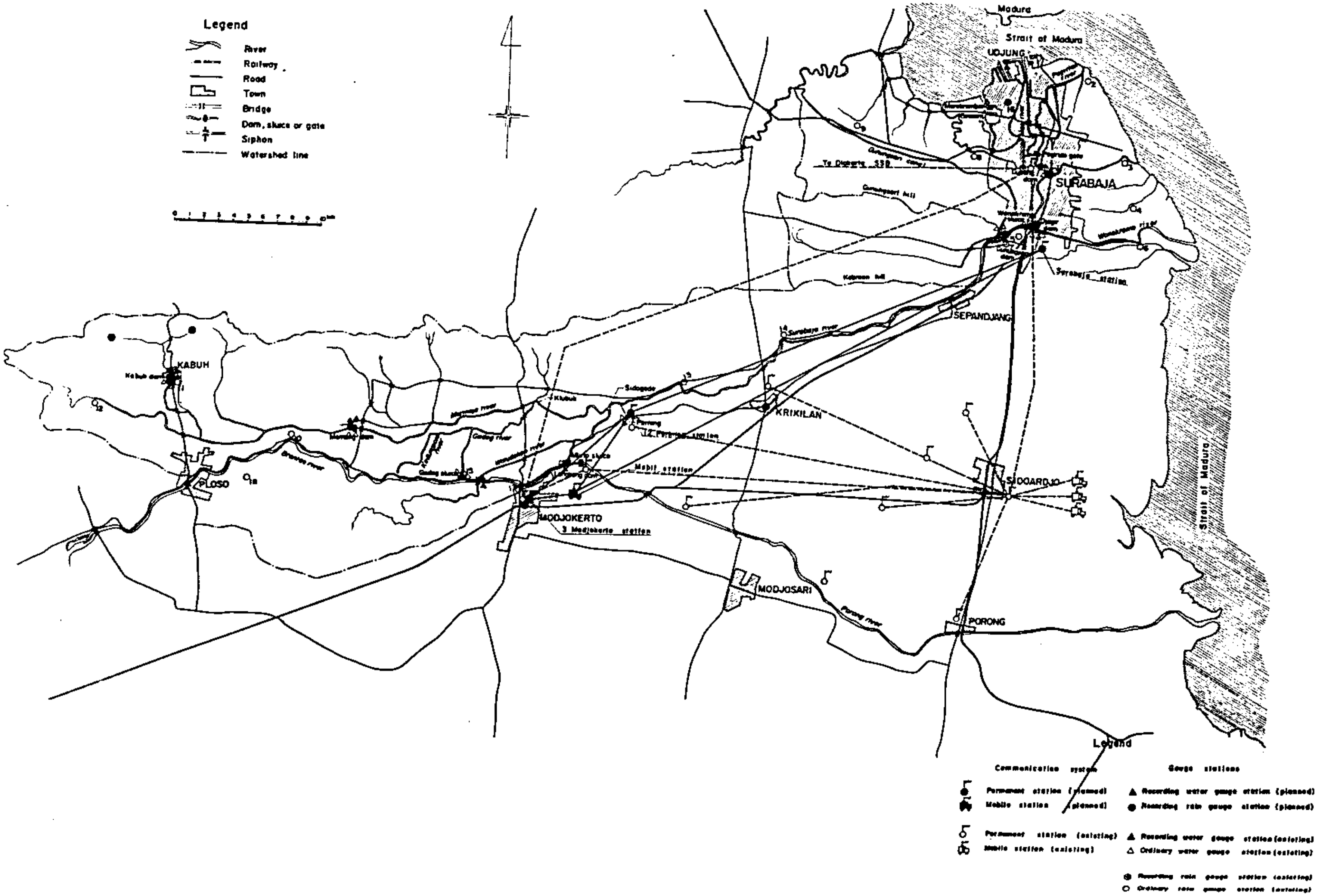
Rain-gage stations which are located in the area containing the Marmojo river basin as well as the Great Surabaja area are shown in Fig. 1 and required explanations on them are given in Table 1.

Table 1 Rain-gage Station

Station No.	Station name	Type of rain-gage	Year of start	Managing office	Elevation of station-site (m, SHVP)	Gaging time of daily rain-fall (WIB)	Period of daily rainfall data
1	Gubeng	nonrec.	1910	Wkr	+ 6.0	7.00	1910,'50~'72
2	Ked.tjowek	nonrec.	before WWII	Wkr	+ 3.5	7.00	1950~'72
3	Larangan	nonrec.	"	Wkr	+ 3.5	7.00	1950~'72
4	Keputih	nonrec.	"	Wkr	+ 3.5	7.00	1950~'72
5	Kebonagung	nonrec.	"	Wkr	+ 4.0	7.00	1950~'72
6	Wonoredjo	nonrec.	"	Wkr	+ 3.5	7.00	1950~'72
7	Gunungsari	nonrec.	"	Wkr	+ 6.0	7.00	1950~'72
8	Banjuurip	nonrec.	"	Wkr	+ 6.0	7.00	1950~'72
9	Semimi	nonrec.	"	Wkr	+ 5.5	7.00	1950~'72
19	Wonokromo	nonrec.	1971	Wkr	+ 5.54	7.00	1971~'72
19	Wonokromo	rec. (7 days)	Apr. 1971	Wkr	+ 5.54	-	-
20	Kedung	nonrec.	before WWII	Djb	+28.0	7.00	1951~'72
10	Tapen	nonrec.	"	Djb	+30.0	7.00	1950~'72
11	Kabuh	nonrec.	"	Djb	+50.0	7.00	1950~'72
12	Tandjung	nonrec.	"	Djb	+55.0	7.00	1950~'72
15	Gedeg	nonrec.	1910	Mdk	+26.0	7.00	'10 '33 '50~'72
17	Terusan	nonrec.	1910	Mdk	+25.0	7.00	'10 '33 '51~'72
13	Wringinanom	nonrec.	before WWII	Sda	no data	7.00	1950~'72
14	Krikilan	nonrec.	"	Sda	no data	7.00	1950~'72
18	Djatisari	nonrec.	"	Mda	+33.0	7.00	1950~'72
16	Surabaja	rec. (1 day)	1960	Met	+10 ft from MSL	-	-
16	Surabaja	nonrec.		Met	"	7.00	1960~'72

WWII : World War II
SHVP : Surabaja Haven Vloed Peil
WIB : Waktu Indonesia Barat (West Indonesian Time)
nonrec. : Nonrecording gage
rec. : Recording gage
Wkr : Wonokromo
Djb : Djombang
Mdk : Modjokerto
Sda : Sidoardjo
Mda : Modjoagung
Met : Stasium Meteorologie dan Geofisika Surabaja

Fig.1 Distribution of Rain - Gage Stations



2. Correlation of Daily Rainfall among Rain-Gage Stations.

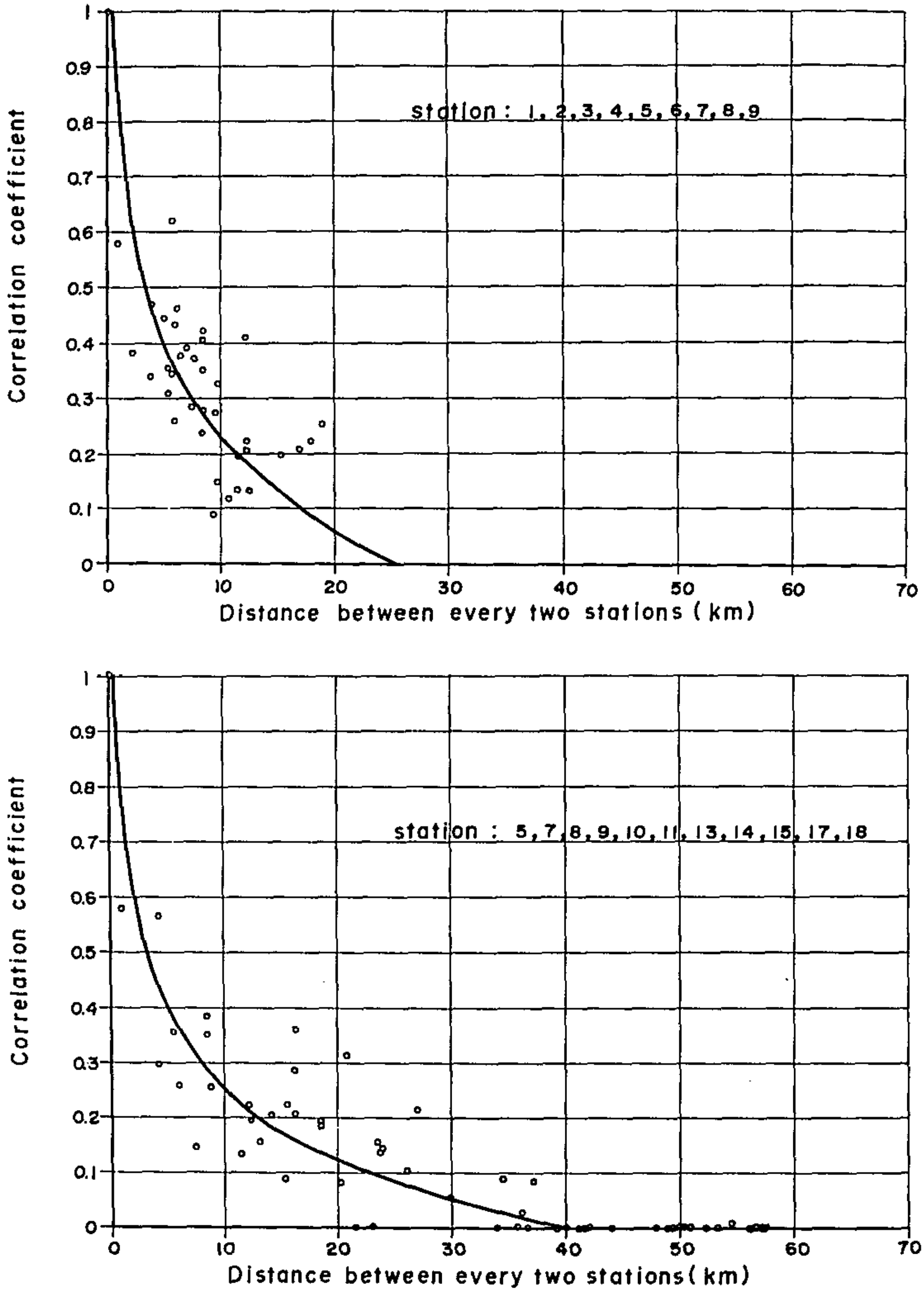
Of the above-mentioned gage stations, eighteen stations from Station No. 1 to Station No. 18 were used for analyzing correlation by reason that they have records over a period long enough to analysis.

Distance and correlation coefficient of daily rainfall between every two stations are shown in Table 2. After some study, it was considered to be reasonable that the above-mentioned stations were classified into two groups; (1) city area group of nine stations from No. 1 to No. 9 and (2) upstream g group of eleven stations from No. 5 to No. 18 excluding No. 6, 12 and 16 which were judged to be unsuitable for analysis. Two kinds of correlation curve for the two groups are shown in Fig. 2 which shows that the nature of curves for distances less than 10 km is almost the same.

Table 2 Distance and Correlation Coefficient between Every Two Stations

Stations	Dist. (km)	Corr. coef.	Stations	Dist. (km)	Corr. coef.	Stations	Dist. (km)	Corr. coef.
1 - 2	6.6	0.374	3 - 4	3.95	0.471	5 - 6	7.65	0.373
3	5.0	0.444	5	9.30	0.089	7	0.85	0.578
4	6.2	0.464	6	5.65	0.310	8	6.00	0.259
5	5.7	0.345	7	9.80	0.148	9	12.20	0.223
6	7.1	0.388	8	8.55	0.280	10	49.85	0
7	5.9	0.435	9	17.10	0.208	11	56.85	0
8	3.7	0.341	10	68.65	0	12	61.50	0
9	12.3	0.411	11	65.25	0	13	23.65	0.156
10	53.8	0	12	70.00	0	14	16.25	0.358
11	60.4	0	13	32.80	0	15	39.90	0
12	65.2	0	14	25.35	0.206	16	9.30	0
13	28.3	0.025	15	49.00	0	17	36.40	0
14	20.8	0.304	16	8.40	0.128	18	53.25	0
15	44.3	0	17	45.65	0			
16	5.0	0	18	62.20	0	6 - 7	8.50	0.406
17	41.1	0				8	10.65	0.118
18	57.4	0	4 - 5	7.65	0.284	9	18.85	0.254
			6	1.75	0.384	10	57.25	0
2 - 3	5.85	0.618	7	8.45	0.422	11	64.35	0
4	9.70	0.327	8	9.60	0.273	12	69.00	0
5	12.35	0.206	9	18.05	0.225	13	30.75	0
6	11.40	0.193	10	57.50	0	14	23.50	0.365
7	12.50	0.133	11	69.35	0	15	46.90	0.170
8	8.40	0.238	12	69.15	0	16	12.20	0.008
9	15.35	0.198	13	31.15	0	17	43.25	0.062
10	58.90	0	14	23.75	0.068	18	60.55	0.149
11	65.00	0	15	47.40	0			
12	69.75	0	16	10.85	0.048			
13	34.15	0	17	43.85	0			
14	26.85	0.210	18	60.95	0			
15	50.00	0.051						
16	5.90	0.019						
17	47.00	0						
18	62.50	0						

Fig. 2 Correlation of Daily Rainfall



Stations	Dist. (km)	Corr. coef.	Stations	Dist. (km)	Corr. coef.	Stations	Dist. (km)	Corr. coef.
7 - 8	5.65	0.356	10 - 11	86.50	0.384	14 - 15	23.70	0.136
9	11.40	0.133	12	12.45	0	16	21.45	0
10	49.20	0	13	26.85	0.217	17	20.30	0.080
11	56.15	0	14	34.00	0	18	37.30	0.086
12	60.90	0	15	12.25	0.195			
13	23.05	0	16	53.20	0	15 - 16	44.35	0.041
14	15.60	0.222	17	16.35	0.206	17	4.15	0.564
15	39.20	0	18	4.35	0.294	18	14.35	0.206
16	9.0	0						
17	35.80	0	11 - 12	4.90	0.110	16 - 17	41.40	0
18	52.65	0	13	34.50	0.094	18	56.85	0
			14	41.20	0			
8 - 9	8.60	0.352	15	20.80	0.315	17 - 18	18.50	0.193
10	50.80	0	16	59.15	0			
11	59.15	0	17	24.85	0.143			
12	62.00	0	18	8.85	0.254			
13	25.85	0.104						
14	18.50	0.191	12 - 13	39.05	0.010			
15	41.70	0	14	45.95	0			
16	3.50	0.251	15	24.70	0			
17	38.65	0	16	63.95	0			
18	54.50	0.009	17	28.85	0			
			18	11.20	0			
9 -10	44.00	0						
11	49.75	0	13 - 14	7.45	0.146			
12	54.60	0	15	16.20	0.285			
13	21.35	0	16	28.70	0			
14	15.15	0.088	17	12.90	0.153			
15	36.15	0.028	18	30.00	0.056			
16	9.60	0						
17	33.65	0						
18	47.95	0						

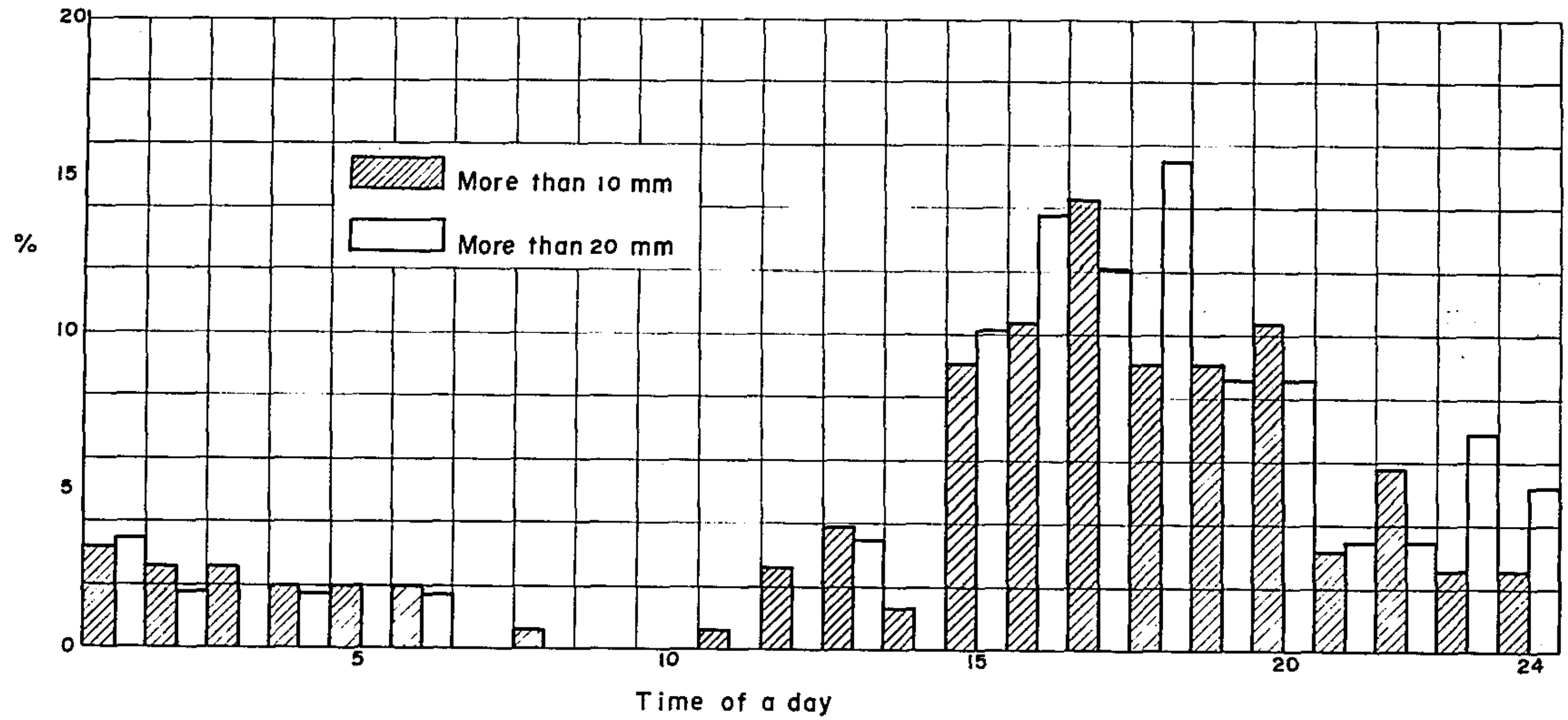
If we integrate the correlation curve with respect to distance, we get a linear dimension. This means a radius of an area where daily rainfall is sufficiently correlated almost at the same time, or this means an effective radius of an area where daily rainfall is sufficiently correlated, because major amount of daily rainfall is concentrated in four to six hours from 15 to 20 o'clock of a day and one daily rainfall is almost independent of the next one as shown in Fig. 3 [1].

Integrating these curves respectively, we get 6.2 km for city group and 7.7 km for upstream group or we get the result that effective diameters of simultaneous daily rainfall are 12.5 km for the city area and 15.5 km for upstream area.

3. Probability of Daily Rainfall at Surabaja City Area.

[1] T. Tanimoto; Colombo Plan Expert; Revised and enlarged edition of the hourly rainfall analysis in Java. Institute of Hydraulic Engineering, Directorate General of Water Resources Development, DPUT.

Fig. 3 Frequency Distribution of Hourly Rainfall at Surabaja



In order to study the return period of daily rainfall in the Surabaja city area, 9 rain-gage stations, from No. 1 to No. 8 and No. 16 shown in Fig. 3 were picked up from the above-mentioned 18 stations, and the rainfall records during 22 years from 1950 to 1971 were used for probability analysis. The results are shown in Fig. 4-1 and 4-2.

It can be seen from Fig. 5 that the nine rain-gage stations may be classified into five groups along the Mas river from Gunungsari to the river mouth, (1) No. 7, 5 and 6, (2) No. 4, (3) No. 8, 1 and 3, (4) No. 16 and (5) No. 2. From the probability curves in Fig. 4, we read daily rainfall at every station for every return period of 1.25, 2, 5, 10, 20, 25, 50 and 100 years classifying the stations into the above-mentioned groups. They are shown in Table 3.

Table 3 Daily Rainfall (mm) of the Same Return Period at Nine Rain-Gage Stations

Return period (years)	1.25	2	5	10	20	25	50	100
Station								
7	80	103	130	147	163	170	184	197
5	77	94	113	126	137	141	152	160
6	62	87	123	147	170	180	202	223
Mean	73	95	122	140	157	164	179	193
4	77	100	130	150	168	174	190	206
8	82	106	137	158	178	185	204	220
1	78	103	133	153	172	176	195	210
3	71	94	122	140	158	163	180	195
Mean	77	101	134	150	169	175	193	208
16	77	89	103	112	118	122	127	133
2	96	117	145	163	178	184	197	210
Mean	79	99	126	145	161	166	181	195

Fig. 6 shows the trend of variation of daily rainfall for every return period mentioned above along the Mas river from upstream to downstream, and in this figure mean values were plotted for the daily rainfall at No. 7-5-6 group and No. 8-1-3 group. It is easily seen from Fig. 6 that the daily rainfall at Station No. 16 is abnormally small compared with the others (this rainfall was measured by nonrecording gage). Therefore, in order to modify it, daily rainfall for every return period at Station No. 16 was read from Fig. 5 as shown below

Table 4 Daily Rainfall at Surabaja (Station No. 16) for Different Return Period

Return period (years)	1.25	2	5	10	20	25	50	100
Daily rainfall (mm)	87	109	140	156	174	180	194	209

and probability curve which might be correct was drawn as shown in Fig. 6 by using the above values.

Fig. 4-1 Probability of Daily Rainfall

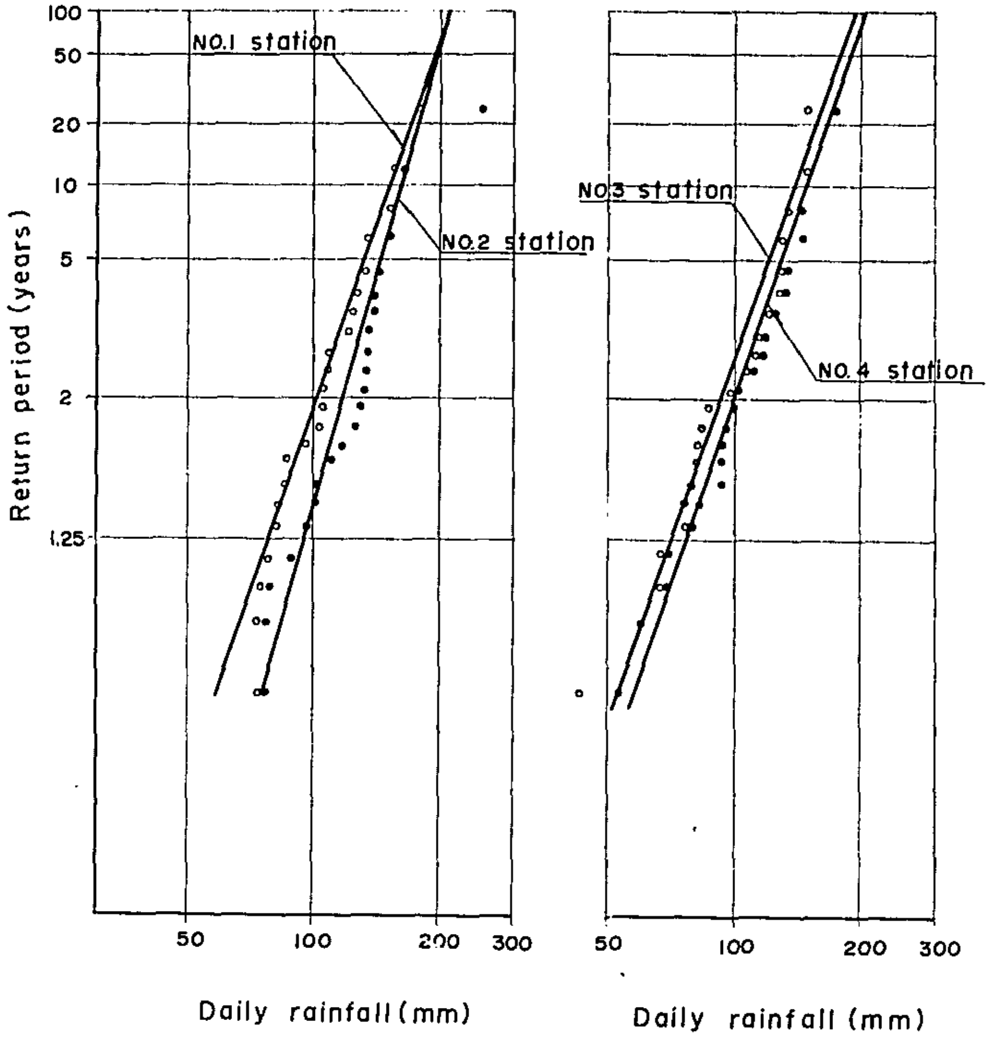


Fig. 4-2 Probability of Daily Rainfall

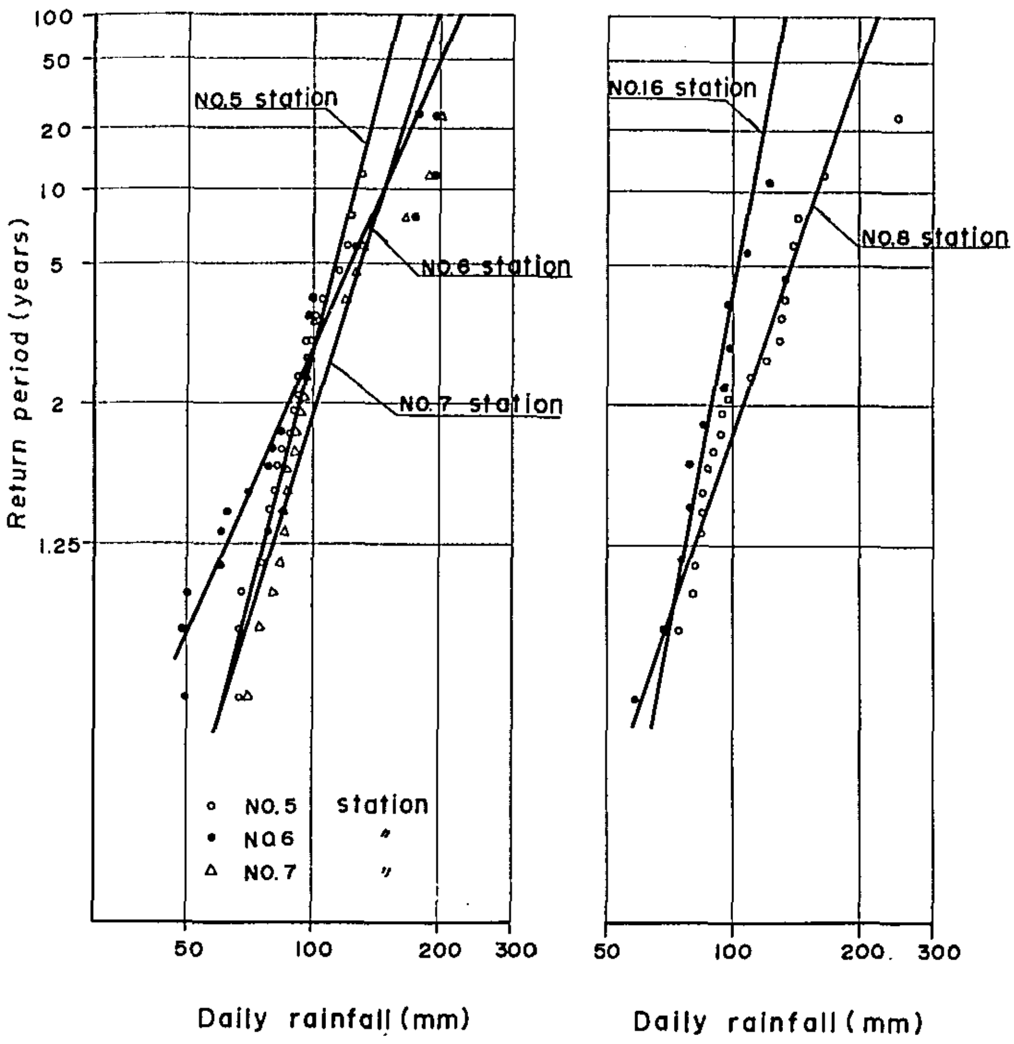


Fig. 5 Rain-gage Stations in Surabaja Area

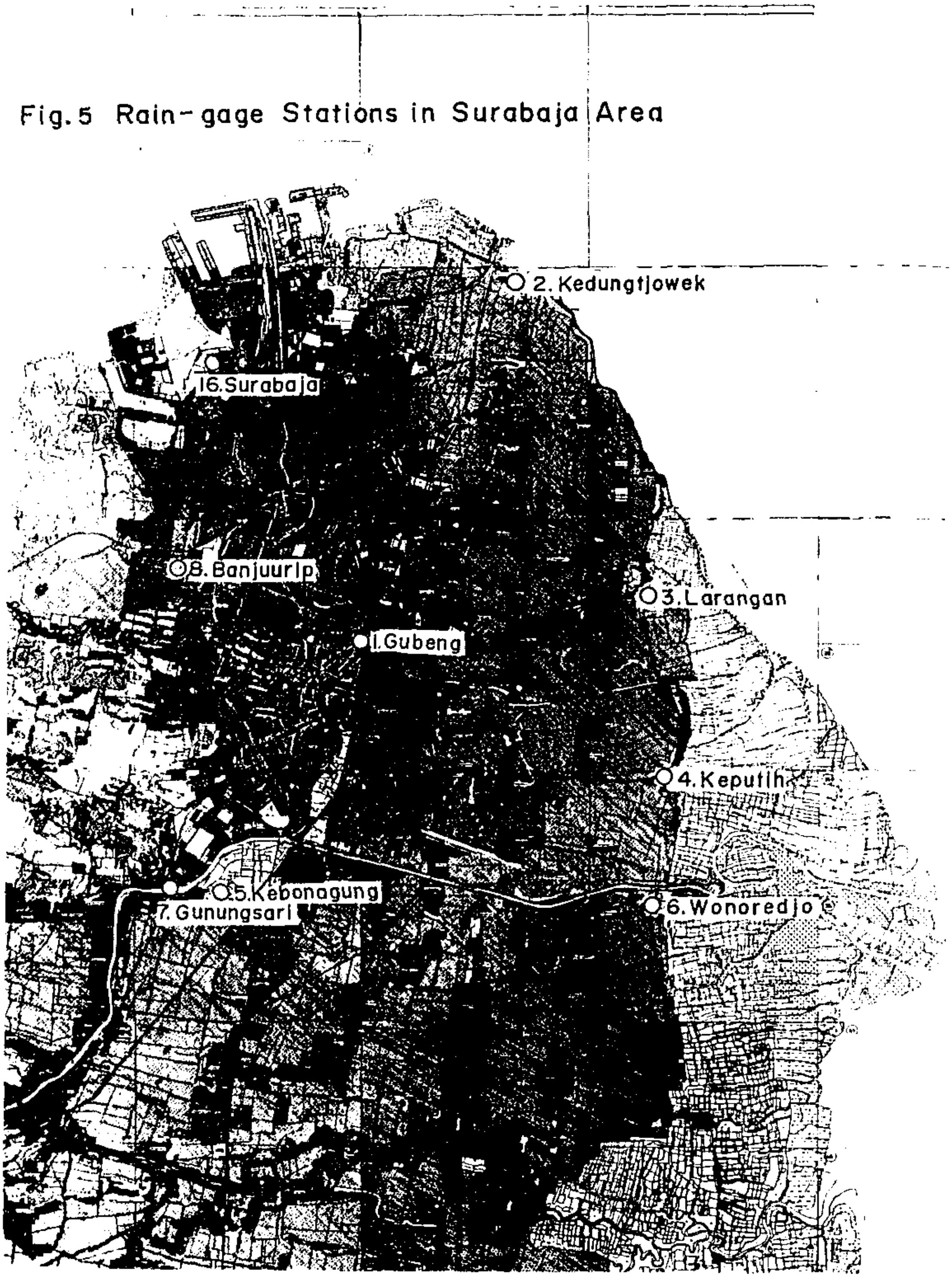


Fig. 6

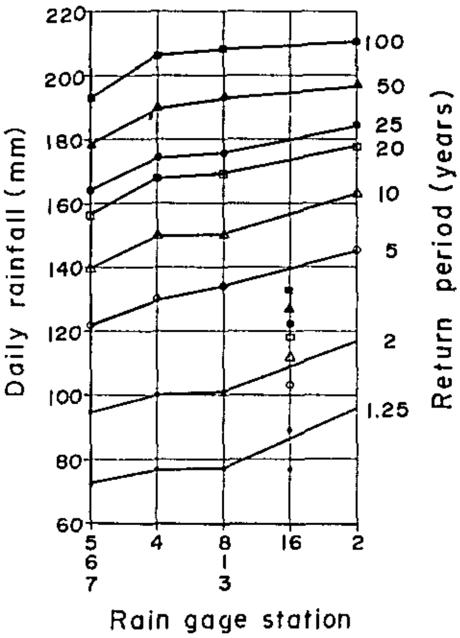
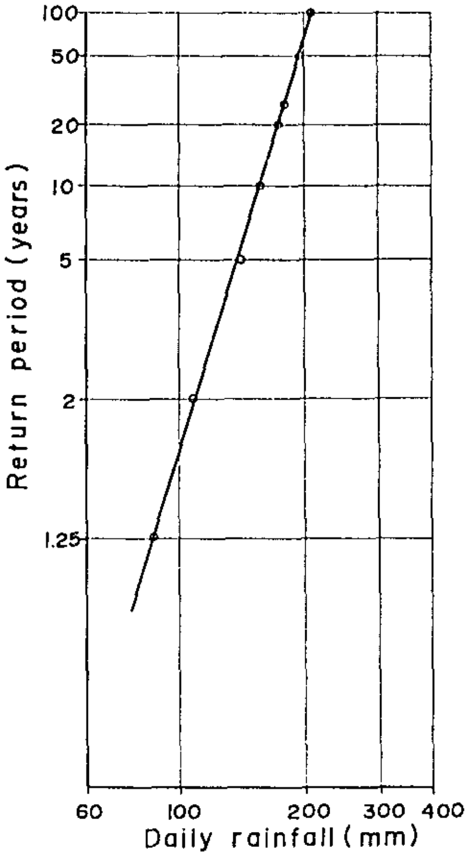

220
200
180
160
140
120
100
80
60
Daily rainfall (mm)
100
50
25
20
10
5
2
1.25
Return period (years)
5
6
7
4
8
1
3
16
2
Rain gage station

Fig. 7


100
50
20
10
5
2
1.25
Return period (years)
60
100
200
300
400
Daily rainfall (mm)

4. Rainfall Intensity-Duration-Frequency Curves at Surabaja.

For calculation of runoff from rainfall, rational formula is used in this study.

$$Q = 0.2778\ frA \qquad (1)$$

Q = peak discharge (m^3/s)
f = runoff coefficient
A = catchment area (km^2)
r = mean intensity of maximum rainfall which corresponds to concentration time (mm/hr)

Let D be duration of rainfall, the relation between duration and mean intensity of rainfall within the duration is usually expressed by the following equations.

$$r_D = \frac{a}{D + b} \qquad (2)$$

$$r_D = \frac{c}{D^n} \qquad (3)$$

where a, b, c and n are experimental constants and r_D is expressed in mm/hr when D is expressed in hr. The equation (2) is called Talbot type and the equation (3) is called Sherman type. According to Linsley and others [2], the former is applicable to the duration from 5 to 120 min and latter is applicable to the duration larger than 2 hr. The value of n is usually estimated at 0.4 to 0.6.

Necessary rainfall data for the drainage system in Surabaja have been collected from the Meteorological Agency in Djakarta, since all the self-registering papers observed at Surabaja Meteorological Station, Surabaja, have been sent to Djakarta every month. The self-registering papers have been analized for each rainfall between 1962 and 1972 and then the heaviest rainfall in different durations that occured in each year are picked out and tabulated as given in Table 5.

Table 5 The Heaviest Rainfall Intensity in Each Year in Different Durations

Serial No.	Year	Month & Date	Rainfall intensities in different durations					
			5	15	30	45	60	120
1	1962	1 - 11	120	96	68	48.5	40	23.3
2	1963	1 - 2, 3	101	34	31.4	22.4	18	10.4
3	1964	4 - 13	140	80	80	60	60	39
4	1965	1 - 30	115	80	62	53.3	49	26.4
5	1967	11 - 18	110	80	80	80	70	40
6	1968	1 - 15	180	150	120	107	83.3	42.2
7	1969	12 - 5	120	60	60	48	40	21.9
8	1970	1 - 20 11 - 28	120	120	80	62	54.2	36.2
9	1971	2 - 26	150	120	100	83.4	70	43.5
10	1972	1 - 11	118	80	48	33.6	25.6	13

[2] P.K. Linsley, M.A. Kohler, J.L.H. Paulhus: Applied hydrology, p.91.

Basic data for rainfall intensity-duration-frequency are derived from self-registering record papers of 11 years record, from 1962 through 1972, missing one year in 1966, as given in Table 5.

In statistical sence, recurrence interval of rainfall intensity that will probably be exceeded as an average once in given years may be expressed mathematically in different methods which have been developed in the past few decades. Among which are: i) Approximate solution process by either Hazen Plot or Thomas Plot, ii) Gumbel-Chow's method, iii) Logarithmic normal distribution method, and iv) Iwai-Takase's method etc. The first one is probably most widely used today because of its simplified process.

For the better understanding, example of the computation of Approximate Solution process by Hazen Plot is conducted as explained in Appendix.

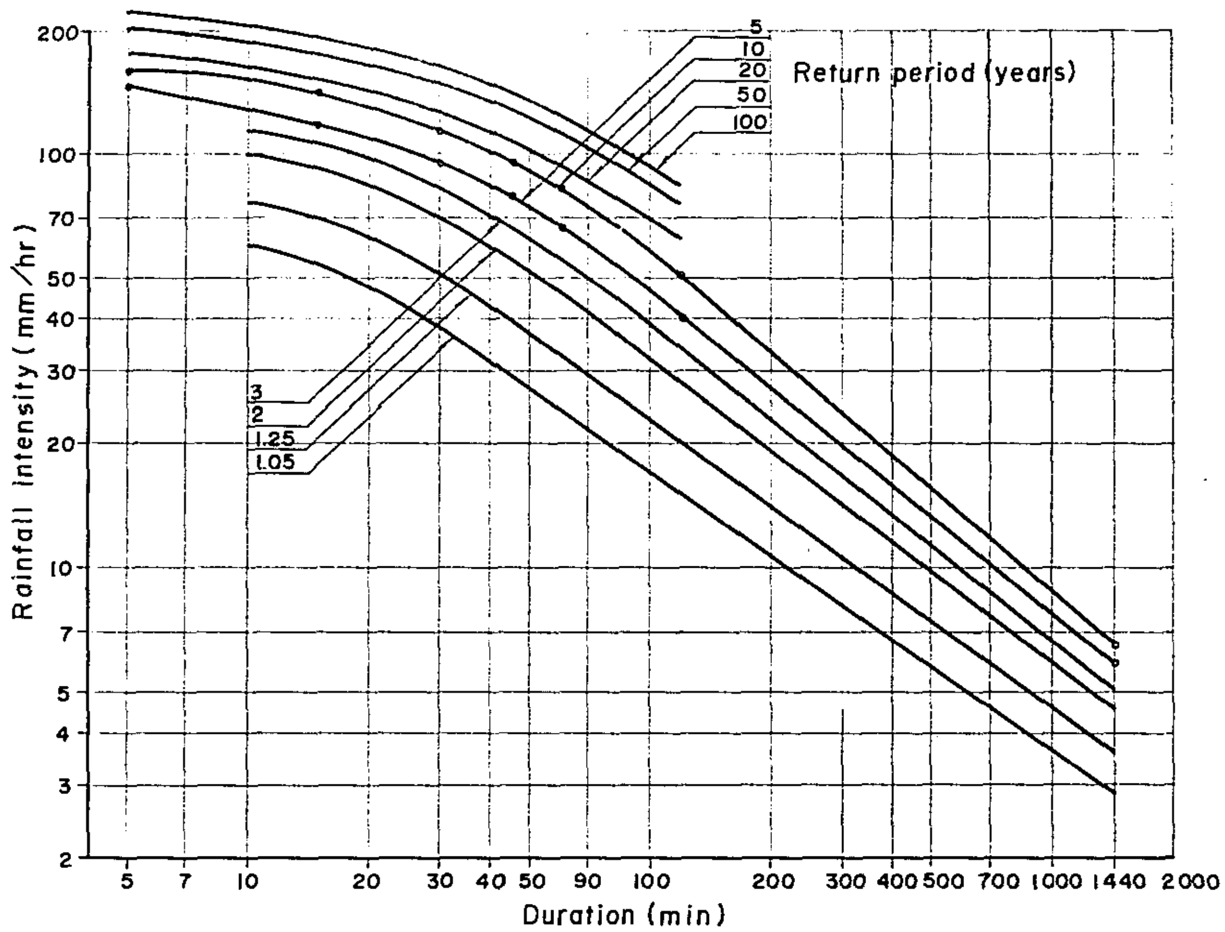
The constants a and b in the equation (2) are computed by using the least-square method as given in Appendix.

From the data as given in Table 5, rainfall-intensity-duration equations at different return periods have been calculated for the durations less than 120 min by means of an electronic computer as given below:

5 yr	return	period;	$I = \frac{6,680}{t + 41}$
10	"	"	$I = \frac{8,800}{t + 50}$
20	"	"	$I = \frac{11,000}{t + 57}$
25	"	"	$I = \frac{11,700}{t + 59}$
50	"	"	$I = \frac{13,800}{t + 63}$
100	"	"	$I = \frac{15,700}{t + 66}$
200	"	"	$I = \frac{17,600}{t + 68}$

These relations were connected with the other relations of Sherman type for the duration larger than 120 min so that the latter curves may pass through the points which express the daily rainfall at Surabaja for every return period of 10, 5, 3, 2, 1.25 and 1.05. The curves are shown in Fig. 8.

Fig.8 Rainfall Intensity, Duration and Return-period at Surabaja



APPENDIX

CALCULATIONS OF RAINFALL-INTENSITY-DURATION-EQUATION BY HAZEN'S PLOT

In Hazen's plot, for each duration selected such as 5, 15, 30, 45, 60, and 120 minutes, the heaviest rainfall that occurred in each year is listed, with no tabulation of lesser intensities during that same calender year, even though some of them might be greater than rainfall intensities that occurred in other years of record. This method gives the probability of occurrence that the maximum rainfall in any one year for a specified duration will equal or exceed a given intensity. Years heaviest rainfall intensity for each durations are arrayed in Table 1.

Table 1 Years Heaviest Rainfalls in the Last Eleven Years

Order of Occurrence	Rainfall Intensities in Different Durations (mm/hr)					
	5	15	30	45	60	120
1	180	150	120	107	83.3	43.5
2	150	120	100	83.4	70	42.2
3	140	120	80	80	70	40
4	120	96	80	62	60	39
5	120	80	80	60	54.2	36.2
6	120	80	68	53.3	49	26.4
7	118	80	62	48.5	40	23.3
8	115	80	60	48	40	21.9
9	110	60	48	33.6	25.6	13
10	101	34	31.4	22.4	18	10.4

In accordance with Hazen's Plot method, the excess probability of each order have been computed as follows:

$$P_1 = \frac{2j - 1}{2n} = \frac{2x1 - 1}{2x10} = 0.05 = 5\%$$

$$P_2 = \frac{''}{''} \quad \frac{2x2 - 1}{2x10} = 0.15 = 15\%$$

$$P_3 = \frac{''}{''} \quad \frac{2x3 - 1}{2x10} = 0.25 = 25\%$$

$$P_4 = \frac{''}{''} \quad \frac{2x4 - 1}{2x10} = 0.35 = 35\%$$

$$P_5 = \frac{''}{''} \quad \frac{2x5 - 1}{2x10} = 0.45 = 45\%$$

$$P_6 = \frac{''}{''} \quad \frac{2x6 - 1}{2x10} = 0.55 = 55\%$$

$$P_7 = \frac{''}{''} \quad \frac{2x7 - 1}{2x10} = 0.65 = 65\%$$

$$P_8 = \frac{''}{''} \quad \frac{2x8 - 1}{2x10} = 0.75 = 75\%$$

$$P_9 = \frac{''}{''} \quad \frac{2x9 - 1}{2x10} = 0.85 = 85\%$$

$$P_{10} = \frac{2j - 1}{2n} = \frac{2x10 - 1}{2x10} = 0.95 = 95\%$$

For the least-square fitting, the above-calculated figures are plotted on probability papers for individual durations, and the straight lines are illustrated on Fig. 1.

From Fig. 1, we have obtained intensities for different durations corresponding to 3, 5 and 10 years frequencies as given in the following Table.

Table 2 Rainfall Intensities of Different Durations for 3, 5 and 10-yr return period

	5	15	30	45	60	120 (min)
5 yr	146	117	95	80	66	40
10 yr	158	140	112	96	83	50
3 yr	135	98	82	68	56	33.5

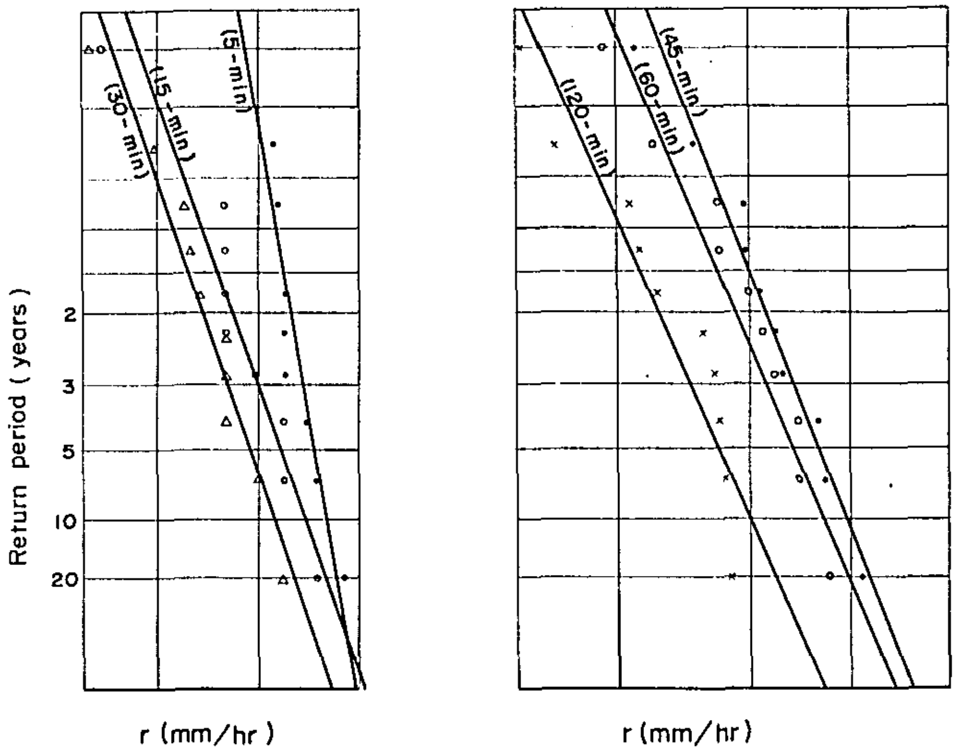
From the above table constants "a" and "b" of Talbot-type equation for 5-yr return period are then computed by using the least square method as follows:

$$a = 6,550$$
$$b = 40$$

Then the Equation is,

$$I = \frac{6,550}{t + 40}$$

Fig. I Intesity-Duration-Frequency Relationship



CHAPTER IX

DISCHARGE AT MERNUNG DAM

1. Location and Structure of the Dam.

Mernung dam is located at about 20 km upstream of the confluence of the Marmojo river and the Surabaja river. The dam was constructed in about 1920 for the purpose of serving irrigation water. Mernung dam has four gates with wooden stop logs. General view and dimensions are shown in Fig. 1. Mernung dam is one of the best stations for discharge analysis, for it is located out of innundated area of lower basin of the Marmojo river and data on water level and the number of stop logs have been recorded.

2. Interpretation of Data.

Data on water level upstream and downstream of dam are available for the following periods;

1949 (Nov.) - 1962 (Jun.)
1964 (Jan. - Apr.)
1966 (Jan.) - 1969 (Aug.)
1970 (Jan.) - 1971 (Oct.)

Number of stop logs, however, are recorded only for the following periods among the above;

1957 (Feb.) - 1961 (Dec.)
1966 (Oct.) - 1970 (Mar.)

Data on water level and number of stop logs are recorded more than three times a day.

Examining the data, the following characters have been revealed.

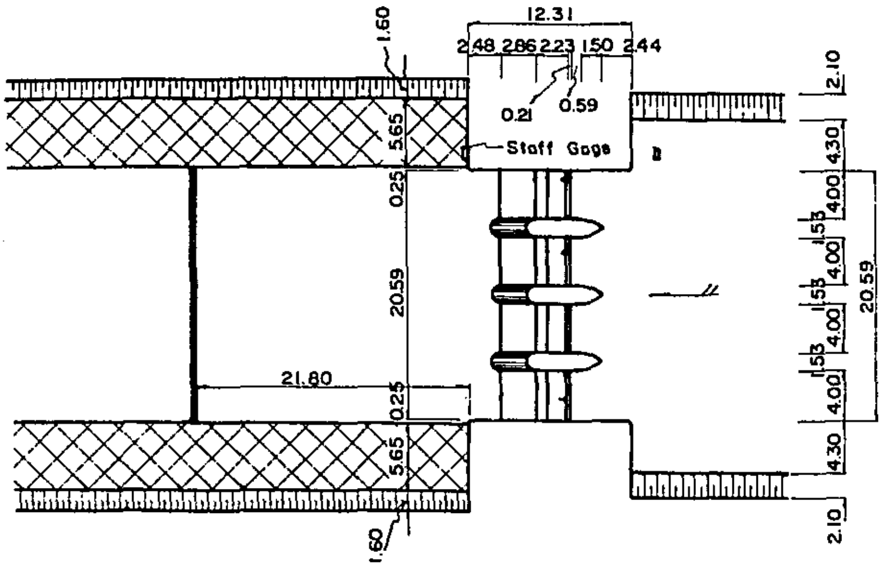
(1) Stop logs were operated at units of four logs and four gates of the dam were operated in the same way at the same time.

(2) There is no description about whether water level was recorded before the operation of gates or after it. From further examination of data on water level upstream and downstream of the dam and number of stop logs, it seems that water level upstream of the dam was recorded before the operation of stop logs and water level downstream was recorded after the operation.

(3) Water levels downstream of the dam are mostly recorded on the unit of 0.2 m. The relation between water level downstream of the dam and number of stop logs is shown in Fig. 2, which shows that water level downstream and number of stop logs have a too close relation. We interpret this fact as follows; namely, the recorder might make a convenience of a curve or a table in order to estimate downstream water level from number of lifted stop logs, for counting of number of lifted stop-logs must be easier than measurement of water level, especially in case of measurement at night. From this point of view, it is concluded that records of down-

Fig. 1 Mernung Dam

PLAN : 0 10 20m



ELEVATION ; 0 1 2 3 4 5m

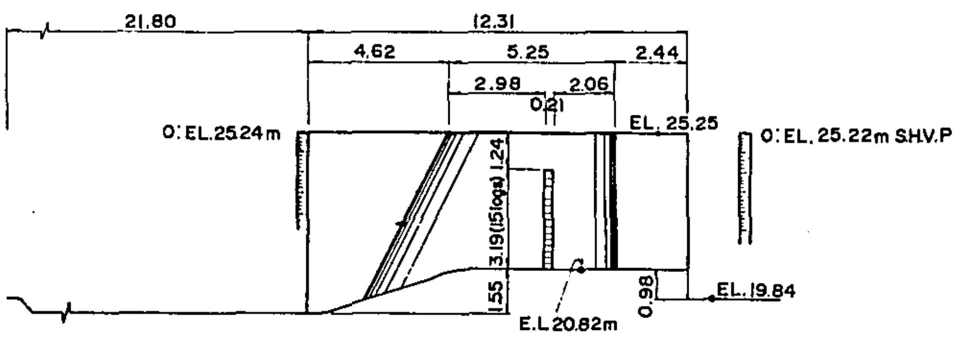
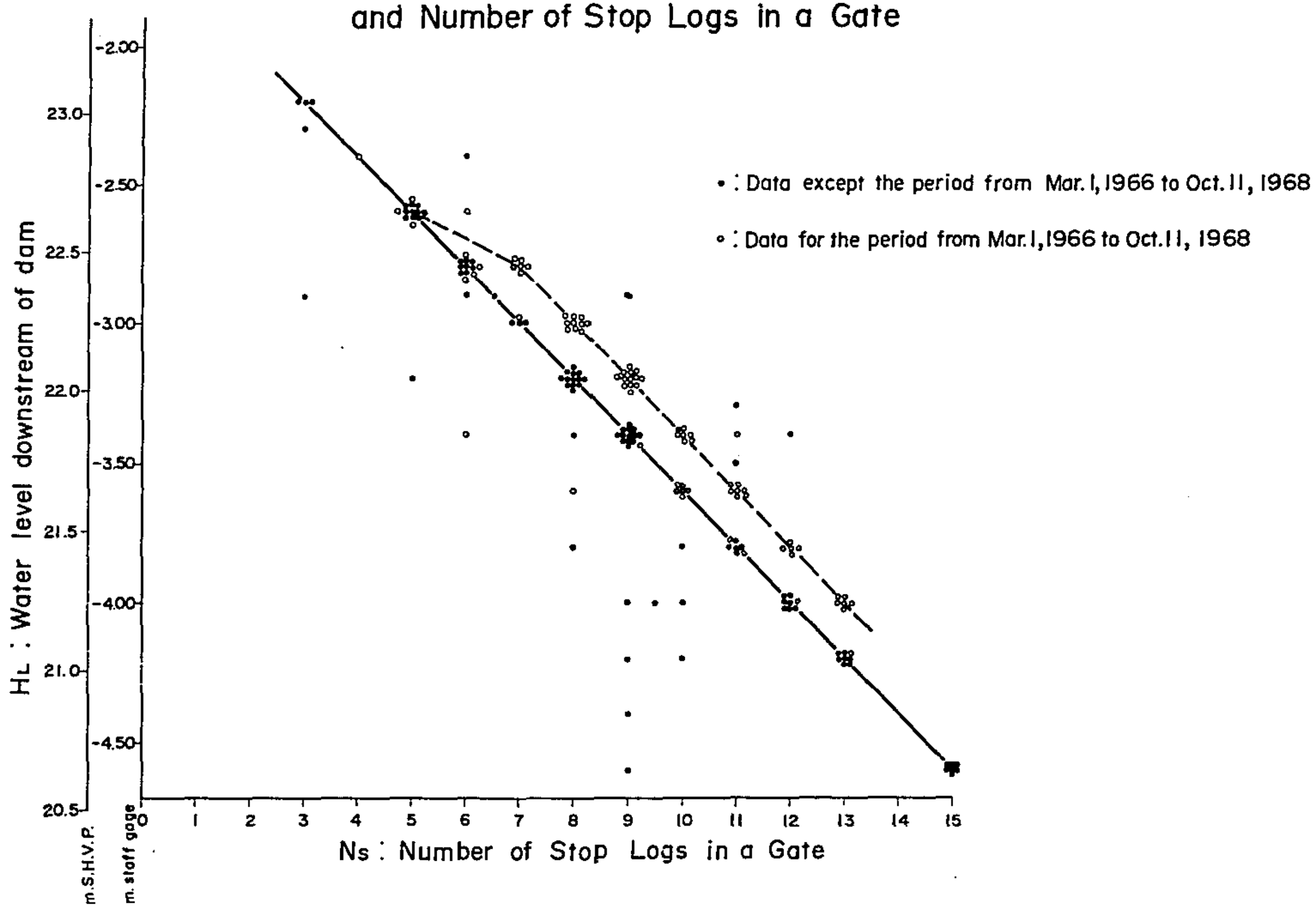


Fig. 2 Relation Between Water Level Downstream of Dam and Number of Stop Logs in a Gate



stream water level are not reliable but applicable for estimation of number of stop logs for the period when stop log data are not available.

3. Discharge Rating Curve.

Discharge rating curve is obtained as a solution of simultaneous equations shown below;

Free overflow discharge of the dam.

$$Q_o = f_1(H_u, N_s)$$

where Q_o = free overflow discharge,
H_u = water level upstream of the dam,
N_s = number of stop logs in one gate.

Stage-discharge relation at tailwater of the dam.

$$H_1 = f_2(Q)$$

where H_1 = water level downstream of the dam,
Q = discharge.

Submerged overflow discharge of the dam.

$$Q = f_3(Q_o, H_u, H_1)$$

In the three equations mentioned above, Q_o, Q and H_1 are unknown and H_u and N_s are given by data. Thus, if the functions are known, three unknowns, Q_o, Q and H_1 can be determined.

(1) Free overflow discharge from the dam.

(i) $1 \leq N_s \leq 15$

The flow can be approximated to that of sharp edged weir (Fig. 3), and empirical formula by Rouse is applicable.

$$Q'_o = CBh_1^{3/2}$$

where Q'_o = free overflow discharge (m^3/s),
B = width of weir (m),
h_1 = overflow depth (m),
C = coefficient of discharge for free overflow; empirical diagram shown in Fig. 4 is provided by Rouse,
D = height of weir (m).

Mernung dam consists of four gates of 4 m in each width and total width of weir (B) is 16 m. When water level upstream of the dam (H_u) is given, the overflow depth (h_1) is determined according to the number of stop logs per gate (N_s), which is at most 15 stop logs of which thickness is about 21 cm each.

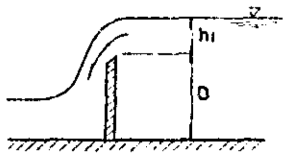


Fig. 3

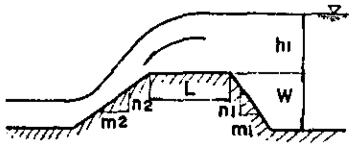


Fig. 5

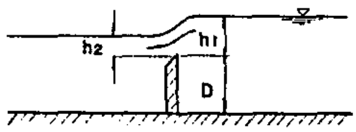


Fig. 9

Fig. 11

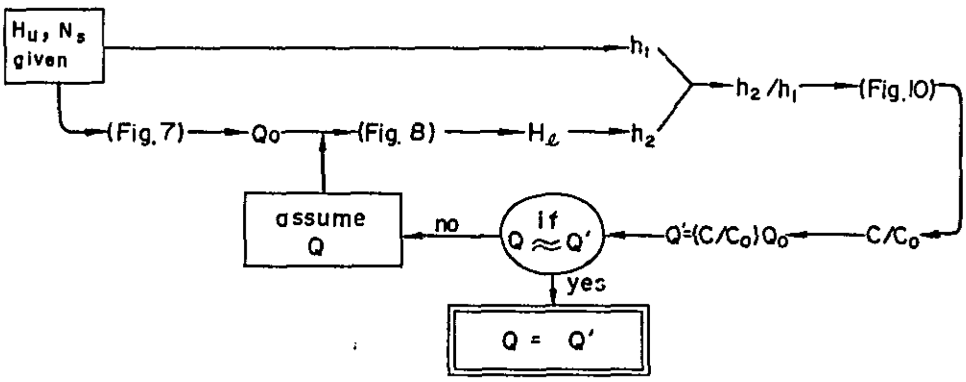


Fig. 4 Discharge Coefficient for Flow over Shurp Edged Weir

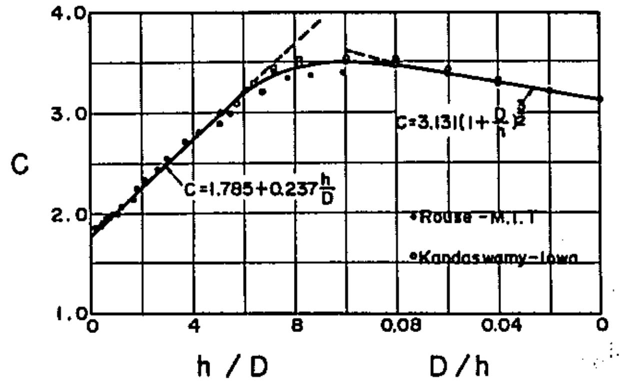
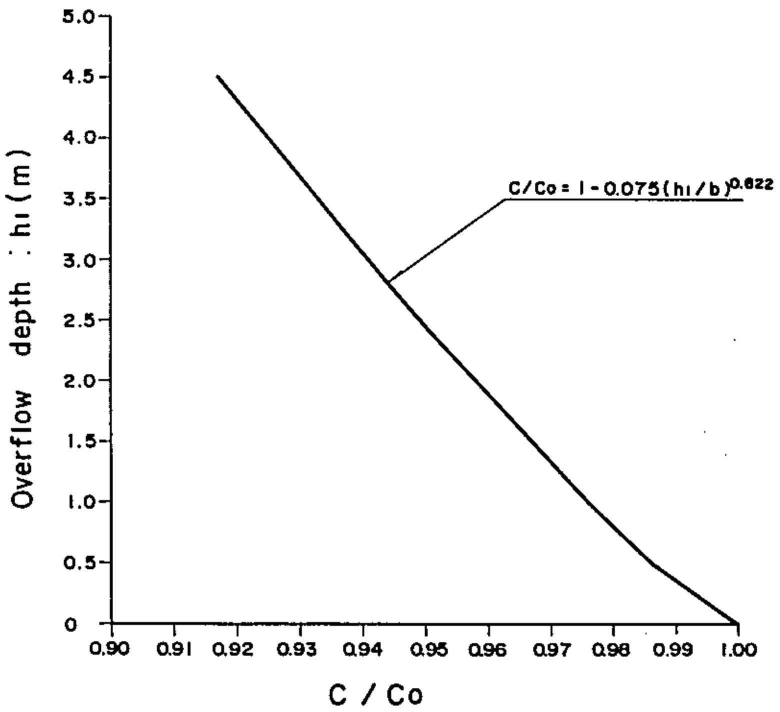


Fig. 6 Influence of Piers to Two Dimensional Flow



(ii) $N_s = 0$

The flow is considered to be that of a trapezoidal weir (Fig. 5), and empirical formula by Honma is applicable.

$$Q_o' = \mu B h_1 \sqrt{2gh_1}$$

where Q_o' = free overflow discharge (m^3/s),
B = width of weir (m),
h_1 = overflow depth (m),
μ = coefficient of discharge for free overflow; empirical formula shown below are given by Honma,

Downstream (m_2/n_2)	Upstream (m_1/n_1)	μ
less than 3/5	0 - 3/4	$0.31 + 0.23 \frac{h_1}{W}$
around 1/1	0 - 3/2	$0.29 + 0.32 \frac{h_1}{W}$
around 3/2	0 - 3/1	$0.28 + 0.37 \frac{h_1}{W}$
rectangular section $h_1/L < 1/2$		0.35

g = acceleration of gravity (= 9.8 m/sec^2),
m_1/n_1, m_2/n_2 = slope of the weir upstream side and downstream side,
W = depth of weir (m),
L = length of the weir crest (m).

Merunung dam has the following demensions.

$B = 16$ m
$m_1/n_1 = 0$, $m_2/n_2 = 0.28 < 3/5$
$W = 1$ m

Thus, the following equation is usable for the calculation of coefficient of discharge.

$$\mu = 0.31 + 0.23\ (h_1/W)$$

In case μ is larger than 1, μ must be taken as 1 in this analysis.

(iii) Influence of piers to two dimensional flow.

The above formula holds for two dimensional flow. Hence the calculation should be adjusted in consideration of the influence of piers. Empirical formula by Sato is applicable to this case.

$$C/C_o = 1 - 0.075\ \left(\frac{h_1}{b}\right)^{0.822} \quad (\text{m, sec})$$

C = coefficient of discharge adjusted
C_o = coefficient of discharge for two dimensional flow
h_1 = overflow depth (m)
b = net span between piers (m)

The equation mentioned above is shown graphically in Fig. 6, which was calculated for any overflow depth (h_1) taking net span between piers (b) as 4 m.

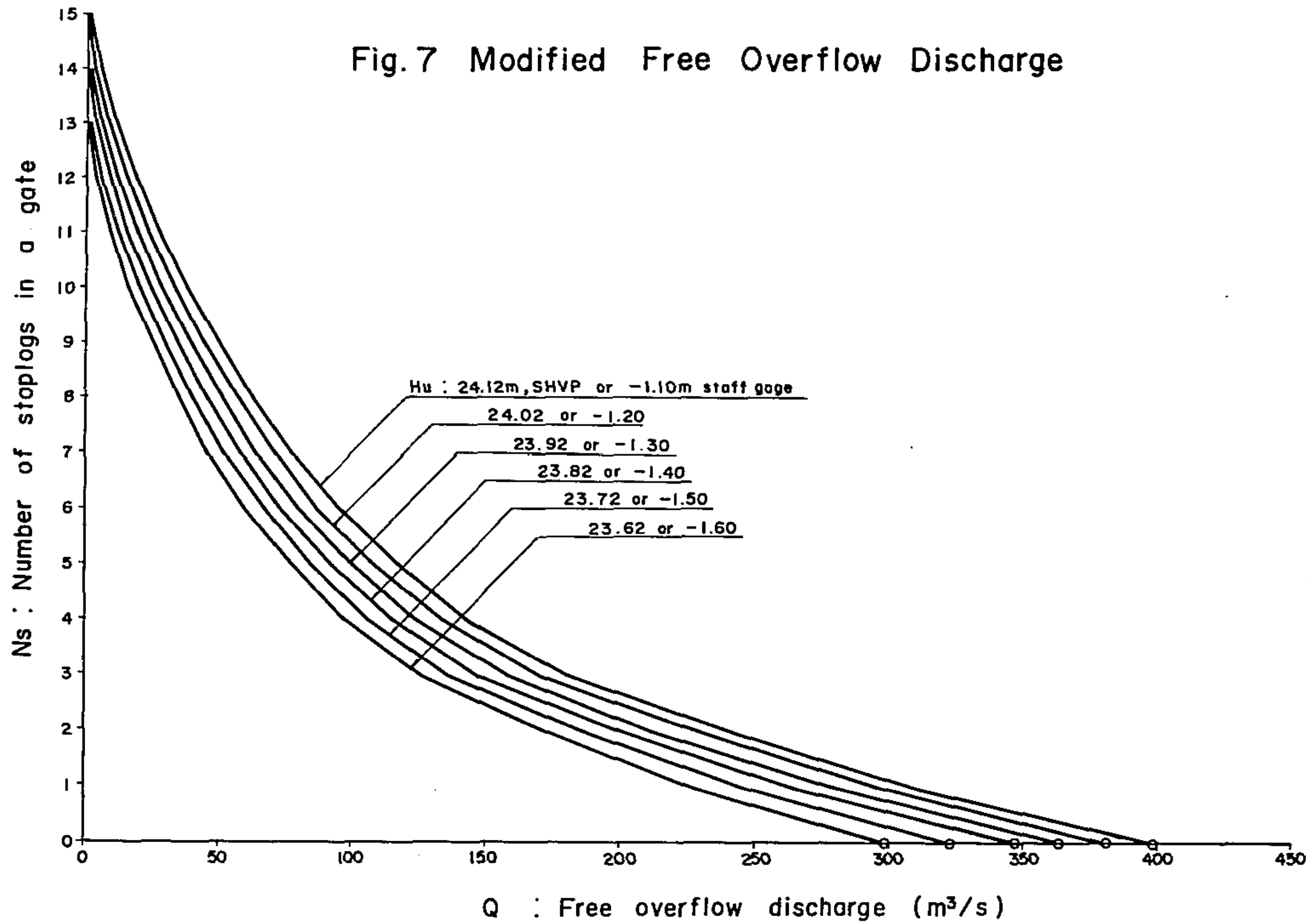
The range of water level upstream of dam (H_u) and number of stop logs (N_s) for the calculation of rating curve were taken as follows looking over the operation records;

H_u: 24.12, 24.02, 23.92, 23.82, 23.72 and 23.62 in m, SHVP (or -1.10, -1.20, -1.30, -1.40, -1.50 and -1.60 in m on staff gage upstream of dam)

N_s: from 1 to 15, and 0

Results of calculation are shown in Table 1-1, 1-2 and 2 and Fig. 7. The results of calculation for the case $N_s = 1$ were modified so that the curve for each H_u might be smooth. The modified portion of free overflow discharge is also shown in Table 3 and Fig. 7.

Fig. 7 Modified Free Overflow Discharge



(2) Stage-discharge relation at tail water of the dam.

Stage-discharge relation at tail water of the dam was obtained by non-uniform flow calculation which was made under the following conditions:

a) River channel; cross-sections which were surveyed at intervals of 100 m along the Marmojo river were used,

b) Manning's coefficient of roughness;

$n = 0.030.$

The results of calculation are given below,

discharge (m^3/s)	water level (m, SHVP)
50	21.390
100	22.736
150	23.446
200	23.970

Fig. 8 shows the relation between stage and discharge at tail water of the dam.

(3) Submerged overflow of the dam.

Submerged overflow discharge (Fig. 9) was calculated from free overflow discharge making use of Villemonte's formula.

$$\frac{Q}{Q_o} = \left\{1 - \left(\frac{h_2}{h_1}\right)^n\right\}^{0.385}$$

where Q = submerged overflow discharge (m^3/s),
Q_o= free overflow discharge (m^3/s),
h_1= upstream depth above the crest of the weir (m),
h_2= downstream depth above the crest of the weir (m),
n = constant depending on the shape of the weir.

The equation mentioned above is shown graphycally in Fig. 10, which was calculated for h_2/h_1 taking constant n as 1.5.

Making use the curves shown in Fig. 7, 8 and 10, the rating curves of the dam were made by trial and error method shown in Fig. 11.

The results are shown in Table 4 and Fig. 12. Overflow discharge of Mernung dam can be determined from Fig. 12 if a water level upstream of the dam (H_u) and number of stop logs per gate (N_s) are given.

4. Probability of Annual Maximum Discharge.

In order to find the maximum discharge in a year, data where water level upstream of the dam (H_u) was higher than 23.62 m SHVP and number of stop logs per gate (N_s) was less than 10 were first picked up and converted

Fig.8 Stage-Discharge Relation at Tail Water of Dam

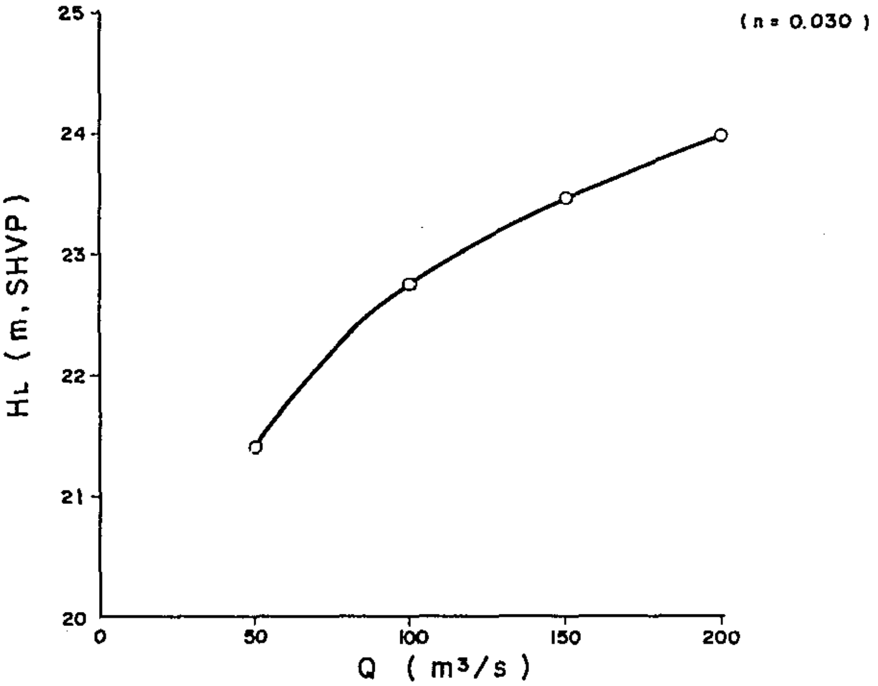
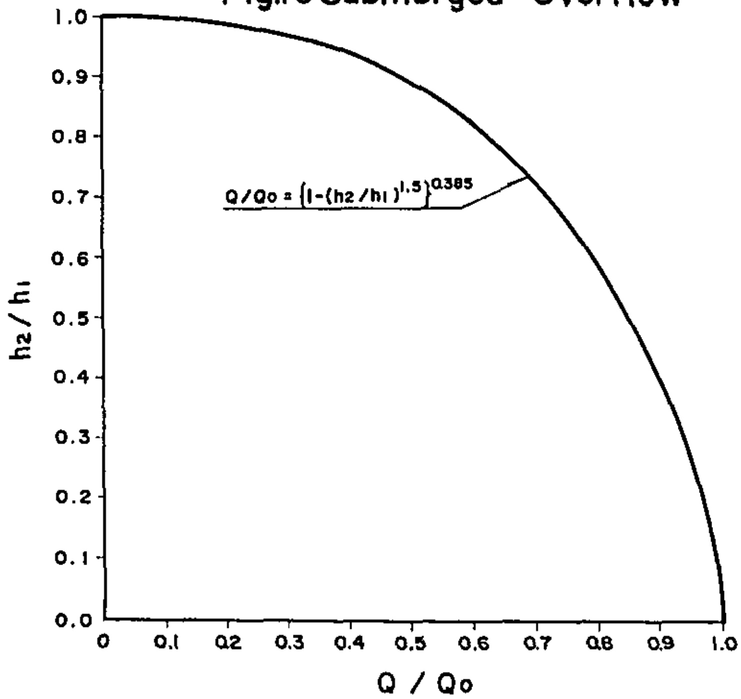


Fig.10 Submerged Overflow



into discharge making use of Fig. 11.

The maximum discharge in each year was selected out of them as shown in Table 5. The annual maximum discharges in 1962, 1964 and 1971 were selected from the data including some lacks, but examination of rainfall records of those years verified that they were the maximums.

In Table 5, the exceedance probability is expressed by Thomas as follows.

$$P_r = \frac{i}{(n + 1)}$$

P_r = probability of exceedance
n = number of data
i = order of magnitude

Discharge and its exceedance probability were plotted on a logarithmic probability paper shown in Fig. 13. From this figure, discharge for each return period was obtained as follows.

Return period (years)	Discharge (m^3/s)
50	190
20	166
10	149
5	130
2	101

Fig.12 Overflow Discharge of Mernung Dam

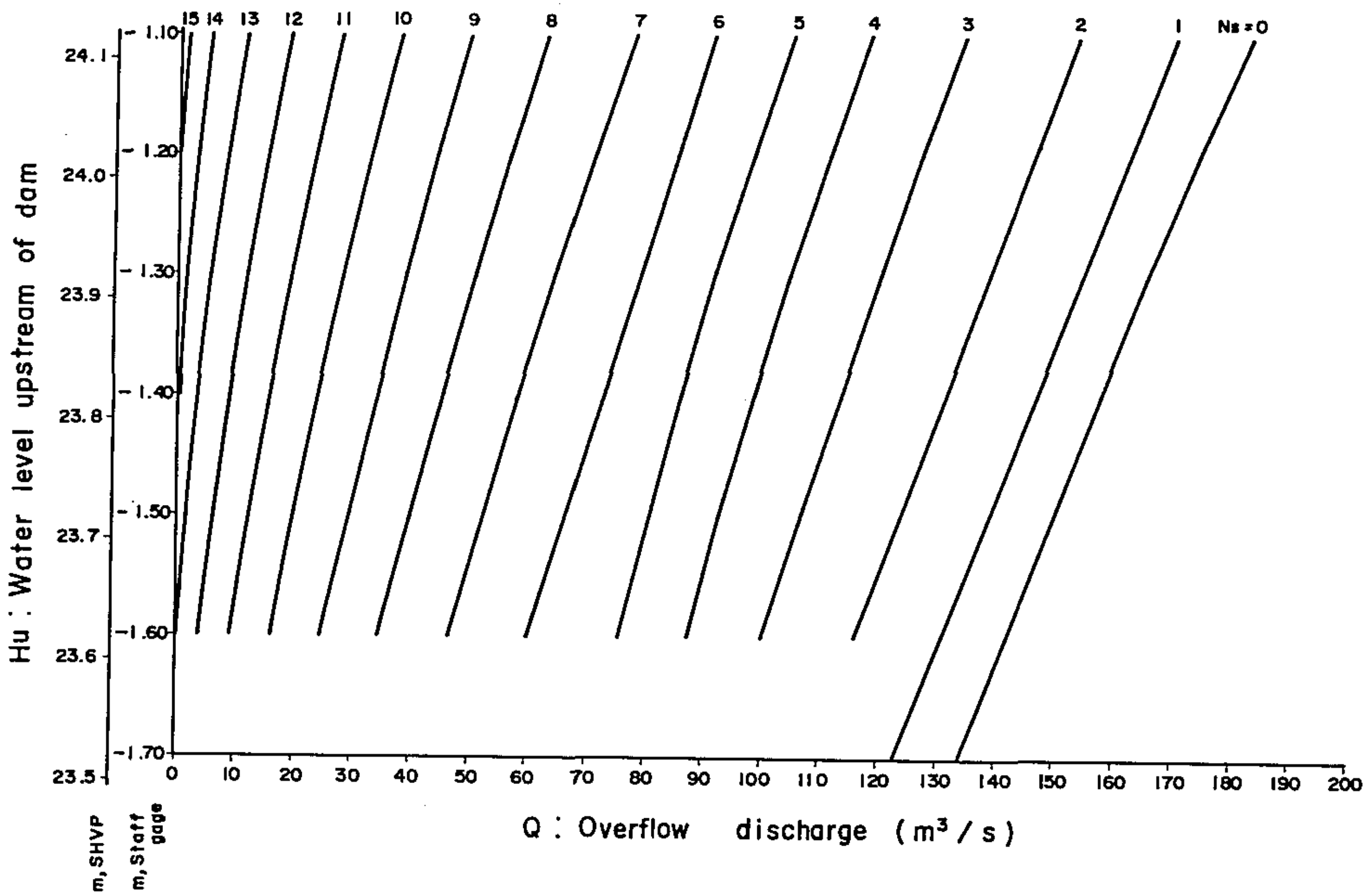


Fig.13 Return-Period of Discharge at Mernung Dam

(by Thomas plot)

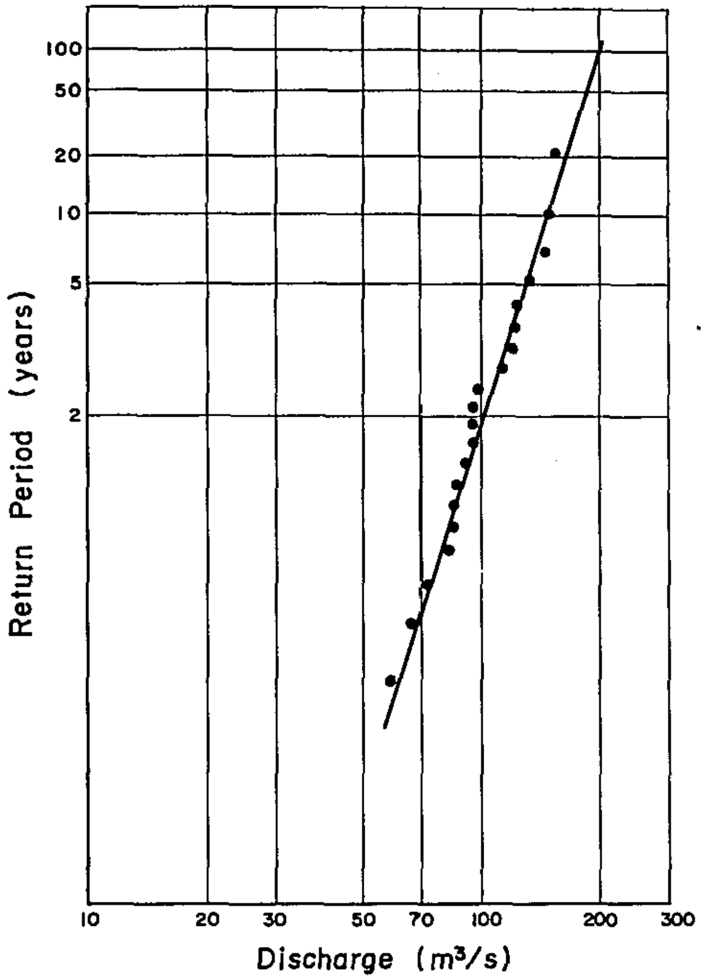


Table 1-1 Calculation of Free Overflow Discharge

($1 \leq N_s \leq 15$)

WL	N_s	D (m)	H_D (m, SHVP)	h_1 (m)	(D/h_1) h_1/D	C_o	Q_o' (m^3/s)	C/C_o	Q_o (m^3/s)
H_u=24.12m, SHVP(-1.10m on staff gage)	1	0.21	21.03	3.09	(0.068)	3.4	295.5	0.939	277.5
	2	0.43	21.25	2.87	6.67	3.3	256.7	0.943	242.1
	3	0.64	21.46	2.66	4.16	2.77	192.3	0.946	181.9
	4	0.85	21.67	2.45	2.88	2.47	151.5	0.950	143.9
	5	1.06	21.88	2.24	2.11	2.29	122.8	0.954	117.2
	6	1.28	22.10	2.02	1.58	2.16	99.2	0.957	94.9
	7	1.49	22.31	1.81	1.215	2.07	80.6	0.961	77.5
	8	1.70	22.52	1.60	0.941	2.01	65.1	0.965	62.8
	9	1.91	22.73	1.39	0.728	1.96	51.4	0.969	49.8
	10	2.13	22.95	1.17	0.549	1.92	38.9	0.973	37.8
	11	2.34	23.16	0.96	0.410	1.88	28.3	0.977	27.6
	12	2.55	23.37	0.75	0.294	1.85	19.2	0.981	18.8
	13	2.77	23.59	0.53	0.191	1.83	11.3	0.985	11.1
	14	2.98	23.80	0.32	0.107	1.81	5.2	0.990	5.1
	15	3.19	24.01	0.11	0.035	1.79	1.1	0.995	1.1
H_u=24.02m, SHVP(-1.20m on staff gage)	1	0.21	21.03	2.99	(0.070)	3.4	281.2	0.940	264.3
	2	0.43	21.25	2.77	6.44	3.3	243.4	0.945	230.6
	3	0.64	21.46	2.56	4.00	2.73	178.9	0.948	169.6
	4	0.85	21.67	2.35	2.76	2.44	140.7	0.952	133.9
	5	1.06	21.88	2.14	2.02	2.26	113.2	0.955	108.1
	6	1.28	22.10	1.92	1.55	2.14	91.1	0.959	87.4
	7	1.49	22.31	1.71	1.148	2.06	73.7	0.963	71.0
	8	1.70	22.52	1.50	0.882	1.99	58.5	0.967	56.6
	9	1.91	22.73	1.29	0.675	1.94	45.5	0.971	44.2
	10	2.13	22.95	1.07	0.502	1.90	33.6	0.974	32.7
	11	2.34	23.16	0.86	0.368	1.87	23.8	0.979	23.3
	12	2.55	23.37	0.65	0.255	1.85	15.5	0.983	15.2
	13	2.77	23.59	0.43	0.155	1.82	8.2	0.987	8.1
	14	2.98	23.80	0.22	0.074	1.80	3.0	0.993	3.0
	15	3.19	24.01	0.01	0.003	1.79	0.0	0.996	0
H_u=23.92m, SHVP(-1.30m on staff gage)	1	0.21	21.03	2.89	(0.073)	3.4	267.3	0.943	252.1
	2	0.43	21.25	2.67	6.21	3.2	223.4	0.946	211.3
	3	0.64	21.46	2.46	3.84	2.7	166.6	0.950	153.8
	4	0.85	21.67	2.25	2.65	2.41	130.1	0.953	124.0
	5	1.06	21.88	2.04	1.925	2.24	104.4	0.957	99.9
	6	1.28	22.10	1.82	1.422	2.12	83.3	0.961	80.1
	7	1.49	22.31	1.61	1.081	2.04	66.7	0.965	64.4
	8	1.70	22.52	1.40	0.824	1.98	52.5	0.969	50.9
	9	1.91	22.73	1.19	0.623	1.93	40.1	0.972	39.0
	10	2.13	22.95	0.97	0.455	1.89	28.9	0.976	28.2
	11	2.34	23.16	0.76	0.325	1.86	19.7	0.981	19.3
	12	2.55	23.37	0.55	0.216	1.84	12.0	0.985	11.8
	13	2.77	23.59	0.33	0.119	1.81	5.5	0.990	5.4
	14	2.98	23.80	0.12	0.040	1.79	1.2	0.995	1.2
	15	3.19	24.01	-	-	-	-	-	-

Table 1-2 Calculation of Free Overflow Discharge

($1 \leqq N_s \leqq 15$)

WL	Ns	D (m)	H_D (m,SHVP)	h_1 (m)	(D/h_1) h_1/D	C_o	Q'_o (m^3/s)	C/C_o	Q_o (m^3/s)
H_u=23.82m,SHVP(-1.40m on staff gage)	1	0.21	21.03	2.79	(0.075)	3.4	253.5	0.944	239.3
	2	0.43	21.25	2.57	5.98	3.20	210.9	0.948	200.0
	3	0.64	21.46	2.36	3.69	2.66	154.3	0.952	146.9
	4	0.85	21.67	2.15	2.53	2.38	120.0	0.955	114.6
	5	1.06	21.88	1.94	1.830	2.22	96.0	0.959	92.1
	6	1.28	22.10	1.72	1.344	2.10	75.8	0.963	73.0
	7	1.49	22.31	1.51	1.013	2.03	60.3	0.967	58.3
	8	1.70	22.52	1.30	0.765	1.97	46.7	0.970	45.3
	9	1.91	22.73	1.09	0.571	1.92	35.0	0.975	34.1
	10	2.13	22.95	0.87	0.408	1.88	24.4	0.979	23.9
	11	2.34	23.16	0.66	0.282	1.85	15.9	0.983	15.6
	12	2.55	23.37	0.45	0.177	1.83	8.8	0.987	8.7
	13	2.77	23.59	0.23	0.083	1.80	3.2	0.992	3.2
	14	2.98	23.80	0.02	0.007	1.79	0.1	0.999	0.1
	15	3.19	24.01	-	-	-	-	-	-
H_u=23.72m,SHVP(-1.50m on staff gage)	1	0.21	21.03	2.69	(0.078)	3.5	247.1	0.946	233.8
	2	0.43	21.25	2.47	5.74	3.15	195.7	0.950	185.9
	3	0.64	21.46	2.26	3.53	2.62	142.4	0.953	135.7
	4	0.85	21.67	2.05	2.41	2.36	110.9	0.957	106.1
	5	1.06	21.88	1.84	1.736	2.20	87.8	0.961	84.4
	6	1.28	22.10	1.62	1.266	2.09	69.0	0.965	66.6
	7	1.49	22.31	1.41	0.946	2.01	53.8	0.968	52.1
	8	1.70	22.52	1.20	0.706	1.95	41.0	0.972	39.9
	9	1.91	22.73	0.99	0.518	1.91	30.1	0.976	29.4
	10	2.13	22.95	0.77	0.362	1.87	20.2	0.980	19.8
	11	2.34	23.16	0.56	0.293	1.85	12.4	0.985	12.2
	12	2.55	23.37	0.35	0.137	1.82	6.0	0.989	5.9
	13	2.77	23.59	0.13	0.047	1.80	1.4	0.995	1.4
	14	2.98	23.80	-	-	-	-	-	-
	15	3.19	24.01	-	-	-	-	-	-
H_u=23.62m,SHVP(-1.60m on staff gage)	1	0.21	21.03	2.59	(0.081)	3.5	233.4	0.948	221.3
	2	0.43	21.25	2.37	5.51	3.09	180.3	0.951	171.5
	3	0.64	21.46	2.16	3.38	2.59	131.6	0.955	125.7
	4	0.85	21.67	1.95	2.29	2.33	101.5	0.959	97.3
	5	1.06	21.88	1.74	1.64	2.17	79.7	0.963	76.8
	6	1.28	22.10	1.52	1.19	2.07	62.1	0.966	60.0
	7	1.49	22.31	1.31	0.879	1.99	47.8	0.970	46.4
	8	1.70	22.52	1.10	0.647	1.94	35.8	0.974	34.9
	9	1.91	22.73	0.89	0.466	1.90	25.5	0.978	24.9
	10	2.13	22.95	0.67	0.315	1.86	16.3	0.983	16.0
	11	2.34	23.16	0.46	0.197	1.83	9.1	0.987	9.0
	12	2.55	23.37	0.25	0.098	1.81	3.6	0.993	3.6
	13	2.77	23.59	0.03	0.011	1.79	0.1	0.998	0.1
	14	2.98	23.80	-	-	-	-	-	-
	15	3.19	24.01	-	-	-	-	-	-

Table 2 Calculation of Free Overflow Discharge at $N_s = 0$

H_u (m,SHVP)	h_l (m)	h_l/W	μ	C_o	Q_o' (m^3/s)	C/C_o	Q_o (m^3/s)
24.12	3.30	3.3	1.07 (1.00)	4.43	425.0	0.936	397.8
24.02	3.20	3.2	1.05 (1.00)	4.43	405.7	0.938	380.5
23.92	3.10	3.1	1.02 (1.00)	4.43	386.9	0.940	363.3
23.82	3.00	3.0	1.00	4.43	368.3	0.941	346.6
23.72	2.90	2.9	0.98	4.34	343.0	0.943	323.4
23.62	2.80	2.8	0.95	4.21	315.5	0.944	297.8

Table 3 Free Overflow Discharge for Any Value of N_s

N_s	Modified free overflow discharge (m^3/s)					
	H_u=24.12	H_u=24.02	H_u=23.92	H_u=23.82	H_u=23.72	H_u=23.62
0	397.8	380.5	363.3	346.6	323.4	297.8
1	308.9	296.1	279.2	264.3	246.0	227.5
2	242.1	230.0	211.3	200.0	185.9	171.5
3	181.9	169.6	158.3	146.9	135.7	125.7
4	143.9	133.9	124.0	114.6	106.1	97.3
5	117.2	108.1	99.9	92.1	84.4	76.8
6	94.9	87.4	80.1	73.0	66.6	60.0
7	77.5	71.0	64.4	58.3	52.1	46.4
8	62.8	56.6	50.9	45.3	39.9	34.9
9	49.8	44.2	39.0	34.1	29.4	24.9
10	37.8	32.7	28.2	23.9	19.8	16.0
11	27.6	23.3	19.3	15.6	12.2	9.0
12	18.8	15.2	11.8	8.7	5.9	3.6
13	11.1	8.1	5.4	3.2	1.4	0.1
14	5.1	3.0	1.2	0.1	-	-
15	1.1	0	-	-	-	-

Table 4 Overflow Discharge of Mernung Dam

N_s	H_u=24.12	H_u=24.02	H_u=23.92	H_u=23.82	H_u=23.72	H_u=23.62	
0	183.2	175.1	165.6	157.7	150.1	141.6	Submerged overflow
1	170.4	162.3	154.2	146.2	138.7	130.5	
2	153.7	146.2	139.1	131.9	123.7	115.9	
3	134.6	126.9	120.2	113.3	106.5	100.0	
4	117.9	111.2	104.6	98.5	92.4	87.1	
5	104.2	98.4	91.5	86.0	80.5	75.5	
6	91.2	84.9	79.2	72.8	66.6	60.0	Free overflow
7	77.5	71.0	64.4	58.3	52.1	46.4	
8	62.8	56.6	50.9	45.3	39.9	34.9	
9	49.8	44.2	39.0	34.1	29.4	24.9	
10	37.8	32.7	28.2	23.9	19.8	16.0	
11	27.6	23.3	19.3	15.6	12.2	9.0	
12	18.8	15.2	11.8	8.7	5.9	3.6	
13	11.1	8.1	5.4	3.2	1.4	0.1	
14	5.1	3.0	1.2	0.1	-	-	
15	1.1	0	-	-	-	-	

Table 5 Annual Maximum Discharge and Its Exceedance Probability by Thomas Plot

Date				H_u	H_l	N_s	Q (m^3/s)	Order i	Exceedance probability	Lack of data
Year	Mon.	Day	Time							
1950	Feb.	11	8	1.50	2.00	2	123.3	5	0.238	
1951	Feb.	20	6	1.30	1.85	1.5	146.6	3	0.143	
1952	Mar.	17	18	1.43	1.57	0	155.2	1	0.048	
1953	Apr.	9	12	1.50	1.48	0	149.8	2	0.095	
1954	Feb.	24	12	1.54	2.00	2	120.3	6	0.286	
1955	Mar.	4	12	1.45	2.60	5	83.3	17	0.810	
1956	Jun.	6	12	1.25	2.60	5	95.0	11	0.524	
1957	Jan.	4	12	1.20	2.60	5	98.2	9	0.429	
1958	Apr.	2	18	1.25	2.60	5	95.0	11	0.524	
1959	Dec.	19	18	1.30	2.30	3	120.0	7	0.333	
1960	Feb.	26	6	1.40	2.20	3	113.2	8	0.381	
1961	Feb.	16	6	1.20	2.80	6	85.2	15	0.714	
1962	Apr.	17	12	1.50	2.80	6	66.5	19	0.905	Jul.1 - Dec.31
1964	Mar.	21	12	1.40	2.80	6	72.6	18	0.857	Apr.27 - Dec.31
1966	Nov.	24	6	1.40	2.60	5	86.0	14	0.667	
1967	Jan.	16	6	1.40	2.80	7	58.3	20	0.952	
1968	May	16	18	1.45	2.40	4	95.5	10	0.476	
1969	Feb.	20	6	1.42	2.60	5	85.0	16	0.762	
1970	Mar.	19	6	1.30	2.60	5	91.6	13	0.619	
1971	Jan.	21	18	1.40	2.00	2	131.0	4	0.190	Oct.23 - Dec.31

CHAPTER X

DISCHARGE OF THE WATUDAKON RIVER

1. Location and Structure of Watudakon Syphon.

Watudakon syphon crosses the Brantas river at a point about 6.5 km upstream from Mlirip sluice. The syphon was constructed around 1920 and has been serving to pass the runoff from the Watudakon river basin. Outline of the structure is shown in Fig. 1.

2. Upper Limit of Discharge Through the Syphon.

The structure of the syphon restricts to pass the runoff from the Watudakon river basin of which area is 99.4 km^2, and upper limit of discharge through the syphon ought to be the maximum discharge which may run into the Surabaja river.

The upper limit of discharge through the syphon was calculated according to considerations on the discharge through the syphon and that in the channel downstream of the syphon.

(1) Discharge through the syphon.

The discharge through the syphon (Q_s) may be calculated by the following equations.

$$Q_s = CA_s \sqrt{2g(H_u - H_\ell)}$$
$$C = 1/\sqrt{(1 + f_e + 2gn^2 l/R_s^{4/3}}$$

where

C = coefficient of discharge,
A_s= cross-sectional area of flow,
g = acceleration of gravity (= 9.8 m/s^2),
H_u,H_ℓ = water level at the entrance and the exit of the syphon,
f_e= coefficient of entrance loss,
n = Manning's coefficient of roughness of the syphon,
l = length of the syphon,
R_s= hydraulic mean depth of the syphon.

According to dimensions of the syphon, A_s = 8.13 m^2 (for one channel), R_s = 0.740 m and l = 170 m. Considering the structure of the syphon f_e and n were determined at 0.2 and 0.018 respectively. Thus, the equations mentioned above will be

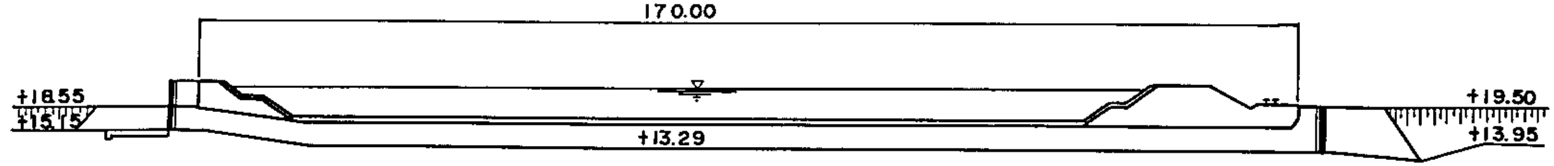
$$C = 1/\sqrt{1 + 0.2 + 1.61} = 0.595$$
$$Q_s = 0.595 \times (3 \times 8.13) \times \sqrt{19.6}\,(H_u - H_\ell)^{1/2} = 64.1\,(H_u - H_\ell)^{1/2}$$

(2) Discharge of the downstream channel.

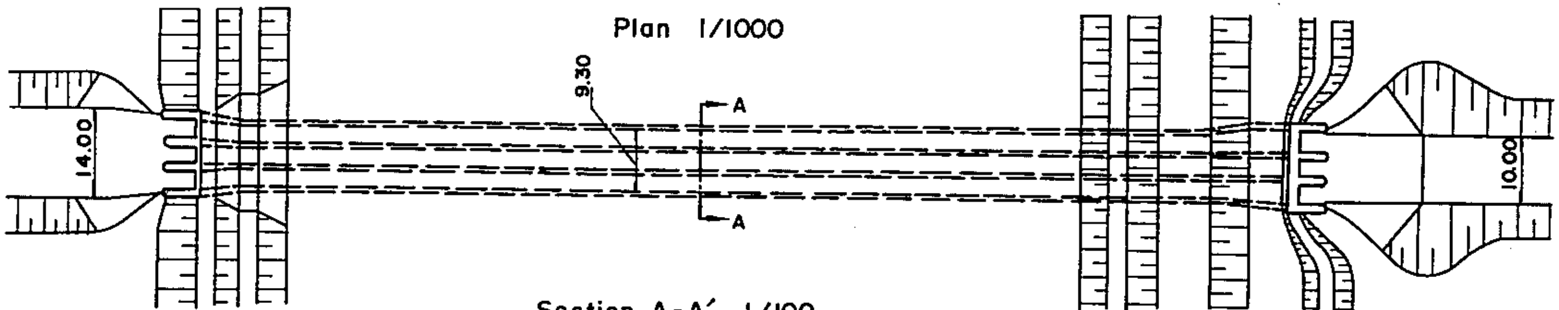
The flow in the channel downstream of Watudakon syphon was first assumed to be uniform and the lateral profile at the exit of the syphon

Fig. 1 Watudakon Syphon

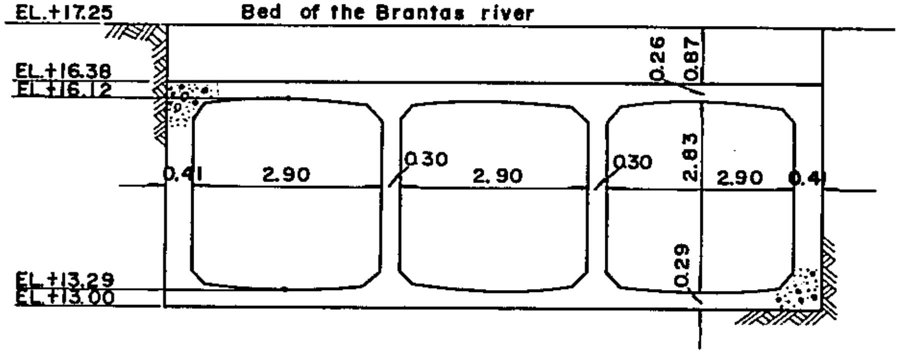
Longitudinal Profile 1/1 000



Plan 1/1000



Section A-A′ 1/100



was used in the calculation of the discharge in the said channel. The river bed slope (I) was estimated at. $I = 1/4,670$ according to lateral profiles of the Watudakon river at the exit of the syphon and joining point to the Surabaja river, which are the only points the lateral profiles are available. Considering the condition of the river, Manning's coefficient of roughness was taken as $n = 0.025$. Hence, discharge (Q) of the channel is expressed by.

$$Q = \frac{1}{n} AR^{2/3} I^{1/2} = 0.586\ AR^{2/3}$$

where

A = cross-sectional area (m^2),
R = hydraulic mean depth (m) (= A/P),
P = wetted perimeter (m).

The values of A and R will be calculated if water level (H) is given.

(3) Upper limit of discharge through the syphon.

Taking water level upstream of the syphon H_u = 18.55 m SHVP which is the same as the elevation of the dike, the discharge through the syphon can be expressed by the following equation.

$$Q_s = 64.1\ (18.55 - H_\ell)^{1/2}$$

H_ℓ(m,SHVP)	18.55	18.30	18.05	17.80	17.55	17.05	16.55
$H_u - H_\ell$(m)	0	0.25	0.50	0.75	1.00	1.50	2.00
Q (m^3/s)	0	32.1	45.3	55.5	64.1	78.5	90.7

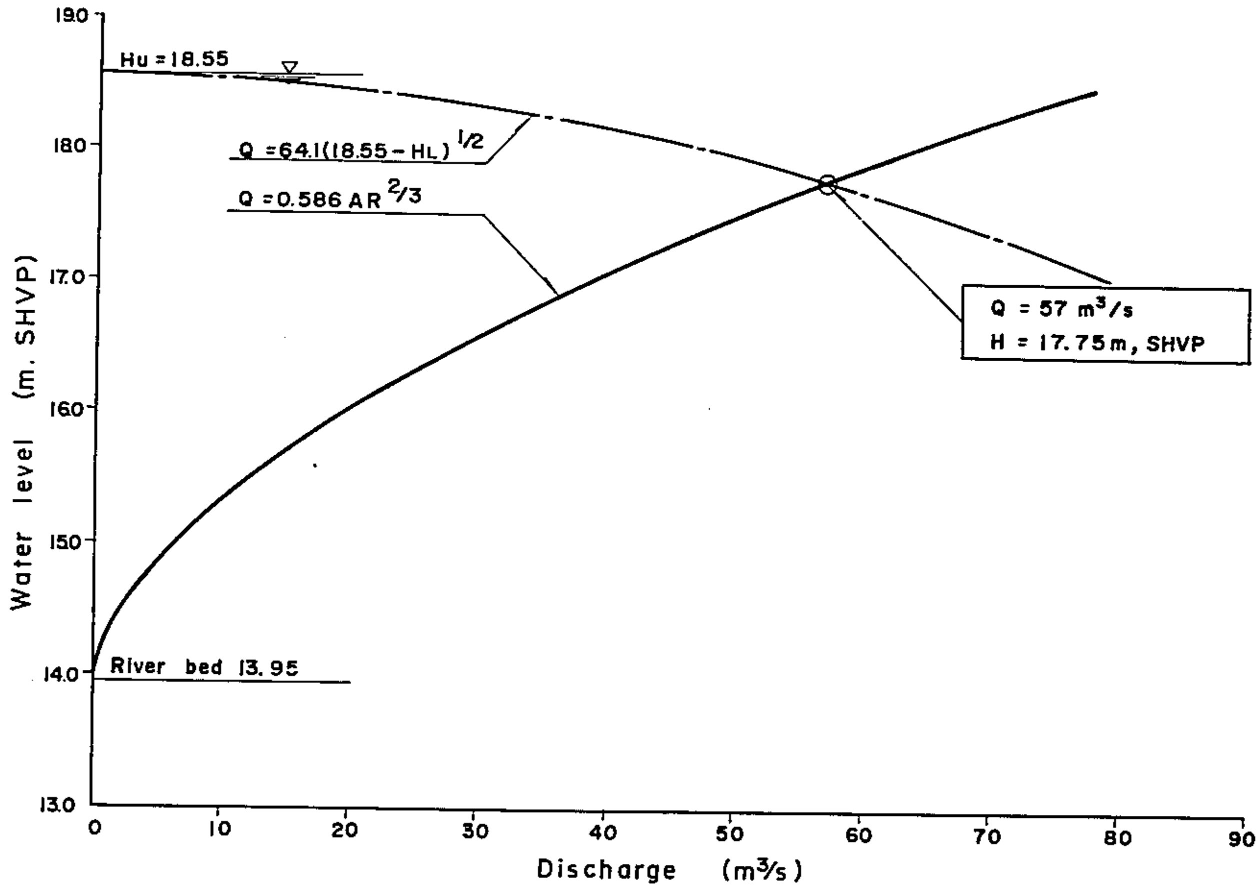
On the other hand, the values of water level and discharge mentioned above must satisfy the following equation of discharge of the downstream channel.

$$Q = 0.586 . AR^{2/3}$$

H (m,SHVP)	13.95	15.95	16.45	16.95	17.45	17.95	18.45
Depth (m)	0	2.0	2.5	3.0	3.5	4.0	4.5
A (m^2)	0	24.02	31.28	39.05	47.31	56.08	65.35
$R^{2/3}$	0	1.328	1.496	1.645	1.779	1.904	2.019
Q (m^3/s)	0	18.7	27.4	37.6	49.2	62.5	77.2

Namely, these equations should be solved simultaneously, for instance using a graphical method as shown in Fig. 2. Thus, the upper limit of discharge was estimated at 60 m^3/s, which ought to be the maximum discharge of the Watudakon river.

Fig. 2 Upper Limit of Discharge Through Watudakon Syphon



CHAPTER XI

FLOOD DISCHARGES OF THE MAS RIVER

1. Drainage Basin of the Mas River.

This river originates in the Darmo area and pours into sea at Udjung. Its drainage area is 13.8 km^2 and the river length from Wonokromo sluice to the river mouth is 14.4 km. The basin is divided into six subbasins shown in Fig. 1 and their dimensions are shown in Table 1.

Table 1 Drainage Basin of the Mas River

Name of subbasin	Area (km^2) Left	Right	Total	Cumulative	Name of runoff point	River length (m) of each subbasin
Darmo	2.1	0.15	2.25	2.25	Darmo	2500
Kupang	2.6	0.45	3.05	5.30	Kupang	1500
Gubeng	0.36	0.64	1.00	6.30	Gebeng dam	1500
Ngemplak	2.9	0.70	3.60	9.90	Ngemplak	2000
Merah br.	1.4	0.70	2.10	12.00	Merah br.	3500
River mouth	0.9	0.90	1.80	13.80	River mouth	4500

br.: Bridge

2. Discharges at Six Points on the River.

In the Darmo subbasin, the length of the main drainage canal, say Darmo canal, is estimated at 2,500 m. A branch canal located around the upstream end of the main canal is about 300 m in length. If we assume after some trial operation that the propagation velocity in this branch canal is 0.3 m/s, the propagation time of the branch canal is computed at 17 min. Assuming that the inlet time to this branch canal is 8 min, we get 25 min as the concentration time at the confluence to Darmo canal. Next, we assume the mean propagation velocity in Darmo canal at 0.5 m/s after some trial computation, then the propagation time of Darmo canal is calculated at 83 min. Consequently, the concentration time at the confluence to the Surabaja river is calculated at 108 min.

According to the rainfall intensity-duration-frequency curve at Surabaja which is found in another chapter, the 10-year rainfall intensity corresponding to the duration 108 min is estimated at 55 mm/hr. Then the peak runoff at the confluence, say Darmo, is estimated at 20 m^3/s using the rational formula, where the runoff coefficient is assumed at 0.6.

Next, we estimate the 10-year peak runoff at the lower end of the Kupang subbasin on the Mas river, say Kupang. According to the results of nonuniform flow along the Mas river for some flood discharges in the design channel, the average velocity over the whole length of the river is estimated at about 0.95 m/s. Therefore, the mean propagation velocity is estimated at 1.4 m/s after Kleitz-Seddon. The length of the reaches from Darmo to Kupang on the Mas river is 1,500 m. Consequently, the propagation time for the above reaches is computed at 18 min. Therefore, the concentration time at Kupang becomes 108 + 18 = 126 min.

Fig.1 Drainage Basin of the Mas River

Since the 10-year rainfall intensity corresponding to the above concentration time 126 min is estimated at 49 mm/hr according to the same intensity-duration-frequency curve of rainfall mentioned above, the 10-year peak runoff at Kupang is calculated at 43 m^3/s using the rational formula and assuming the runoff coefficient at 0.6 similarly to the above case.

In the same manner, the 10-year peak-runoff discharges were calculated at the points of Gubeng dam, Ngemplak, Merah bridge, and river mouth. Similarly, the 5-year peak-runoff discharges were also calculated, where the average propagation velocity was assumed at 1.2 m/s and the runoff coefficient was assumed at 0.6. These peak discharges at six points on the Mas river are listed up in Table 2 together with the corresponding specific discharges.

Table 2 Flood Discharges of the Mas River

Name of section	Return period			
	10 years		5 years	
	Discharge (m^3/s)	Spc. dis. ($m^3/s/km^2$)	Discharge (m^3/s)	Spc. dis. ($m^3/s/km^2$)
Darmo	20	8.9	16	7.1
Kupang	43	8.2	34	6.3
Gubeng dam	46	7.3	36	5.7
Ngemplak	63	6.3	50	5.0
Merah br.	64	5.3	50	4.2
River mouth	62	4.5	46	3.3

br.: Bridge
Spc. dis.: Specific discharge

CHAPTER XII

RELATION BETWEEN RUNOFF AND DRAINAGE AREA

1. Relation between Estimated 10-year and 5-year Runoffs and Drainage Area of the Mas River.

In another chapter, the flood discharges at the six points on the Mas river, i.e. at Darmo, Kupang, Gubeng dam, Ngemplak, Merah bridge, and river mouth were estimated for 10 and 5-year return period using the rational method. For the purpose of studying what relation there is between the flood discharges Q and drainage area A for each return period, $\sqrt{A}$ and $Q/\sqrt{A}$ were computed and shown in Table 1.

Table 1 Flood Discharges and Drainage Area of the Mas River

Location	Drainage area $A(km^2)$	$\sqrt{A}$	Return period 10-year		Return period 5-year	
			Discharge $Q(m^3/s)$	K $(Q/\sqrt{A})$	Discharge $Q(m^3/s)$	K $(Q/\sqrt{A})$
Darmo	2.25	1.50	20	13.3	16	10.7
Kupang	5.30	2.30	43	18.7	34	14.8
Gubeng dam	6.30	2.51	46	18.3	36	14.3
Ngemplak	9.90	3.15	63	20.0	50	15.9
Merah bridge	12.00	3.46	64	18.5	50	14.4
River mouth	13.80	3.61	62	17.2	46	12.7
Mean				17.7		13.8

The case of 10-year discharges are shown in Fig. 1, which indicates that the value of $Q/\sqrt{A}$ is nearly constant. This was the same with the case of 5-year discharges. Therefore, it can be seen that the relationship

$$Q = K\sqrt{A} \qquad K = \text{constant}$$

holds good between drainage area and discharge for a return period. The values of constant K were 17.7 and 13.8 for 10-year and 5-year return period.

2. Relation between the Runoff at Mernung Dam and That at the Confluence of the Marmojo River.

Since the discharges at Mernung dam and Perning are known from the records of their water stages, we can find the relation between the two discharges at Mernung and Perning or the relation between discharge and corresponding catchment area, provided that the discharges from Mlirip sluice, the Watudakon river, and the Wonoaju river can be deducted from the discharge at Perning.

For this purpose, we collected the simultaneous discharges at Perning Mlirip, Gedeg, and Kedungsumur (on the Watudakon river) when the discharges at Gedeg were nearly zero or less than 10 m^3/s. These are shown in Table 2 where the discharges at Perning, Mlirip, and Gedeg were read from their respective rating curve by using the records of water stages, while the records of discharges were used for Kedungsumur.

Table 2

Date	Discharge (m^3/s)						
	(1) Perning	(2) Mlirip	(3) Gedeg	(4) Kd.sumur	(1)-(2)-(3)-(4) Q_{mar}	Q_{mer}	$\frac{Q_{mar}}{Q_{mer}}$
1954 Apr. 29	251	88	0	17.9	145	117.6	1.23
May 23	259	119	0	9.9	130	94.9	1.37
Dec. 2	288	119	9	31.9	128	113.2	1.13
1955 Mar. 4	237	85	0	11.3	141	83.3	1.7
1956 Feb. 27	298	131	0	24.6	142	83.3	1.71
Jun. 6	260	94	0	29.1	137	95.0	1.44
1959 Mar. 11	317	175	1	41.1	100	92.0	1.12
Dec. 19	317	135	1	52.1	216	120.0	1.80
1960 Mar. 21	320	182	0	43.1	95	61.0	1.56
Feb. 16	252	80	0	70.5	101	63.0	1.60
Mar. 12	219	129	0	21.1	69	65.0	1.25
1961 Feb. 16	251	80	0	70.5	100	91.4	1.09
1962 Apr. 17	251	118	0	34.2	99	66.5	1.49
1964 Mar. 26	283	142	0	36.8	104	71.1	1.47
1970 Mar. 19	278	137	2	35.8	103	91.6	1.13
Mean							1.40

The Watudakon river and the Wonoaju river join the Surabaja river just before the Marmojo river joins. However, the joining discharge from the Wonoaju to the Surabaja is presumed to be null during the major period of the Marmojo's flood. Therefore, if we deduct the discharges at Mlirip and Kedungsumur from the discharges at Perning, we can find the discharges from the Marmojo river and the Kuwangah river which joins the Surabaja river closely upstream of Perning after the Marmojo river has joined the Surabaja river. This discharge is denoted by Q_{mar} and the discharge at Mernung dam is denoted by Q_{mer} in Table 2.

Out of the data on discharges mentioned above, we have selected the data that Q_{mar} is larger than Q_{mer} and Q_{mer} is larger than 50 m^3/s. These data are shown in Table 2 and the ratios of Q_{mar}/Q_{mer} are plotted against Q_{mer} in Fig. 2. It can be easily seen from this figure that the mean value of Q_{mar}/Q_{mer} is 1.40.

If we assume $Q_{mar} = K_1\sqrt{A_{mar}}$ and $Q_{mer} = K_2\sqrt{A_{mer}}$ where A_{mer} is the drainage area upstream from Mernung dam and A_{mar} is the drainage area which equals to the sum of A_{mer}, the residual drainage area of the Marmojo river except the drainage area of the Wonoaju river, and the drainage area of the Kuwangah river, we get the following equation.

$$\frac{K_1}{K_2} = \frac{\left(\frac{Q_{mar}}{Q_{mer}}\right)}{\left(\frac{A_{mar}}{A_{mer}}\right)^{\frac{1}{2}}}$$

In this equation, A_{mar} = 302.1 km^2 and A_{mer} = 155.1 km^2, hence we get

$$\frac{K_1}{K_2} = \frac{1.40}{1.41} \fallingdotseq 1$$

Therefore, it can be concluded that the relation

$$Q = K\sqrt{A}$$

with K constant holds in this basin.

Fig. 1

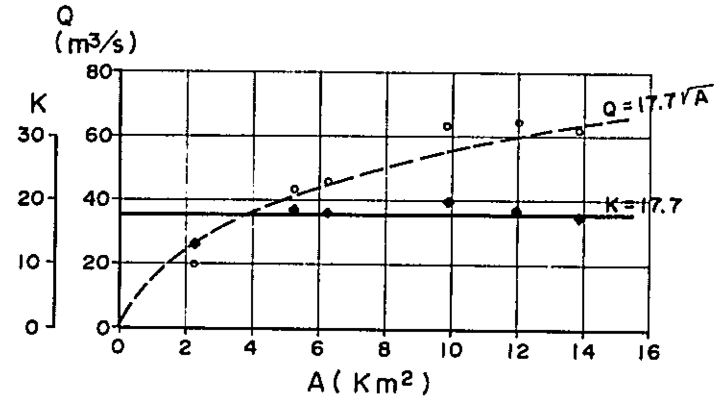
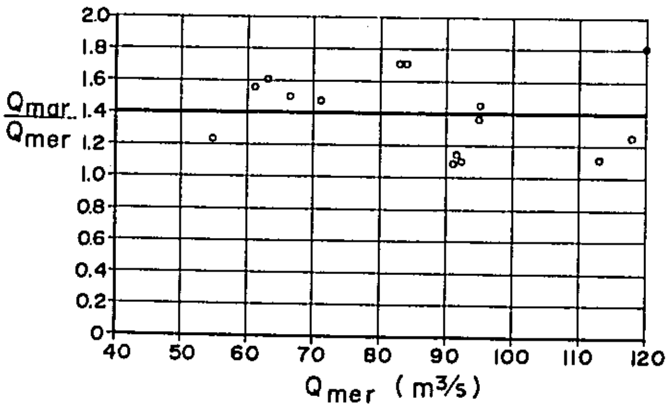


Fig. 2



CHAPTER XIII

FLOOD DISCHARGES OF THE MARMOJO AND SURABAJA RIVERS

In case the flood discharge of the Brantas river is not allowed to branch to the Surabaja river through Gedeg and Mlirip sluices, the Marmojo river is regarded as the main stream of the Surabaja river, and the Watudakon river is regarded as one of tributaries. The subbasins of the Marmojo/Surabaja river in this meaning are shown in Fig. 1 and the drainage area of respective subbasin is shown in Table 1. According to this table, the total drainage area of the Surabaja river is 604.4 km^2.

When a big flood occurs on the Surabaja river, we can assume that the relation

$$Q = K\sqrt{A}$$

holds in this basin in accordance with the previous study. A tributary, the Kedurus river or the Rawa river which flows between Gunungsari hill and Kebraon hill and joins the Surabaja river closely downstream of Gunungsari dam has no contribution to the flood discharge of the Surabaja river because the flood water of the Surabaja river rather goes into the Kedurus river during the flood. On the other hand, the flood discharge from the Watudakon river is presumed to contribute constantly to the discharge of the Surabaja river because of the flatness of its basin.

Therefore, the flood discharges on the Surabaja river can be estimated by $Q = K\sqrt{A}$ for the basin except the Watudakon one. Since the 50-year discharge at Mernung dam is estimated at 190 m^3/s from the probability curve which has already been studied, we can estimate the 50-year discharges at Sidogede, Perning, Krikilan, and Sepandjang on the basis of the discharge 190 m^3/s at Mernung dam. The result of runoff estimation is shown in Table 2 and Fig. 2, where it is assumed, from the viewpoint of safety, that the maximum runoff from the Watudakon river constantly joins the discharge of the Surabaja river.

In the same manner, the 50-year and 20-year runoffs from the subbasins of the Marmojo river have been estimated and shown in Fig. 3 and Table 3.

Fig. I Subbasins of the Marmojo-Surabaja River

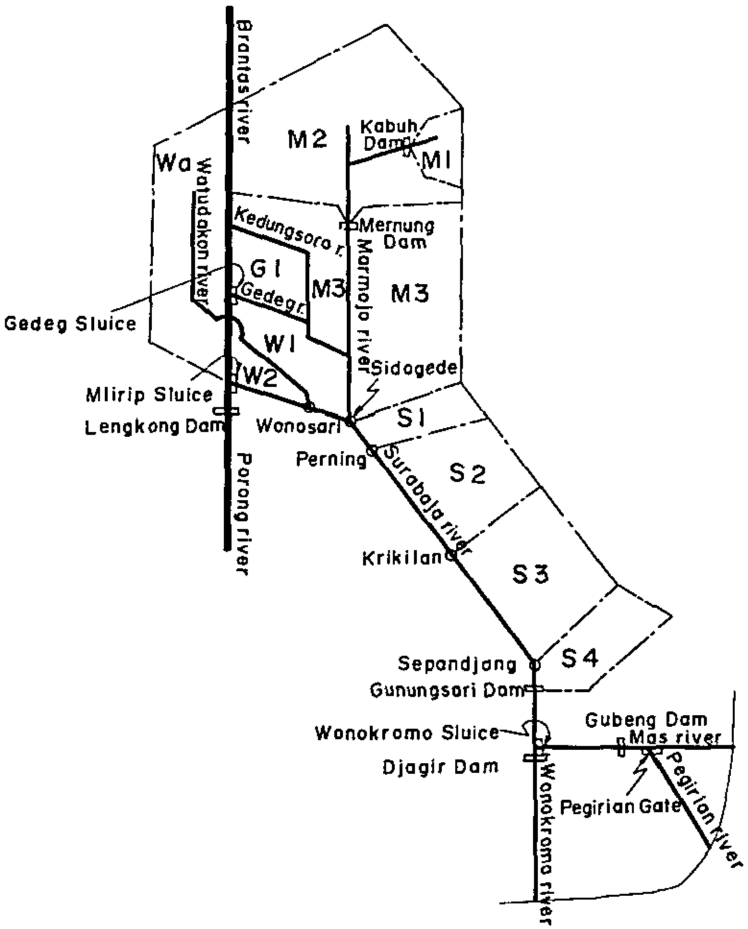


Table I Drainage Area

Name of river	Name of basin	Drainage area (km^2)	
		A_i	ΣA_i
Marmojo river	M 1	28.2	28.2
	M 2	126.9	155.1
	M 3	123.6	278.7
Gedeg river	G 1	11.0	289.7
Watudakon river	W a	99.4	—
Wonoaju river	W 1	22.2	311.9
	W 2	8.0	319.9
Surabaja river	S 1	12.4	332.3
	S 2	64.4	396.7
	S 3	37.2	433.9
	S 4	71.1	505.0
Total		604.4	—

Table 2 Estimation of Discharge Except Run-off from the Watudakon River

Name of the Place under consideration	Drainage area (km^2)			Discharge (m^3/s)	Remarks
	name of drainage	Ai	ΣAi		
Mernung Dam	M1+M2	155.1	155.1	190	
Sidogede	M3+G1	134.6	289.7	260	Confluence of the Marmojo
Perning	S1	12.4	302.1	265	
Krikilan	S2	64.4	366.5	292	
Sepandjiang	S3	37.2	403.7	306	
River mouth	—	0	403.7	306	

Fig.2 Result of Runoff Analysis

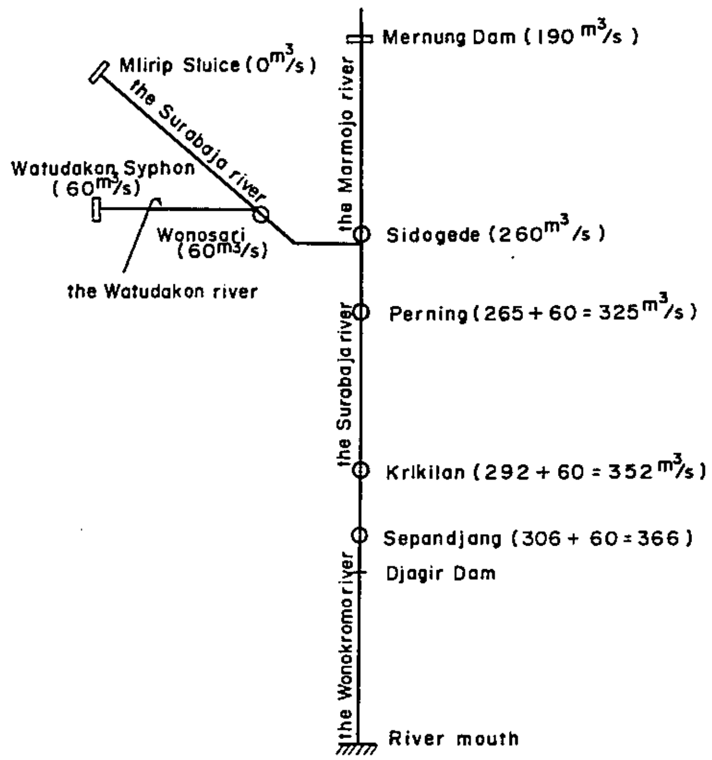
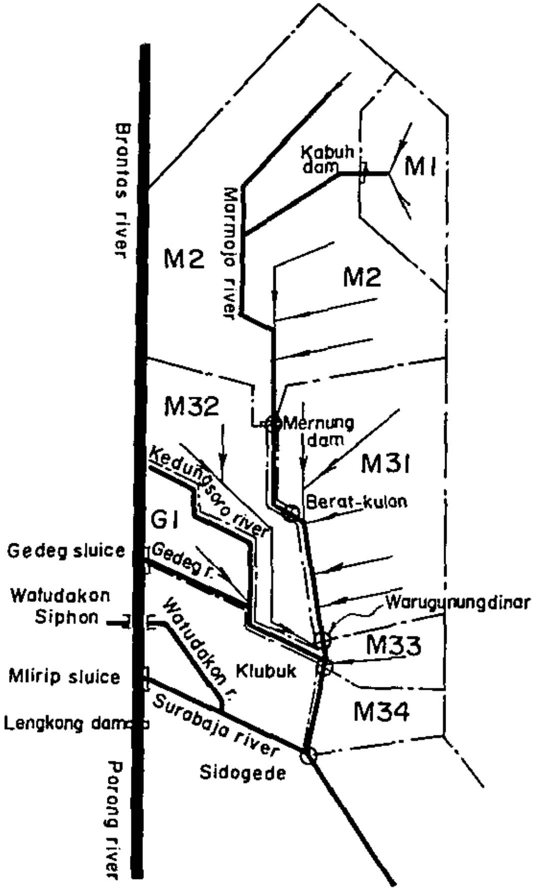


Fig. 3 Drainage Area of the Marmojo River



Name of basin	Drainage area (km^2)
M1	28.2
M2	126.9
M31	81.4
M32	20.3
M33	9.3
M34	12.6
G1	11.0
Total	289.7

Table. 3 Estimation of Discharge

Name of the place under consideration	Drainage area (km^2)			Discharge (m^3/s)	
	Name of drainage	Ai	ΣAi	1/50	1/20
	M1+M2	155.1			
Mernung dam			155.1	190	166
		0			
Berat-kulon			155.1	190	166
	M31	81.4			
Warugunungdinar			236.5	235	205
	M32+M33+G1	40.6			
Klubuk			277.1	254	222
	M34	12.6			
Sidogede			289.7	260	227

CHAPTER XIV

MOROKREMBANGAN BOEZEM

1. Discharge Curve of Morokrembangan Gate.

(1) Stage-volume curve of the existing Morokrembangan boezem.

Fig. 1 shows the storage-capacity curves of the existing Morokrembangan boezem which consists of two boezems, South Boezem and North Boezem. These curves were made by using planimetric map, longitudinal profile and cross-sections which were provided by DPPDT.

(2) Out-flow discharge through Morokrembangan gate.

Morokrembangan gate shown in Fig. 2 consists of 3 spans of mitre gates which open with the difference between inner and outer water levels or with the discharge through the opening. Therefore, the outflow discharge may be expressed by the following equation

$$Q = C\alpha B h_2 \sqrt{2g(H - H_s)}$$

where C = coefficient of discharge (estimated at 0.9 in this case)
H = inner water level of the boezem (m, SHVP)
H_s = outer water level of the boezem or tide level (m, SHVP)
h_1, h_2 = water depths inside and outside of the gate, shown in Fig. 2 (m)
g = acceleration of gravity (9.8 m/s^2)
B = whole width of three gates fully open (5m x 3 = 15m)
α = degree of opening of gates

Rewriting the above equation, we get

$$Q = \alpha Q_o$$

where
$$\left.\begin{aligned} Q_o &= 59.8 h_2 \sqrt{\Delta H} \\ \Delta H &= H - H_s \end{aligned}\right\} \qquad (1)$$

and the nature of α which shows the degree of opening of gates should be studied.

Since no rainfall was recorded during August in 1970, inflow from the Greges river was estimated at nearly null by the end of that month. Therefore, the records of variation of water level of the boezem are presumed to indicate outflow or leakage inflow through mitre gates. From the records during the period from 24th to 28th August 1970, outflow and inflow discharges were calculated as in Table 1 by using Fig. 1.

Table 1 Water Levels Inside and Outside of the Gates and Discharges

Dt.	Hr.	H (m)	H_s (m)	$\triangle H$ (m)	V (m^3)	Q (m^3/s)	Dt.	Hr.	H (m)	H_s (m)	$\triangle H$ (m)	V (m^3)	Q (m^3/s)
24.	6	1.67	0.70	-0.97	92.5	0		6	1.83	0.50	-1.33	70.0	0
	7	"	0.85	-0.72	"	0		7	"	0.40	-1.43	"	0
	8	1.66	0.99	-0.67	93.5	-1.0		8	1.82	0.50	-1.32	71.0	-1.0
	9	"	1.15	-0.51	"	0		9	"	0.80	-1.02	"	0
	10	"	1.26	-0.40	"	0		10	"	0.95	-0.87	"	0
	11	"	1.45	-0.21	"	0		11	1.81	1.25	-0.56	73.0	-2.0
	12	1.65	1.55	-0.10	95.0	-1.5		12	"	1.55	-0.26	"	0
	13	"	1.62	-0.03	"	0		13	1.82	1.70	-0.12	71.0	2.0
	14	1.66	1.68	0.02	93.5	1.5		14	"	1.84	0.02	"	0
	15	1.68	1.70	0.02	90.0	3.5		15	1.83	1.87	0.04	70.0	1.0
	16	1.69	1.65	-0.04	89.0	1.0		16	1.85	1.91	0.06	67.0	3.0
	17	"	1.55	-0.14	"	0		17	1.89	1.95	0.06	61.0	6.0
	18	1.68	1.40	-0.28	90.0	-1.0		18	1.95	1.90	-0.05	53.0	8.0
	19	"	1.30	-0.38	"	0		19	"	1.85	-0.10	"	0
	20	1.69	1.25	-0.44	89.0	1.0		20	"	1.80	-0.15	"	0
	21	"	1.20	-0.49	"	0		21	1.94	1.76	-0.18	54.0	-1.0
	22	"	1.12	-0.57	"	0		22	"	1.53	-0.41	"	0
	23	1.68	1.05	-0.63	90.0	-1.0		23	"	1.47	-0.47	"	0
	24	"	0.99	-0.69	"	0		24	1.93	1.28	-0.65	56.0	-2.0
25.	1	1.67	0.92	-0.75	92.5	-2.5	27.	1	"	1.20	-0.73	"	0
	2	"	0.84	-0.83	"	0		2	1.92	1.05	-0.87	58.0	-2.0
	3	1.66	0.76	-0.90	93.5	-1.0		3	"	0.86	-1.06	"	0
	4	1.65	0.67	-0.98	95.0	-1.5		4	1.91	0.72	-1.19	59.0	-1.0
	5	"	0.58	-1.07	"	0		5	"	0.60	-1.31	"	0
	6	"	0.60	-1.05	"	0		6	1.90	0.50	-1.40	60.0	-1.0
	7	"	0.75	-0.90	"	0		7	"	0.55	-1.35	"	0
	8	"	0.86	-0.79	"	0		8	"	0.60	-1.30	"	0
	9	1.67	0.99	-0.68	92.5	2.5		9	"	0.79	-1.11	"	0
	10	"	1.20	-0.47	"	0		10	"	0.98	-0.92	"	0
	11	"	1.35	-0.32	"	0		11	1.89	1.17	-0.72	61.0	-1.0
	12	"	1.50	-0.17	"	0		12	"	1.40	-0.49	"	0
	13	"	1.69	0.02	"	0		13	"	1.65	-0.24	"	0
	14	1.69	1.75	0.06	89.0	3.5		14	1.88	1.79	-0.09	63.0	-2.0
	15	1.78	1.83	0.05	77.0	12.0		15	"	1.85	-0.03	"	0
	16	1.83	1.87	0.04	70.0	7.0		16	1.89	1.94	0.05	61.0	2.0
	17	1.85	1.89	0.04	67.0	3.0		17	1.92	1.98	0.06	58.0	3.0
	18	1.87	1.70	-0.17	64.0	3.0		18	1.94	1.99	0.05	54.0	4.0
	19	"	1.60	-0.27	"	0		19	1.98	1.95	-0.03	49.0	5.0
	20	"	1.50	-0.37	"	0		20	"	1.79	-0.19	"	0
	21	"	1.55	-0.32	"	0		21	"	1.70	-0.28	"	0
	22	1.86	1.46	-0.40	66.0	-2.0		22	1.97	1.69	-0.29	50.0	-1.0
	23	"	1.35	-0.51	"	0		23	"	1.57	-0.40	"	0
	24	1.85	1.20	-0.65	67.0	-1.0		24	"	1.46	-0.51	"	0
26.	1	"	1.15	-0.70	"	0	28.	1	1.96	1.35	-0.61	52.0	-2.0
	2	1.84	0.96	-0.88	69.0	-2.0		2	"	1.15	-0.81	"	0
	3	"	0.72	-1.12	"	0		3	1.95	0.92	-1.03	53.0	-1.0
	4	1.83	0.60	-1.23	70.0	-1.0		4	"	0.86	-1.09	"	0
	5	"	0.55	-1.28	"	0		5	"	0.75	-1.20	"	0

Dt.: Date
Hr.: Hour
H, H_s: Elevation measured downwards from the zero of SHVP

Fig. 1 Storage-Capacity Curve of Existing Morokrembangan Boezem

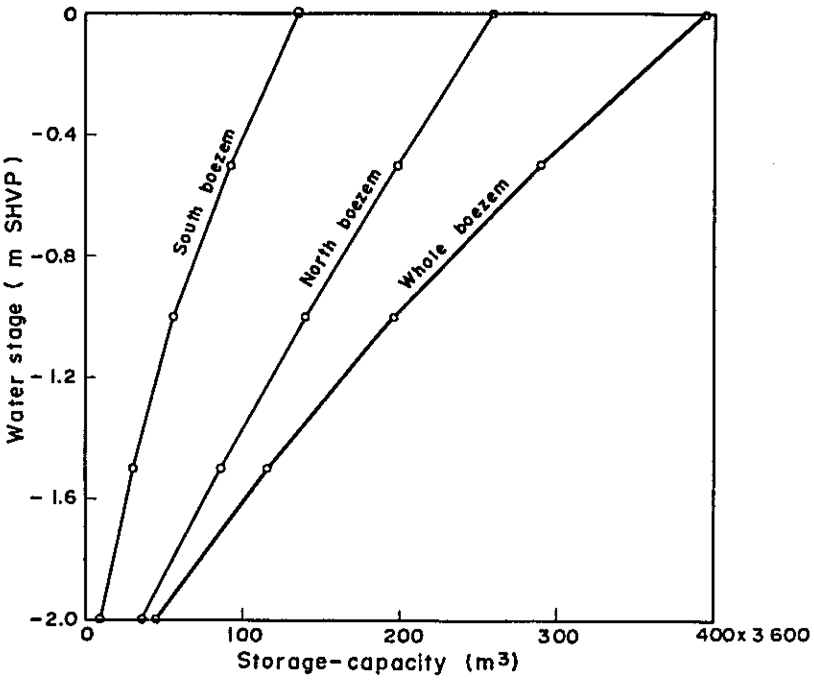
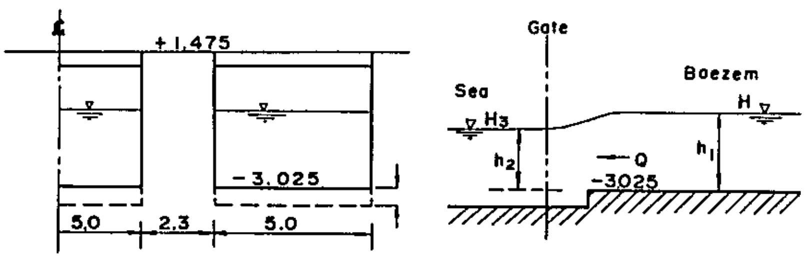


Fig. 2 Morokembangan Gate



Outflows calculated in Table 1 are shown in Fig. 3, where hourly outflow discharges are modified and smoothened considering that the outflow should occur only for the period when inner water level is higher than outer one.

Using the modified hourly discharges and the equation (1), values of α were calculated as shown in Table 2.

Table 2 Value of α

h_2 (m)	ΔH (m)	Q_o (m^3/sec)	Q (m^3/sec)	α
1.345	0.02	11.37	4.2	0.369
1.325	0.02	11.20	4.0	0.357
1.275	0.06	18.67	10.3	0.555
1.195	0.05	15.98	7.5	0.469
1.155	0.04	13.81	5.6	0.405
1.135	0.04	13.57	5.1	0.378
1.185	0.02	10.02	3.0	0.299
1.155	0.04	13.81	5.8	0.420
1.115	0.06	16.33	7.9	0.484
1.075	0.06	15.74	7.1	0.451
1.085	0.05	14.51	5.6	0.386
1.045	0.06	15.30	6.9	0.451
1.035	0.05	13.84	5.4	0.390

If we assume α to be a function of only Q_o, α can be expressed by the following equation as seen in Fig. 4.

$$\alpha = 0.0299\, Q_o$$

Therefore, outflow discharge from the gates can be expressed by the following equations.

$$\left.\begin{array}{lll} Q = 106.9\, h_2^2 \Delta H & \text{for} & Q < 33.5\ m^3/s \\ Q = 59.8\, h_2\sqrt{\Delta H} & \text{for} & Q \geqq 33.5\ m^3/s \\ \text{and further}\quad Q = 23.0\, h_1\sqrt{h_1} & \text{for} & h_2/h_1 < 2/3 \end{array}\right\} \quad \text{(Fig. 5)} \quad (2)$$

(3) Leakage inflow through the existing mitre gates.

From Table 1, the volume of leakage inflow from 19 o'clock on 25th to 12 o'clock on 26th is estimated at 9 x 3600 m^3 or the average discharge of inflow is 0.5 m^3/s, while the average head difference during the same period is calculated to be 0.807 m. Similarly, the average discharge of inflow from 19 o'clock on 26th to 15 o'clock on 27th is estimated at 0.477 m^3/s, while the average head difference during the same period is 0.703 m.

If we assume that the discharge of leakage inflow through the existing mitre gates is roughly proportional to $\sqrt{\Delta H}$ or

$$Q = C\sqrt{\Delta H},$$

we can get values of C from the two examples mentioned above

$$C = 0.557 \text{ and } 0.569$$

Fig. 3 Outflow Discharges from Morokremgan Gates, August 1970

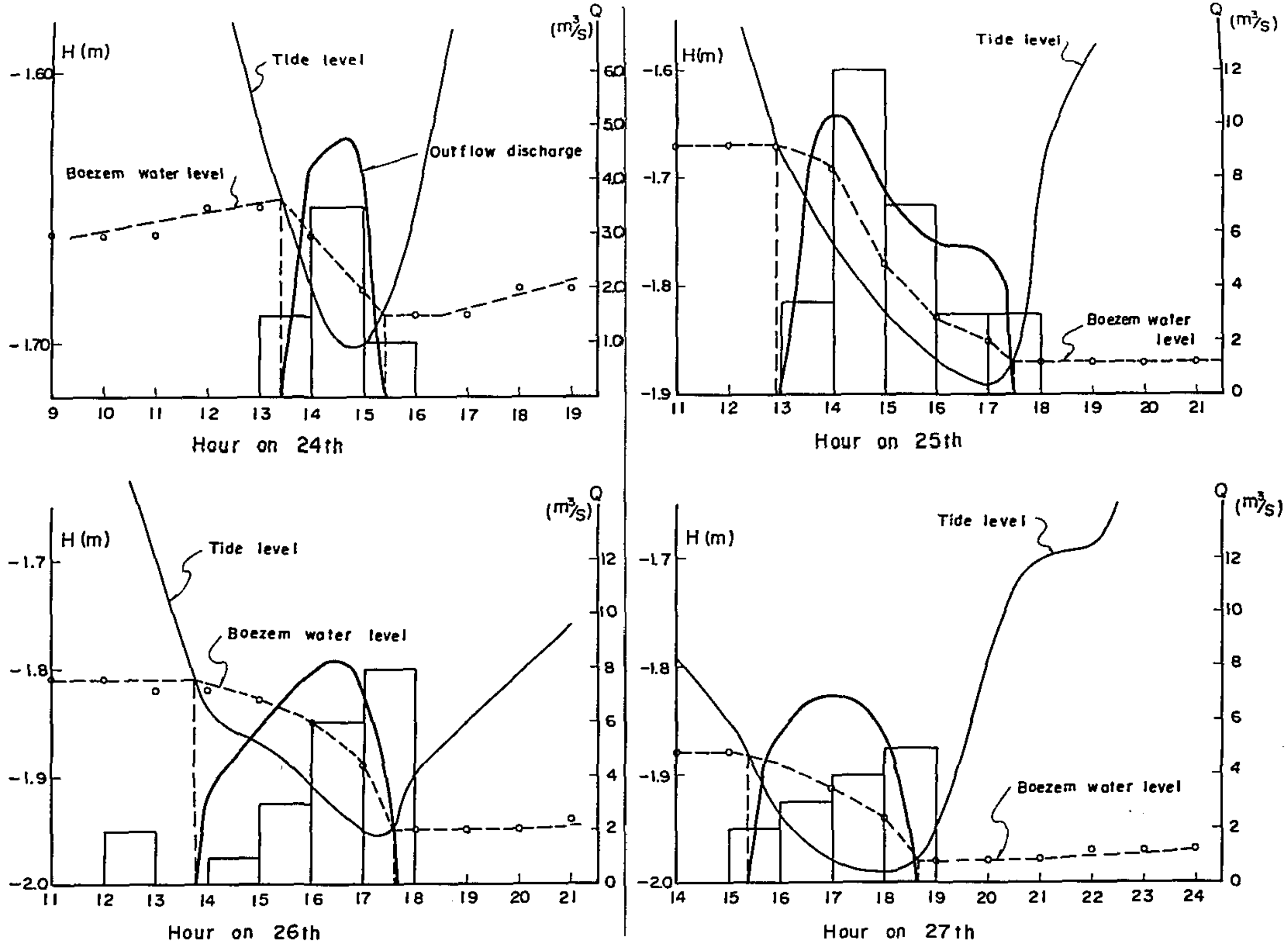


Fig. 4 Relation between α and Q_0

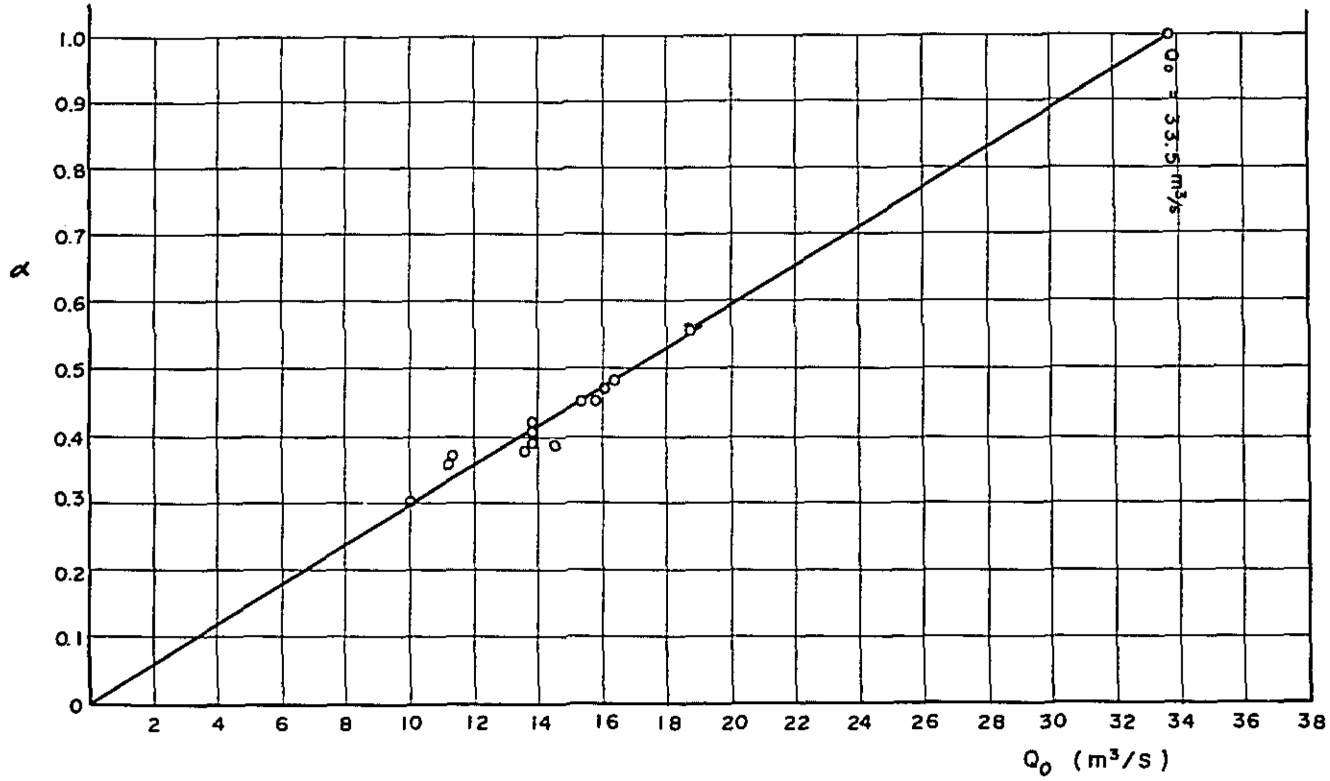
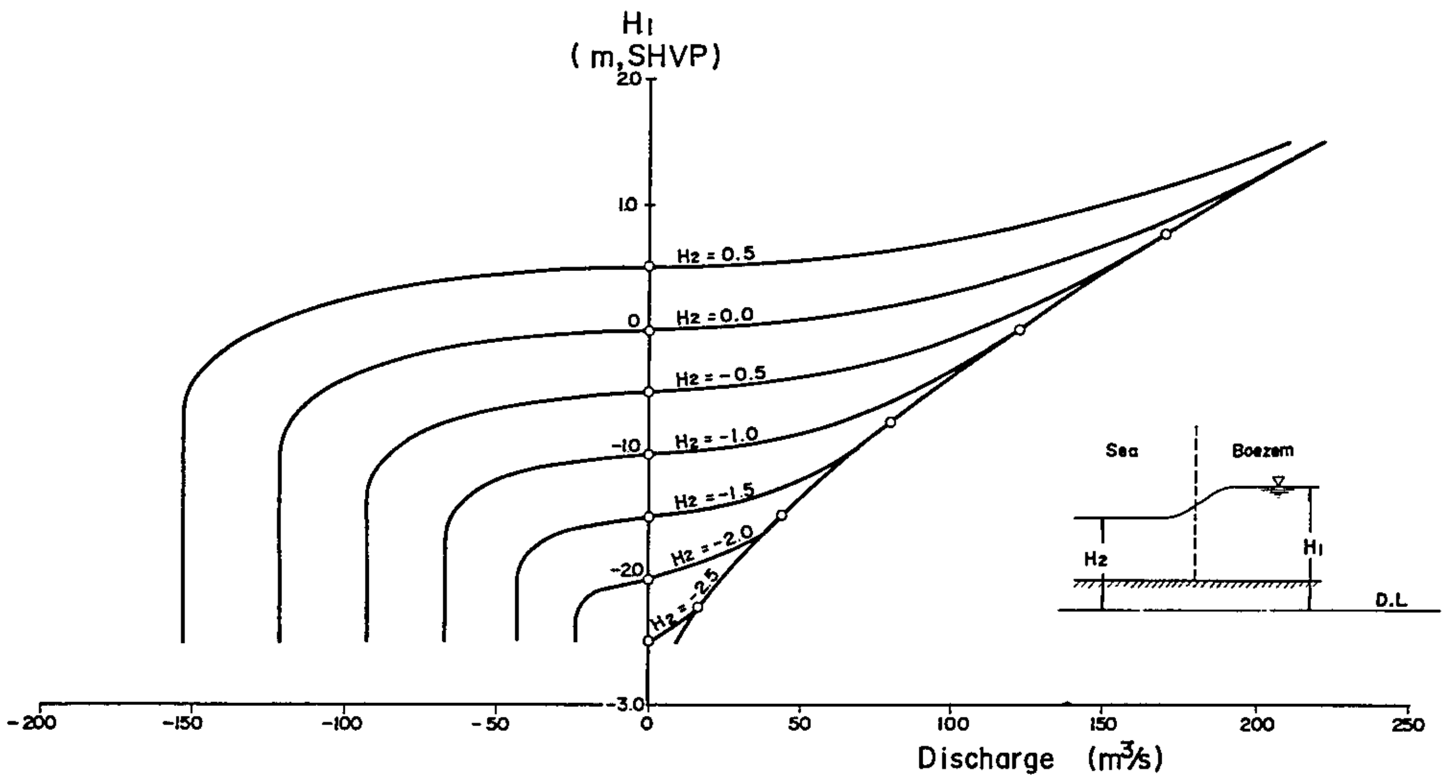


Fig. 5 Discharge Rating Curve of Morokrembangan Boezem Gate



$$C = 0.557 \text{ and } 0.569$$

On the average, we get the following equation

$$Q = 0.563\sqrt{\triangle H} \qquad (3)$$

2. Present Value of Runoff Coefficient of the Rivers Pouring into the Boezem.

Present value of runoff coefficient of the rivers pouring into the boezem has been studied by using flood records from 5th to 6th in February 1970.

The inflow discharges to the boezem, I, were calculated from the records of water levels inside and outside of Morokrembangan gates during the period from 10 o'clock on 5th to 15 o'clock on 7th, as shown in Table 3, where

H = inner water level of the boezem (m, SHVP)
H_s = sea water level outside of the gates (m, SHVP)
V = stored volume estimated from Fig. 1 $(\times \frac{1}{3600}, m^3)$
$\triangle T$ = 1 hour
h_2 = water depth shown in Fig. 2 (m)
$\triangle H = H - H_s$ = head difference (m)
O = outflow discharge through the gates, and minus-signed one is leakage inflow through the gates (m^3/s)
I = inflow discharge from the rivers to the boezem (m^3/s)

$$I_i = \frac{\triangle V_i}{\triangle T} + \frac{O_i + O_{i-1}}{2}$$

Table 3 Discharges from and into the Boezem, 4 ~ 7th Feb. 1970

Dt.	Hr.	H	H_s	$V/\triangle T$	$\triangle V/\triangle T$	h_2	$\triangle H$	O	I
5.	10	-1.61	-1.33	100			-0.28	-0.18	1.82
	11	-1.60	-1.10	102	2		-0.50	-0.33	1.745
	12	-1.59	-1.00	103	1		-0.59	-0.38	0.645
	13	-1.59	-0.95	103	0		-0.64	-0.42	-0.40
	14	-1.58	-1.00	104	1		-0.58	-0.34	0.62
	15	-1.57	-0.80	106	2		-0.77	-0.50	1.58
	16	-1.56	-0.83	107	1		-0.73	-0.47	0.515
	17	-1.55	-0.87	108	1		-0.68	-0.44	0.545
	18	-1.25	-0.70	156	48		-0.55	-0.36	47.60
	19	-1.15	-0.60	173	17		-0.55	-0.36	16.64
	20	-1.05	-0.45	189	16		-0.60	-0.39	15.625
	21	-0.90	-0.30	216	27		-0.60	-0.39	26.61
	22	-0.80	-0.10	234	17		-0.70	-0.46	16.575
	23	-0.70	-0.45	254	20		-0.25	-0.16	19.69
6.	0	-0.60	-0.64	272	18	2.385	0.04	24.29	30.065
	1	-0.75	-0.75	244	-28	2.275	0.04	22.10	-4.805
	2	-0.80	-0.89	234	-10	2.135	0.09	38.30	20.20
	3	-0.95	-0.99	207	-27	2.035	0.04	17.68	5.99
	4	-1.19	-1.20	166	-41	1.825	0.01	3.56	-30.38
	5	-1.30	-1.39	148	-18	1.635	0.09	25.68	-3.38
	6	-1.45	-1.50	124	-24	1.525	0.05	12.41	-4.955

Dt.	Hr.	H	H_s	V/ T	V/ T	h_2	H	O	I
6.	7	-1.50	-1.62	115	- 9	1.405	0.12	25.29	9.85
	8	-1.55	-1.67	109	- 6	1.355	0.12	23.45	18.37
	9	-1.52	-1.60	113	4	1.425	0.08	17.34	24.395
	10	-1.50	-1.45	115	2		-0.05	-0.03	10.655
	11	-1.48	-1.22	119	4		-0.26	-0.17	3.90
	12	-1.46	-1.15	122	3		-0.31	-0.20	2.815
	13	-1.45	-1.10	124	2		-0.35	-0.23	1.785
	14	-1.43	-1.05	127	3		-0.38	-0.25	2.760
	15	-1.40	-1.00	132	5		-0.40	-0.26	4.745
	16	-1.35	-0.95	140	8		-0.40	-0.26	7.74
	17	-1.32	-0.89	145	5		-0.51	-0.33	4.705
	18	-1.27	-0.78	153	8		-0.57	-0.37	7.65
	19	-1.20	-0.65	165	12		-0.55	-0.36	11.635
	20	-1.10	-0.50	181	16		-0.60	-0.39	15.625
	21	-1.00	-0.40	197	16		-0.60	-0.39	15.61
	22	-0.90	-0.35	216	19		-0.55	-0.36	18.625
	23	-0.80	-0.20	234	18		-0.60	-0.39	17.625
7.	0	-0.70	-0.09	254	20		-0.60	-0.39	19.61
	1	-0.50	-0.20	291	37		-0.30	-0.20	36.705
	2	-0.30	-0.35	333	42		0.05	35.77	59.785
	3	-0.55	-0.65	281	-52		0.10	44.91	-11.66
	4	-0.79	-1.19	236	-45		0.40	69.40	12.155
	5	-0.99	-1.25	200	-36		0.26	54.12	25.76
	6	-1.20	-1.39	165	-35		0.19	42.62	13.37
	7	-1.45	-1.50	124	-41		0.05	12.41	-13.485
	8	-1.53	-1.60	111	-13		0.07	15.17	0.790
	9	-1.60	-1.69	102	- 9		0.09	17.12	7.410
	10	-1.55	-1.40	108	6		-0.15	-0.10	14.51
	11	-1.51	-1.00	114	6		-0.51	-0.33	5.785
	12	-1.49	-0.95	118	4		-0.54	-0.35	3.66
	13	-1.46	-0.85	122	4		-0.61	-0.40	3.625
	14	-1.45	-1.10	124	2		-0.35	-0.23	1.685
	15	-1.43	-1.15	127	3		-0.28	-0.18	2.795

Dt.: Date
Hr.: Hour

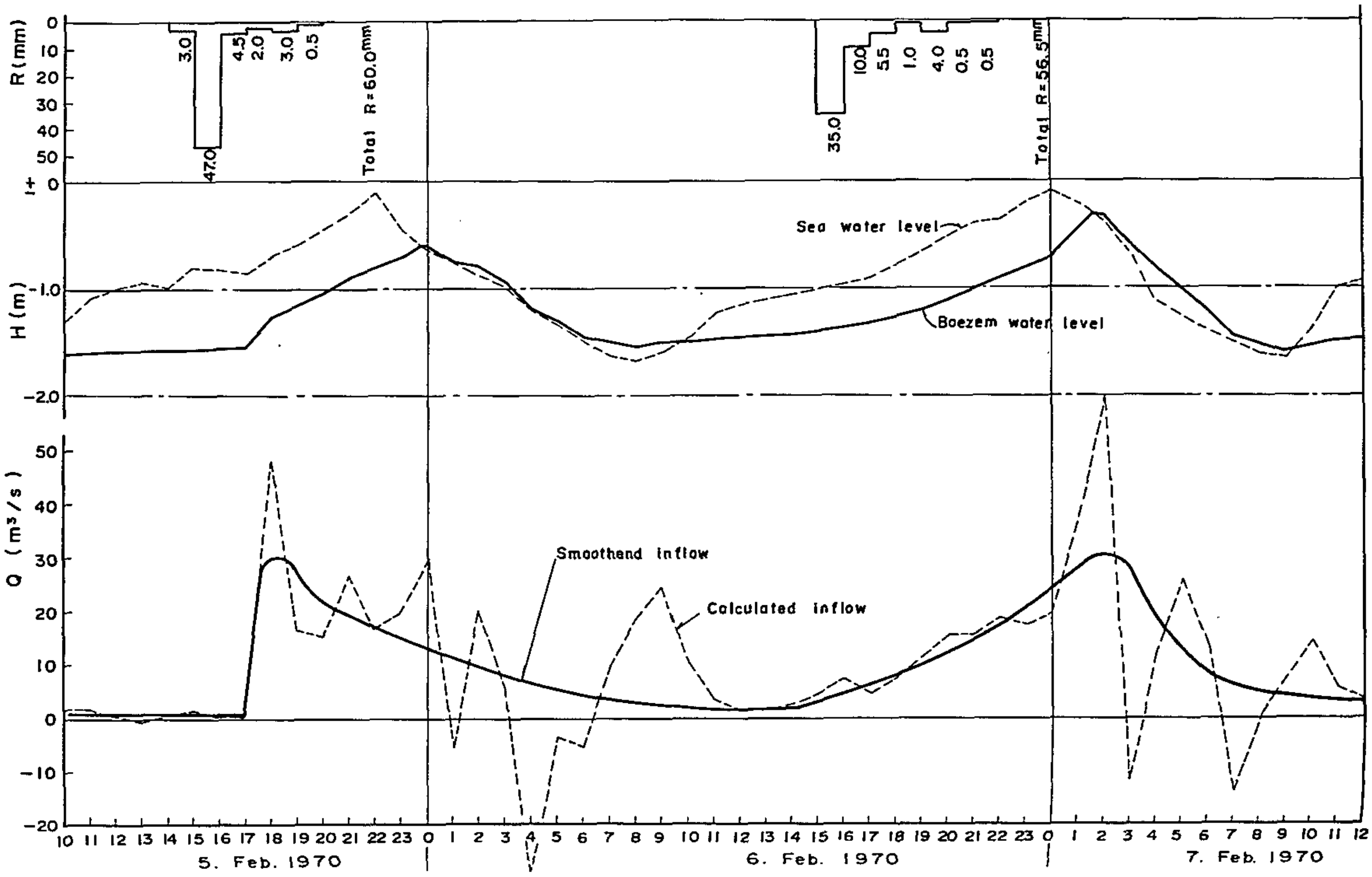
Calculated values of I are shown in Fig. 6, where large fluctuation of values are seen. This is presumed to result from low accuracy and large intervals of observation. Therefore, smoothening should be made over the fluctuated curve of inflow so as to equalize the whole amount of inflow. In Fig. 6, hourly rainfalls recorded at Surabaja Meteorological Station are also shown.

The average inflow from 10 o'clock to 17 o'clock on 5th is estimated at 0.87 m^3/s and this is taken as a base flow after 17 o'clock. Then the whole amount of inflow or the whole amount of surface run-off to the boezem for seventeen hours from 17 o'clock on 5th to 10 o'clock on 6th is

$$(218.745 - 0.87 \times 17)\ 3600 = 204 \times 3600\ m^3,$$

while the total volume of corresponding rainfalls, R = 60 mm, over the area of 15.8 km^2 is

Fig. 6 Inflow to the Boezem from Its Basin



$$15.8 \times 10^6 \times 60 \times 10^{-3} = 263 \times 3600 \text{ m}^3.$$

Therefore, coefficient for the total run-off is 204/263 = 0.776.

Similarly, if we take the average inflow from 11 o'clock to 17 on 6th as a base flow after 17 o'clock on 6th, the whole amount of surface run-off to the boezem for eighteen hours from 17 o'clock on 6th to 11 o'clock on 7th is estimated at

$$(257.50 - 4.07 \times 18)\ 3600 = 184 \times 3600 \text{ m}^3,$$

while the amount of corresponding rainfalls, R = 56.5 mm, is estimated at

$$15.8 \times 10^6 \times 56.5 \times 10^{-3} = 248 \times 3600 \text{ m}^3.$$

Therefore, we get a value of 0.742 as coefficient of total run-off. Consequently, the average value of coefficient of total run-off is estimated at 0.76.

Assuming water areas of the cross sections at the river-mouth and upstream reaches of the Greges river during its flood, we get 0.64 m/s and 0.33 m/s as respective mean velocities. Averaging these two values, we estimate the propagation velocities of flood as follows.

$$w = \frac{3}{2} v = \frac{3}{2}\left(\frac{0.64 + 0.33}{2}\right) = 0.73 \text{ m/s}$$

Since the length of the present Greges river is estimated at 7.0 km and inlet time for the urban sewer at the upstream end of the river is estimated at 24 min, the concentration time at the river-mouth becomes

$$\frac{7000}{0.73 \times 60} + 24 = 184 \text{ min} = 3 \text{ hr.}$$

In case of a flood on 5th February shown in Fig. 6, the peak of flood discharge appears about 3 hours after the peak of rainfall, which seems to prove the calculated value of concentration time to be right. Hence, assuming the concentration time at the river-mouth of the Greges river to be 3 hours, we can calculated value of run-off coefficient f for peak discharge in rational formula

$$Q = \frac{1}{3.6} frA$$

Q = peak discharge (m^3/s)
r = mean intensity of maximum rainfall during concentration time (mm/hr)
A = catchment area (km^2)

for the present condition of the Greges river by using the records of rainfalls and peak discharges on 5th flood and 6th flood.

For 5th flood

$$r = \frac{1}{3} (3.0 + 47.0 + 4.5) = 18.2 \text{ mm/hr}$$

$$Q \fallingdotseq 30 \text{ m}^3/\text{s}$$

$$\therefore f = 0.376$$

For 6th flood

$$r = \frac{1}{3}(35.0 + 10.0 + 5.5) = 16.8 \text{ mm/hr}$$

$$Q \fallingdotseq 30 \text{ m}^3/\text{s}$$

$$\therefore f = 0.407$$

Averaging the two value, we get

$$f = 0.4$$

for the present condition of drainage basin of the Greges river.

3. Water Level of the Boezem during 5-Year Storm.

Since it is expected that the urban drainage of Surabaja city will be planned for the 5-year storm, it is necessary to examine the effect of dredging of the boezem by using the same storm.

If we assume that the concentration time is 3 hours according to the previous study, the rainfall intensity of 5-year return period in Surabaja city is estimated at 29.4 mm/hr from Fig. 8 in Chapter VIII, Part 4. Hence, using the rational method, we obtain the peak discharge at 51.6 m^3/s, where A is taken as 15.8 km^2 and f is 0.4.

On the other hand, daily rainfall of 5-year storm is estimated at 140 mm by the study in Chapter VIII, Part 4 and we assume the coefficient of total runoff at 0.8 on the basis of the aforesaid study. Then, the total volume of surface runoff amounts to

$$140 \text{ mm} \times 15.8 \text{ km}^2 \times 0.8 = 1,767 \times 10^3 \text{ m}^3.$$

If we assume that discharge hydrograph is of triangular shape with a concentration time of 3 hours and a peak discharge of 51.6 m^3/s, the discharge hydrograph for the total runoff volume of 1,767 x 10^3 m^3 will be as follows:

Time (hr)	0	3	6	19
Discharge (m^3/s)	0	51.6	42.0	0

We have obtained the storage-capacity curve illustrated in Fig. 7 according to the dredging shown in Fig. 1 in Chapter V, Part 2. Using these curves and the above-mentioned discharge hydrograph together with the tide curve given in Chapter XVI, Part 4, we calculated the water levels of the boezem which are shown in Fig. 8. On that occasion, we used the equation (3) for outflow from the gate and the following equation (4) for continuity in the boezem

$$\left(\frac{I_{t-1} + I_t}{2} - \frac{O_{t-1} + O_t}{2}\right)\Delta t = V_t - V_{t-1} \qquad (4)$$

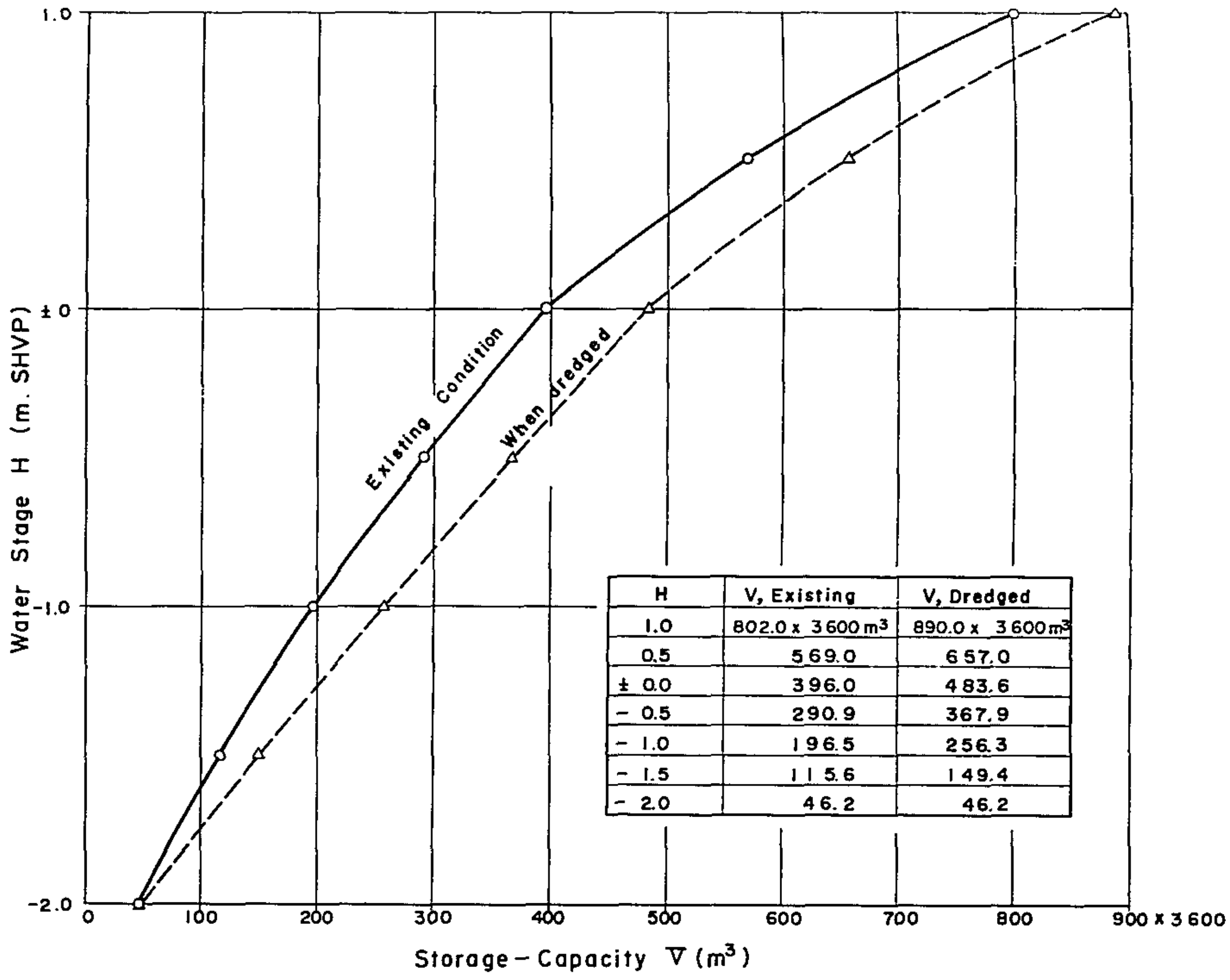
where I_{t-1}, I_t = inflow respectively at time t-1 and t, in m^3/s,

O_{t-1}, O_t = outflow respectively at time t-1 and t, in m^3/s,

V_{t-1}, V_t = stored volume respectively at time t-1 and t, in m^3

and Δt = time interval for computation, in sec.

Fig. 7 Storage-Capacity of Morokrembangan Boezem



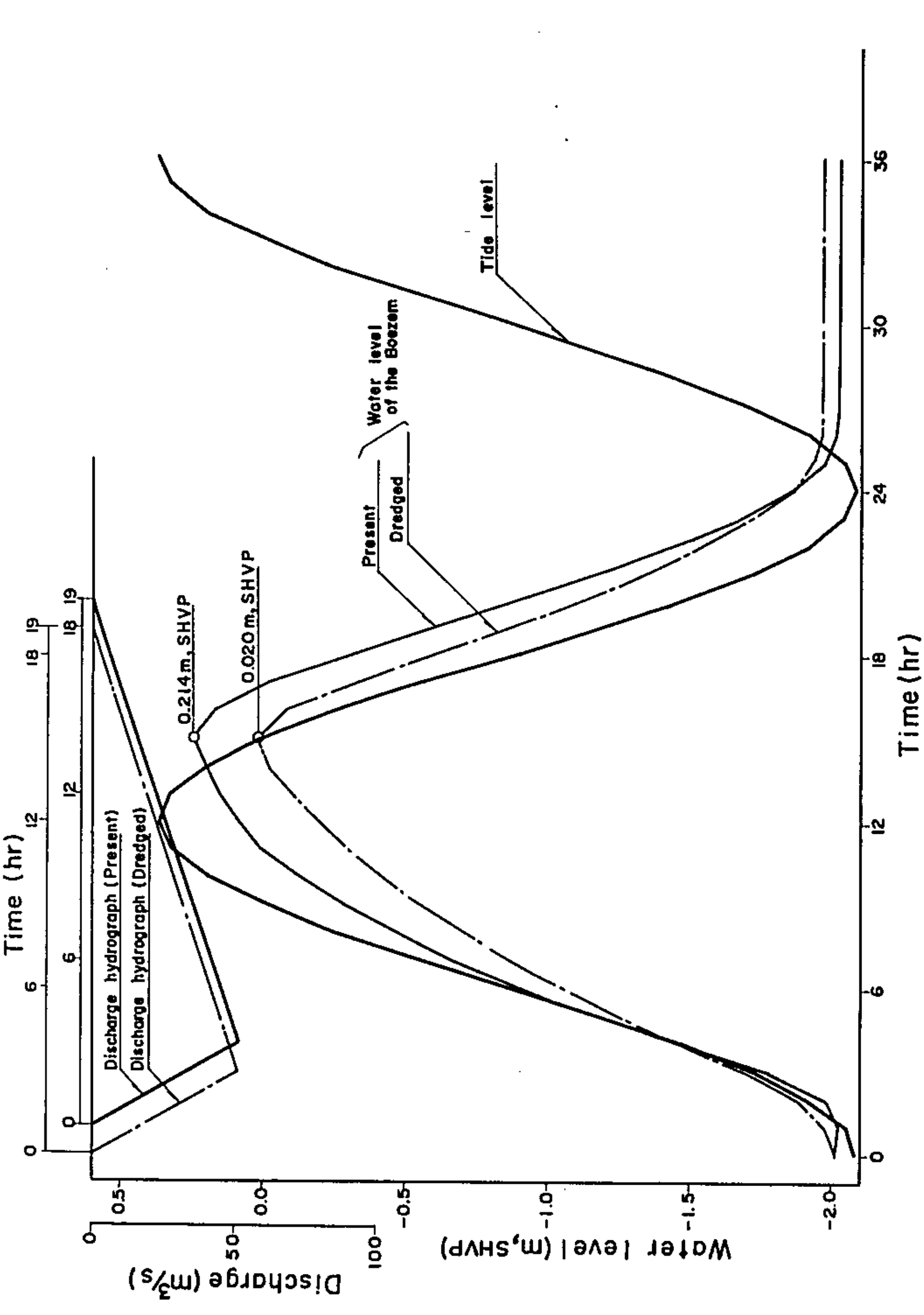
H	V, Existing	V, Dredged
1.0	802.0 x 3600 m^3	890.0 x 3600 m^3
0.5	569.0	657.0
± 0.0	396.0	483.6
− 0.5	290.9	367.9
− 1.0	196.5	256.3
− 1.5	115.6	149.4
− 2.0	46.2	46.2

Initial condition for computation was determined by giving a constant inflow of 1 m^3/s to the above-mentioned tide curve.

Fig. 8 shows the variation of water level of the boezem in the case of the present state of the boezem, where the highest water stage was searched according to the combination of time differences between the discharge hydrograph and the tide curve. Fig. 8 also shows the similar water stage in the case of dredged boezem.

Since the average ground level is estimated at from 0.2 m to 0.3 m, SHVP from the data provided by DPUT, it is obvious from Fig. 8 that the drainage of the land around and between the boezems may be very difficult, since the water stage of the boezem will reach as high as + 0.2 m, SHVP. On the contrary, the condition of the drainage not only of the above-mentioned land but also of the Greges basin will be improved, since the water stage of the boezem will be lowered as low as 0.02 m, SHVP.

Fig. 8 Water Level of the Boezem (5-year flood, f=0.4)



CHAPTER XV

ANALYSIS OF TIDE AND WAVE

1. Fifty-Year Sea Force.

Since the sea dike is faced to the eastern part of the Strait of Madura, it is exposed principally to the waves that raid from the east. The direction of the fetch ranges between E and SE. In other words, the direction of the wind which generates the waves affecting the sea coast ranges between E and SE. In this chapter, we call the wind having this range merely the east wind.

In the Surabaja Meteorological Station, lists of daily wind for ten years from 1962 to 1971 were available. They contain daily average wind directions and the maximum and the minimum wind velocities during the hourly observation from 7:00 a.m. to 7:00 p.m. From the lists, monthly days in which the east wind blew were counted and shown in Table 1 and Fig. 1. According to the average curve given in Fig. 1, the season of the east wind is limitted between April and November.

Next, monthly maximum velocities of the east wind were listed in Table 2 for the same period as mentioned above. These maximum wind velocities were plotted against the 10-year average days of the east wind in the corresponding month (Table 1) as shown in Fig. 2. This indicates that there is no relationship between the season of east wind and the strong east wind, that is, the strong east wind occurs in any season of the year.

Since it has been concluded in Chapter VI of Part 4 that the records of tide level at the outside of Morokrembangan boezem are usable in place of those at Surabaja harbor, the former were used for analysis of tide levels. Out of these records, the common period to both records of tide and wind and seven years from 1965 to 1971 were selected. Then, the wind directions and wind velocities at the time of monthly highest tide levels were listed in Table 3, and the tide levels at the time of the east wind velocities exceeding 15 knot were given in Table 4. These were plotted in Fig. 3, which indicates that there is no relationship between the velocities of the strong east winds and the tide levels, that is, the former occures independently of the latter. This means that the probability of the sea force should be expressed by the compound probability of tide level and wind velocity.

Annual maximum tide levels during the seven years from 1965 to 1971 are given in Table 5 and annual maximum wind velocities during the ten years from 1962 to 1971 are given in Table 6.

Table 1 Days of East Wind (E - SE) at Surabaja

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
1962	0	0	0	14	23	24	29	31	30	27	11	2
1963	0	0	0	11	28	30	31	30	27	28	29	6
1964	3	6	1	18	22	25	25	31	28	22	12	12
1965	1	2	4	23	28	29	30	30	29	21	19	8
1966	0	5	3	16	27	17	27	23	22	14	12	6
1967	0	0	8	27	20	23	18	24	27	18	10	5
1968	0	2	9	16	5	15	11	24	21	23	4	11
1969	4	0	2	21	20	22	29	27	24	24	7	4
1970	2	1	1	14	13	20	26	29	24	16	2	0
1971	0	0	0	1	7	21	21	26	28	4	2	1
Max.	4	6	9	27	28	30	31	31	30	28	29	12
Min.	0	0	0	1	5	15	11	24	21	4	2	0
Mean	1	1.6	2.8	16.1	19.3	22.6	24.7	27.5	26.0	19.7	10.5	5.5

Table 2 Monthly Maximum Velocity of East Wind (E - SE) at Surabaja, Wind Velocity in Knot

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
1962	0	17	0	13	15	15	20	15	17	20	14	10
1963	0	0	0	15	17	17	18	15	18	22	25	12
1964	10	13	7	12	14	15	12	15	16	10	14	11
1965	10	7	7	15	16	15	17	17	17	18	15	15
1966	0	6	7	12	12	12	15	15	15	15	15	12
1967	0	0	9	22	12	15	15	15	15	15	13	10
1968	0	20	10	12	9	15	15	18	15	16	10	15
1969	12	0	7	13	15	18	15	18	17	20	15	12
1970	10	10	7	18	16	18	14	15	18	25	15	0
1971	0	0	0	10	10	16	18	15	20	16	8	10
Max.	12	20	10	22	17	18	20	18	20	25	25	15

Table 3 Wind Direction and Wind Velocity at the Time of Monthly Highest Tide Level

Date	Tide level (m,SHVP)	Wind Direction	Wind Velocity (knot)
Jan.1965	–	–	–
Feb. 15,	+0.20	NE	5
Mar. 16	-0.20	W	10
Apr. 19	-0.15	E	8
May 4	+0.23	E	16
Jun. 2	+0.44	E	10
Jul. 1	+0.35	SE	10
Aug. 17	+0.12	E	12
Sep. 25	-0.15	Var.	12
Oct. 27	-0.20	E	12
Nov. 24	+0.10	NE	10
Dec. 22	+0.24	S	12
Jan.8'66	+0.31	Var.	5
Feb. 4	+0.10	W	4
Mar. 4	-0.25	N	9
Apr. 26	+0.05	E	9
May 22	+0.25	E	6
Jun. 21	+0.25	E	7
Jul. 19	+0.15	SE	8
Aug.	–	–	–
Sep.	–	–	–
Oct. 31	-0.12	SE	12
Nov. 30	+0.30	Var.	15
Dec. 28	+0.28	Var.	5
Jan.10,'67	+0.31	calm	12
Feb. 9	+0.19	W	7
Mar. 25	-0.02	SE	9
Apr. 26	+0.25	E	10
May 26	+0.15	E	15
Jun. 10	+0.17	SE	13
Jul. 9	+0.10	SE	12
Aug. 7	+0.06	Var.	10
Sep. 5	-0.20	SE	12
Oct. 21	-0.10	SE	6
Nov. 4	+0.25	Var.	5
Dec. 31	+0.30	NW	
Jan.1,'68	+0.35	W	6
Feb. 1	-0.10	Var.	9
Mar. 20	-0.14	SE	5
Apr. 17	+0.20	E	8
May 14	+0.25	Var.	5
Jun. 13	+0.37	SW	8

Date	Tide level (m,SHVP)	Wind Direction	Wind Velocity (knot)
Jul. 11	+0.40	SE	15
Aug. 9	+0.10	NW	5
Sep. 5	-0.02	SE	14
Oct. 24	+0.25	E	6
Nov. 23	+0.36	N	10
Dec. 20	+0.30	NW	5
Jan.18,'69	+0.30	E	12
Feb.16	+0.10	NW	5
Mar. 5	-0.09	NW	4
Apr. 21	+0.15	E	7
May 19	+0.17	E	7
Jun. 2	+0.30	Var.	10
Jul. 1	+0.35	–	–
Aug. 27	0.00	SE	12
Sep. 25	-0.20	E	14
Oct. 15	-0.10	SE	15
Nov. 11	+0.25	SE	9
Dec. 10	+0.35	E	12
Jan.7,'70	+0.15	W	6
Feb. 6	-0.09	NW	12
Mar. 14	-0.10	Var.	7
Apr. 25	+0.05	NW	20
May 24	+0.40	E	12
Jun. 5	+0.36	NW	11
Jul.	–	–	–
Aug. 31	+0.35	E	13
Sep. 29	+0.42	E	10
Oct. 6	+0.20	SE	16
Nov. 14	+0.35	Var.	8
Dec. 30	+0.25	NW	8
Jan.12,'71	+0.25	W-NW	20
Feb. 20	+0.20	NW	14
Mar. 20	+0.20	NW	8
Apr. 28	+0.35	Var.	12
May 25	+0.43	Var.	8
Jun. 23	+0.30	E	6
Jul. 9	+0.35	SE	16
Aug. 19	+0.10	SE	12
Sep. 23	+0.20	E	7
Oct. 9	+0.20	Var.	14
Nov. 4	+0.40	E	6
Dec. 4	+0.38	Var.	12

Table 4 Tide Level at the Time of Wind Velocity over 15 knot of E - SE Wind Direction

Year	Date	Wind		Tide level
		Direction	Velocity (knot)	(m, SHVP)
1965	Oct. 31	E	18	-0.15
1966	Jun. 3	SE	15	+0.20
	Aug. 23	E	15	-
	Sep. 27	E	15	-0.54
	Oct. 24	SE	15	-0.55
	Nov. 4	E	15	-0.30
1967	Apr. 8	SE	22	-0.60
1968	Feb. 19	SE	20	-0.55
1969	Oct. 6	E	20	-0.60
1970	Oct. 18	E	25	+0.34
	Oct. 5	E	20	-0.25
1971	Sep. 7	ESE	20	-0.36

Table 5 Annual Maximum Tide Level

Year	Tide Level (m, SHVP)
1965	0.44
1966	0.31
1967	0.31
1968	0.40
1969	0.35
1970	0.42
1971	0.43

Table 6 Annual Maximum Wind Velocity

Year	Wind Velocity (knot)
1962	20
1963	25
1964	16
1965	18
1966	15
1967	22
1968	20
1969	20
1970	25
1971	20

Fig.1 Days of East Wind

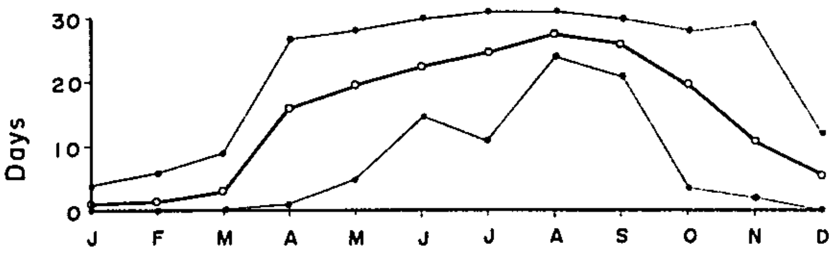


Fig. 2

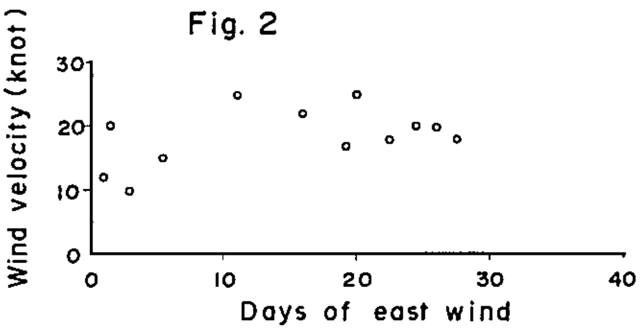
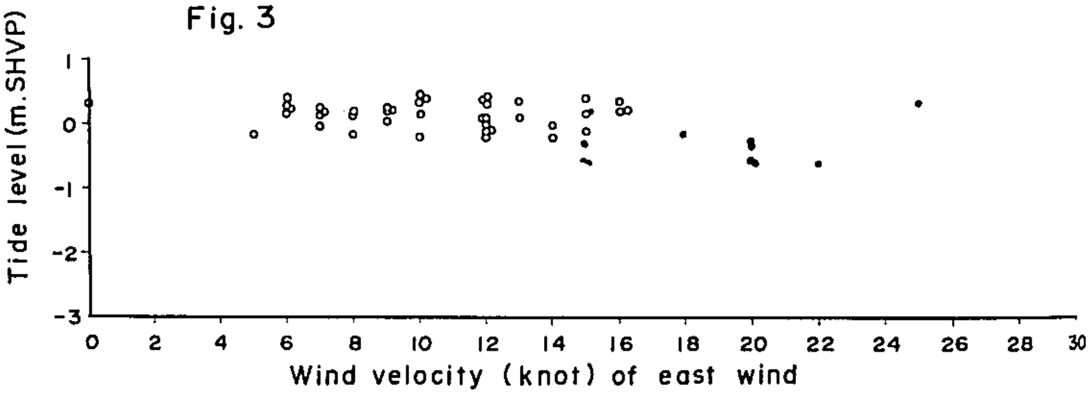


Fig. 3



Figs. 4-1 and 4-2 sho respective return periods of them, from which the compound probability of tide level and the east-wind velocity at 50-year return period can be obtained as shown in Fig. 5. According to the figure, we have no end of compound probabilities. However, among them, there must exist a compound probability corresponding to a combined sea force which attacks the sea dike most strongly. To find this combination of tide level and the east wind velocity requires too much computations. Hence, in this case, we decided to attach importance to the maximum values in the past for both the tide level and the wind velocity.

The highest tide level in the past records collected here is +0.44 m, SHVP. The corresponding value of wind velocity is read at 24.8 knot on the curve shown in Fig. 5. Similarly, from the figure, we obtain +0.437 m, SHVP as the value of tide level which corresponds to the maximum value of the east wind velocity 25 knot. On the basis of these values, the sea force at 50-year return period was determined as follows.

Tide level	+0.44 m, SHVP
The east wind velocity	25 knot = 12.9 m/s

2. Run-Up of Wave.

We selected a typical cross-section of sea dike which seemed to be dangerous against the uprush of sea waves. The elevation of the crown of dike was +0.95 m, SHVP. The average sea-side slope of dike was 1 : 4 and the dike-foot depth was 0.843 m at the design high tide level +0.44 m, SHVP. These dimensions were used for calculation of run-up of wave.

The calculations were made for ten cases of wind velocities U = 12.9 to 4 m/s or 25 to 7.7 knot keeping the tide level constant at +0.44 m, SHVP. The calculated values of run-up R and the corresponding return periods are shown in Table 1. The following explains the calculation of case 1 in Table 1.

(1), (2) U is wind velocity, given in this case. U = 12.9 m/s.

(3) h is the depth of the sea and constant in this case. According to the chart, the elevation of the sea bottom is -3.00 m and the datum is -2.72 m, SHVP. Hence, the water depth at 0 m, SHVP is 5.72 m. Accordingly, the water depth h at the tide level of +0.44 m is 6.16 m, SHVP.

(4) $gh/U^2 = 0.364$.

(5) From Fig. 6, $gH_{1/3}/U^2 = 0.070$. $H_{1/3}$ is significant wave height.

(6) Hence $H_{1/3} = 1.18$ m.

(7) From Fig. 6, $gF/U^2 = 2000$.

(8) Hence F = 33.8 km. F in this case means the minimum distance required for generation of fully developed wave height $H_{1/3}$. The actual fetch is 800 km. Hence the fetch F is enough to the full development of wave.

(9) T is the period of the wave. According to Bretschreider,

$$T = 3.86\sqrt{H_{1/3}} = 4.19 \text{ sec.}$$

Fig. 4 - 1

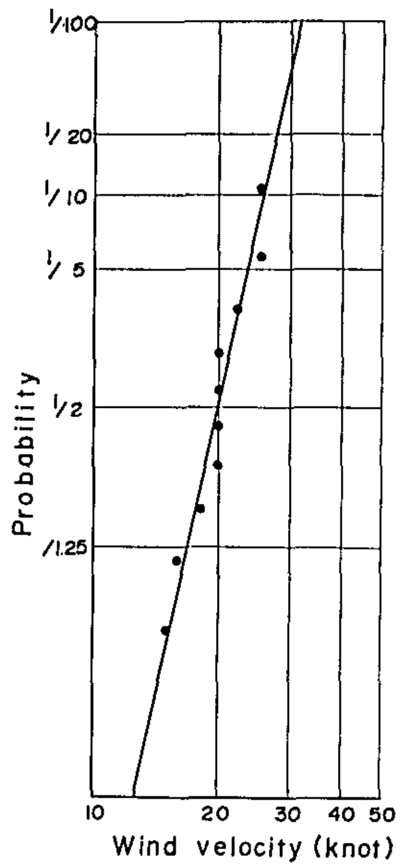


Fig. 4 - 2

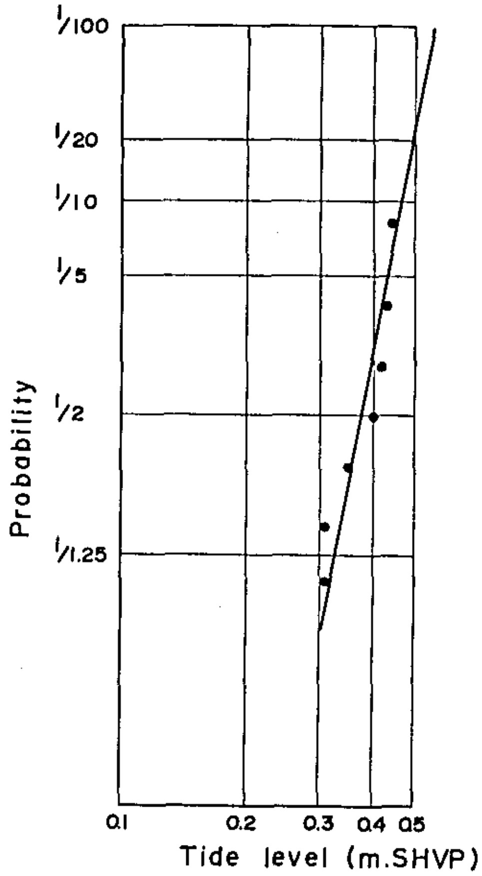
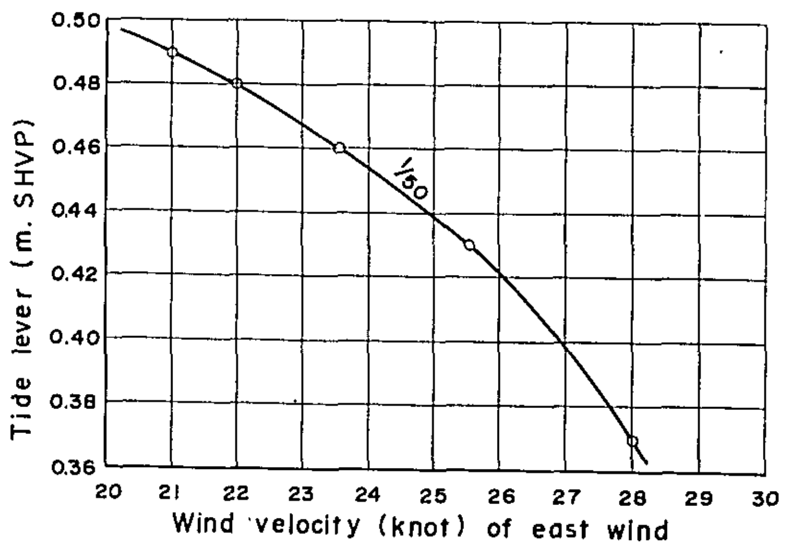


Fig. 5



(10) L_o is the wave length of the deep water wave corresponding to the shallow water wave calculated above.

$$L_o = \frac{gT^2}{2\pi} = 27.4 \text{ m.}$$

(11) d is the dike-foot depth and equal to 0.843 m.

(12) $d/L_o = 0.031$.

(13) α is incident angle of wave in degree. $\alpha = 35^o$ in this case.

(14) K_r is refraction coefficient. From Fig. 7, $K_r = 0.92$.

(15) H_o' is the height of the equivalent deep water wave.

$$H_o' = H_{1/3} \times K_r = 1.09 \text{ m.}$$

(16) H_b is the maximum wave height which may occur at the foot of the dike. The theory of solitary wave was used in this case.

$$H_b = 0.78d = 0.66 \text{ m.}$$

(17) H_b/K_r is the height of deep water wave equivalent to the above wave H_b.

$$K_b/K_r = 0.72 \text{ m.}$$

(18) H_b/K_r is smaller than H_o'. Hence the value of 0.72 m should be taken as the H_o' to be used for the calculation of run-up.

(19) $H_o'/L_o = 0.026$.

(20) i is the slope of the sea-side face of the dike.

$$i = 1/4.$$

(21) From Fig. 8, $R/H_o' = 1.17$, where R is the run-up of the wave measured above the design tide level.

(22) Hence R = 0.842 m.

(23) H_w is the elevation of the run-up.

$$H_w = 0.44 + R = 1.282 \text{ m.}$$

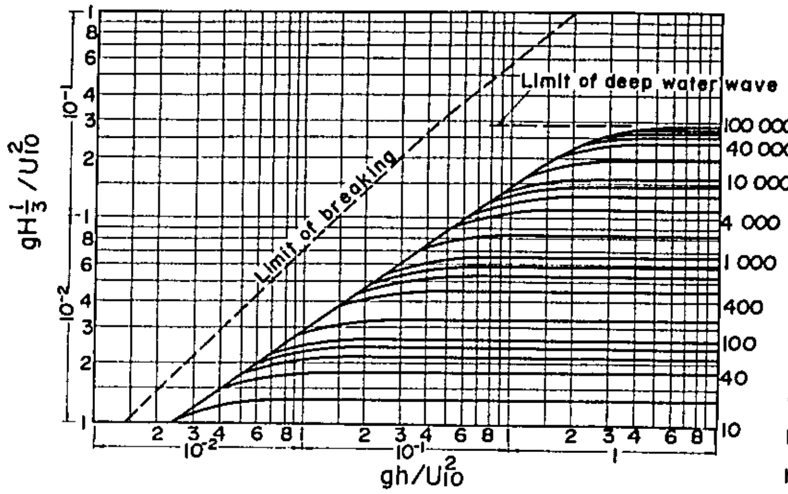
(24) Return period of tide level is 6.45 years for +0.44 m, SHVP.

(25) Return period of wind velocity is 7.75 years for U = 12.9 m/s.

(26) Compound return period is 50 years.

Fig. 9 shows the wave run-ups for several return periods of the sea forces. The run-up for the 50-year sea force is +1.28 m, SHVP. Hence, the elevation of +1.5 m, SHVP was taken as the design height of dike crown, in consideration of some free board.

Fig. 6 $gH_{1/3}/U_{10}^2 - gh/U_{10}^2 - gF/U_{10}^2$ Curve



$H_{1/3}$: Significant wave
U_{10} : Wind velocity
F : Fetch
h : Water depth
f=0.01 : Coefficent of friction

Fig. 7 $\alpha_0 - d/L_0$ Curve

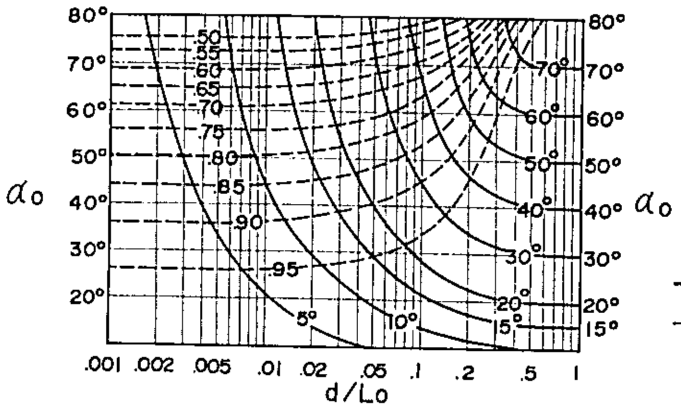


Fig. 8 $H_0'/L_0 - R/H_0'$ Curve

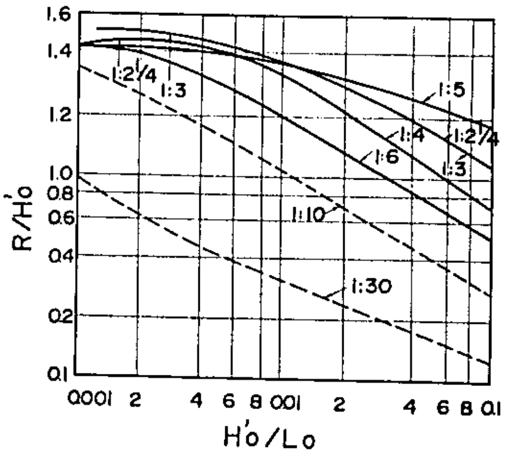


Fig. 9 Run-up of Waves

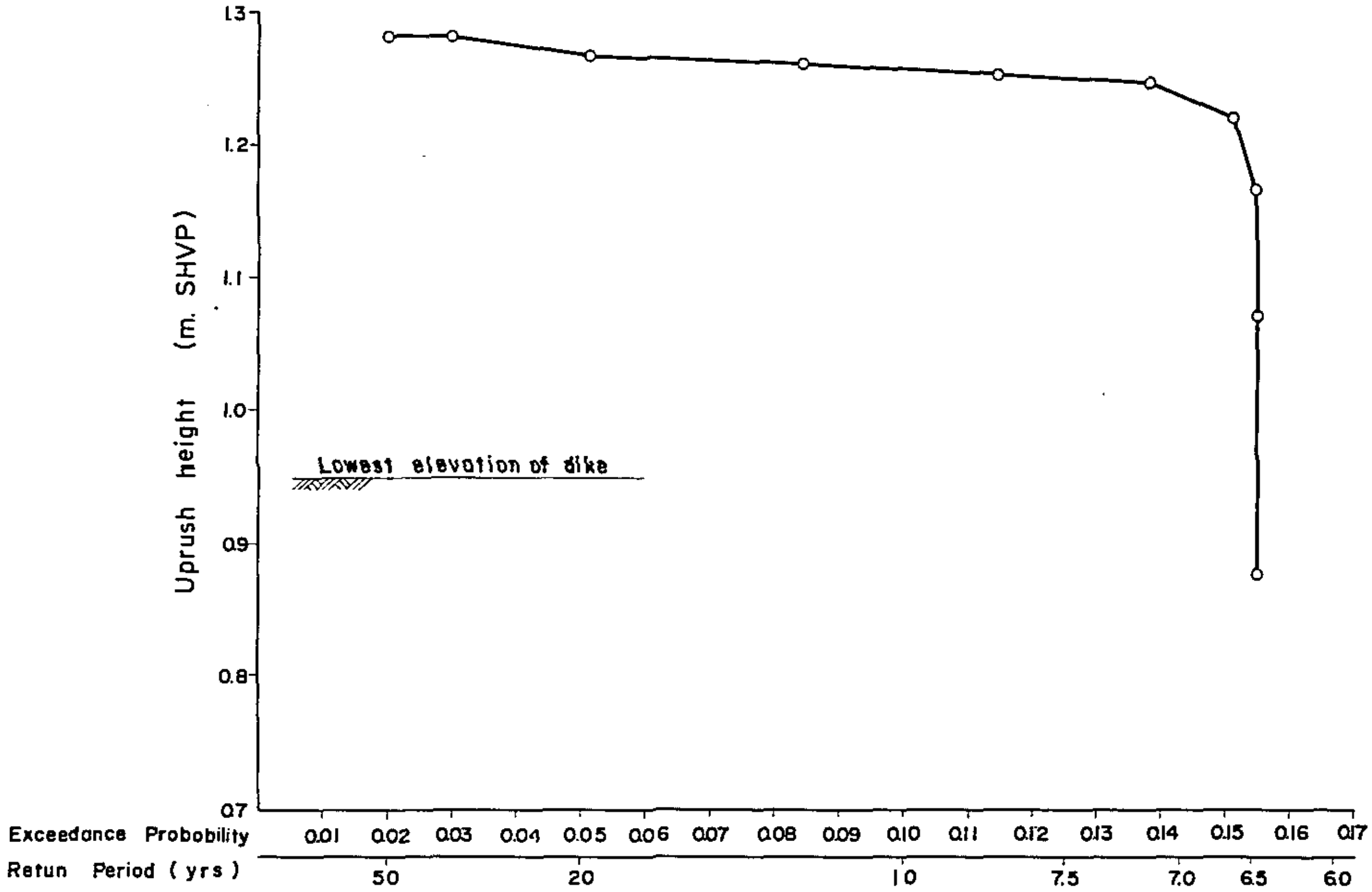


Table 1 Calculation of Run-up

No.	Term	Case-1	Case-2	Case-3	Case-4	Case-5	Case-6	Case-7	Case-8	Case-9	Case-10
1	U (m/s)	12.9	12	11	10	9	8	7	6	5	4
2	" (knot)	25.00	23.30	21.36	19.42	17.48	15.53	13.59	11.65	9.71	7.77
3	h (m)	6.16	6.16	6.16	6.16	6.16	6.16	6.16	6.16	6.16	6.16
4	gh/U^2	0.364	0.419	0.499	0.604	0.745	0.943	1.231	1.68	2.42	3.77
5	$gH_{1/3}/U^2$	0.070	0.078	0.87	0.100	0.120	0.140	0.170	0.20	0.25	0.27
6	$H_{1/3}$ (m)	1.18	1.15	1.08	1.02	0.99	0.91	0.85	0.735	0.638	0.441
7	gF/U^2	2,000	3,000	4,000	6,000	8,000	15,000	20,000	40,000	60,000	80,000
8	F (km)	33.8	44.1	49.4	61.2	66.2	98	100	147	153	131
9	T (sec)	4.19	4.14	4.01	3.90	3.84	3.68	3.56	3.31	3.08	2.56
10	L_o (m)	27.4	27.3	25.1	23.7	23.0	21.2	19.7	17.1	14.8	10.2
11	d (m)	0.843	0.843	0.843	0.843	0.843	0.843	0.843	0.843	0.843	0.843
12	d/L_o	0.031	0.031	0.034	0.036	0.037	0.040	0.043	0.049	0.057	0.083
13	α (degree)	35	35	35	35	35	35	35	35	35	35
14	K_r	0.92	0.92	0.92	0.92	0.92	0.92	0.925	0.925	0.925	0.94
15	H_o' (m)	1.09	1.06	0.99	0.94	0.91	0.84	0.79	0.68	0.59	0.41
16	H_b (m)	0.66	0.66	0.66	0.66	0.66	0.66	0.66	0.66	0.66	0.66
17	H_b/K_r (m)	0.72	0.72	0.72	0.72	0.72	0.72	0.71	0.71	0.71	0.70
18	H_o'	0.72	0.72	0.72	0.72	0.72	0.72	0.71	0.68	0.59	0.41
19	H_o'/L_o	0.026	0.026	0.029	0.030	0.031	0.034	0.036	0.040	0.040	0.040
20	i	1/4	1/4	1/4	1/4	1/4	1/4	1/4	1/4	1/4	1/4
21	R/H_o'	1.17	1.17	1.15	1.14	1.13	1.12	1.10	1.07	1.07	1.07
22	R (m)	0.842	0.842	0.828	0.821	0.814	0.806	0.781	0.728	0.631	0.439
23	H_w (m,SHVP)	1.282	1.282	1.268	1.261	1.254	1.246	1.220	1.168	1.071	0.879
24	Return period of tide level (yr.)	6.45	6.45	6.45	6.45	6.45	6.45	6.45	6.45	6.45	6.45
25	Return Period of wind velocity(yr.)	7.75	5.08	3.03	1.85	1.35	1.12	1.03	1.00	1.00	1.00
26	Return period(yr.)	50.0	32.8	19.5	11.9	8.7	7.2	6.6	6.5	6.5	6.5

CHAPTER XVI

DRAINAGE THROUGH SLUICES ON THE SEA DIKE AND LANDSIDE INUNDATION

1. Storage-Capacity Curves of Inundation Basins.

The area protected by the sea dike is divided into two basins for the purpose of studying landside inundation. The northern one, say north basin, contains the catchment basins of the Pegirian river and Tambakwedi drainage canal and other area served by Tjumpat, Kendjeran and Sukolilo sluices. The other one, say south basin, contains the catchment basins of Larangan, Kalisari, Kalidami, Keputich, and Medokan drainage canals. Using the topographic map made by aerophotographic surveying and the survey result of the sea dike made by Virama Karya, storage-capacity curves for the above two basins have been made as shown in Fig. 1.

2. Discharge through Flap Gates.

The existing sluices on the sea dike are all flap gates. Their general structure is assumed as shown in Fig. 2.

Govinda Rao's formula is applicable to a sluice without gate.

$$Q = CBh^{3/2} \qquad (1)$$

where B is the width in m, h is the water depth in m at the upstream end of the sluice, C is a coefficient such as

$$\begin{aligned} C &= 1.642 \frac{h}{L}^{0.022} && \text{for} \quad 0<\frac{h}{L} \leqq 0.1 \\ C &= 1.552 + 0.083 \frac{h}{L} && \text{for} \quad 0.1<\frac{h}{L}\leqq 0.4 \qquad (2) \\ C &= 1.444 + 0.352 \frac{h}{L} && \text{for} \quad 0.4<\frac{h}{L}\leqq (1.5\sim 1.9) \end{aligned}$$

In our case, the range of h is presumed to be from 1 m to 2 m and the range of L is from 5 m to 10 m. Hence, the value of H/L ranges from 0.1 to 0.4 namely

$$\begin{aligned} C &= 1.560 && \text{for} \quad h/L = 0.1 \\ C &= 1.585 && \text{for} \quad h/L = 0.4 \end{aligned}$$

Since the above two values of C are nearly the same, the average value can be used, then we get

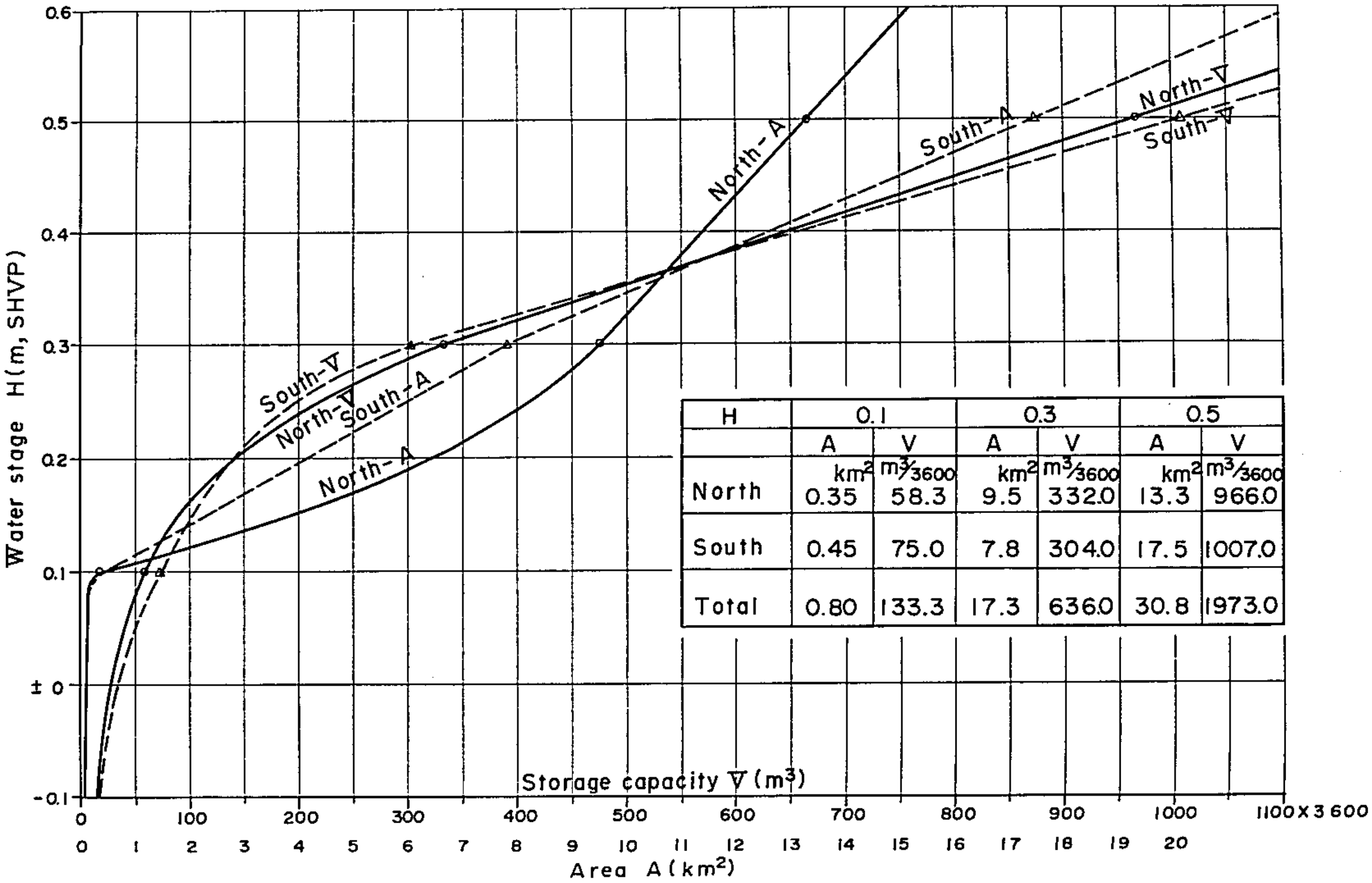
$$Q = 1.573\ Bh^{3/2} \qquad (3)$$

This equation should be applicable to the case of perfect overflow.

When the tide level is high, the overflow may become submerged. For such a case, the following equation may be used

$$Q = C'Bh_s\sqrt{2g\ (h - h_s)} \qquad (4)$$

Fig. 1 Storage-Capacity Curves of South Basin and North Basin



H	0.1		0.3		0.5	
	A	V	A	V	A	V
	km²	m³/3600	km²	m³/3600	km²	m³/3600
North	0.35	58.3	9.5	332.0	13.3	966.0
South	0.45	75.0	7.8	304.0	17.5	1007.0
Total	0.80	133.3	17.3	636.0	30.8	1973.0

Fig. 2

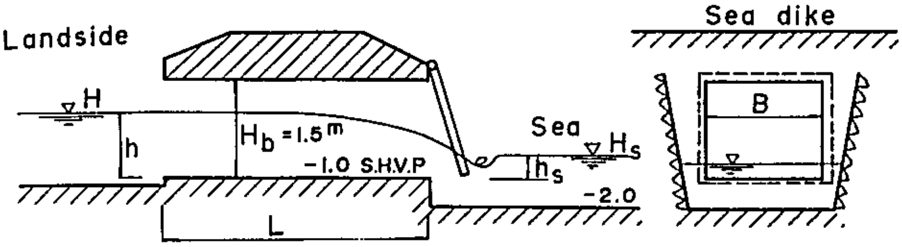


Fig. 3

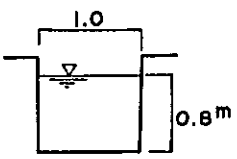


Fig. 4

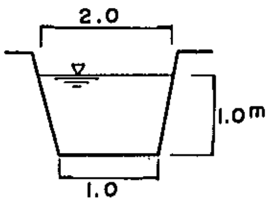
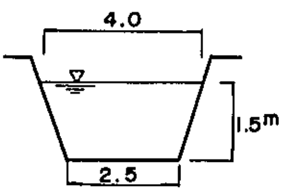


Fig. 5



where h_s is the water depth (shown in Fig. 2) from the bottom of the sluice to the sea water level.

This equation indicates to give the maximum value of Q when $h_s = 2h/3$. Hence, if we assume that this value of Q is equal to the value of Q in the equation (3), we get $C' = 0.923$, or

$$Q = 0.923Bh_s\sqrt{2g\ (h - h_s)} \qquad (5)$$

This equation should be applicable to a case of submerged overflow.

Next, we should study the effect of flap gate, because the above-mentioned equations (3) and (5) are all for the case of full-opened sluice. The existing flap gates are either rusted at their hinges or covered with drifted sand in front of them. This makes the gates difficult or impossible to open. Therefore, we have introduced a correction coefficient β as shown in the following, and assumed $\beta = 0.3$.

$$Q = 1.573 \cdot B \cdot h^{3/2} \cdot \beta \qquad \text{for} \quad h_s < \frac{2}{3}\ h \qquad (6)$$

$$Q = 0.923Bh_s\ \ 2g\ (h - h_s) \cdot \beta \quad \text{for} \quad h_s > \frac{2}{3}\ h \qquad (7)$$

3. Flood Runoff from the Area Protected by the Sea Dike.

(1) Flood runoff of Kalidami canal.

1) Concentration time.

Inspecting the topographic map and aerophotographic map, we assumed the system of canals and creeks as follows.

The most upstream creek is 250 m in length, 1/4000 in slope, and gathers the storm water in the basin 30 m in width on each side of it. An intermediate canal is 1,500 m in length, 1/3000 in slope, and collects the water from the above upstream creeks. And then a main drainage canal called Kalidami canal which is 4,300 m in length and 1/5,000 in slope collects the storm water from the above-mentioned branch canals.

The inlet time to the most upstream creek has been estimated at 8 min in a previous study. Next, if we assumedthe average cross-section of the most upstream creek as in Fig. 3, the mean velocity v is calculated by Manning's formula

$$v = \frac{1}{n} R^{2/3} I^{1/2} = 0.29 \text{ m/s}$$

where R is hydraulic mean depth in m, I is slope, and n was assumed at 0.025. Hence, the propagation velocity w becomes $w = 1.5v = 0.43$ m/s after Kleitz-Seddon. Consequently, the propagation time for this creek becomes 10 min.

Similarly, the propagation velocities are calculated at 36 min and 92 min respectively for the intermediate and main canals assuming the average cross-sections as in Fig. 4 and 5. Consequently, the total concentration time T becomes

$$T = 8 + 10 + 36 + 92 = 146 \text{ min}$$

2) Flood runoff of Kalidami canal.

The peak discharges Q of flood runoff from Kalidami basin are calculated by rational formula for several return periods shown in Table 1, assuming that the concentration time is 146 min for the all return periods mentioned above, giving f = 0.2 to the formula in view of the result obtained in the runoff analysis of the Greges river, and using the rainfall intensity-duration-frequency curve at Surabaja which has already been studied. The result is shown in Table 1.

Table 1 Peak Discharge of Runoff from Kalidami Basin

Return period (years)	f	Catchment area A (km^2)	Rainfall intensity r (mm/hr)	Peak discharge Q (m^3/s)
50	0.2	7.5	64.0	26.7
10	0.2	7.5	43.0	17.9
5	0.2	7.5	34.5	14.4
2	0.2	7.5	24.3	10.1
1.05	0.2	7.5	13.3	5.6

(2) Runoff from North and South Basins.

For convenience of inundation analysis, the area protected by the sea dike has been divided into two basins, North Basin and South Basin, considering the topography. North Basin has an area of 27.4 km^2 which consists of 9.6 km^2 of the Pegirian river, 9.8 km^2 of Tambakwedi canal, and 8.0 km^2 of other creeks served by Tjumpat, Kendjeran, and Sukolilo sluices. South Basin has an area of 32.0 km^2 which consists of 2.8 km^2 of Larangan canal, 4.7 km^2 of Kalisari canal, 7.5 km^2 of Kalidami canal, 5.5 km^2 of Keputih canal, and 11.5 km^2 of Medokan canal.

Since the conditions of the two basins are nearly the same, it is assumed that the runoff from each basin may be estimated from the runoff of Kalidami canal using the ratio of catchment areas of either North or South Basin and Kalidami canal, 3.66 or 4.27. The peak discharges of the runoffs from the two basins for several return periods are shown in Table 2.

Table 2 Runoff from North and South Basins

Return period (years)		50	10	5	2	1.05
Runoff (m^3/s)	North Basin	97.4	65.5	52.7	37.0	20.3
	South Basin	113.9	76.5	61.5	43.2	23.7

4. Drainage through the Existing Flap Gates and Landside Inundation.

(1) Hydrograph of inflow to North and South Inundation Basins.

According to the probability curve of daily rainfall at Surabaja, daily rainfall for several return periods are shown in Table 3.

It is assumed that (1) since a daily rainfall is regarded as independent of either of previous and next ones, runoff due to a single

Table 3 Daily Rainfall at Surabaja

Return period (years)	50	10	5	2	1.05
Daily rainfall (mm)	194	156	140	109	69

daily rainfall may be considered, (2) coefficient of total runoff is 0.8 considering that it was 0.76 in the runoff analysis of Morokrembangan boezem, (3) the basic form of discharge hydrograph is triangle, and (4) the peak discharge appears at the time equal to the concentration time.

The hydrographs of inflow to North and South Basins which were obtained on the basis of the above assumptions are shown in Table 4.

Table 4 Inflow Hydrographs for North and South Basins

in m^3/s

	Return period (years)	Time (hr) 0	3	6	41
North basin ($27.4\ km^2$)	50	0	97.4	48.40	0
	10	0	65.5	39.64	0
	5	0	52.7	36.54	0
	2	0	37.0	29.09	0
	1.05	0	20.5	18.88	0
South basin ($32.0\ km^2$)	50	0	113.9	56.50	0
	10	0	76.5	46.30	0
	5	0	61.5	42.65	0
	2	0	43.2	33.96	0
	1.05	0	23.9	22.01	0

(2) Tide curve.

Tide curve outside of the sluices on the sea dike was assumed as a sine curve which has high water level and low water level studied previously and has a period of 12 hr. This is shown in Table 5.

Table 5 Tide Curve

Time (hr)	0	1	2	3	4	5	
Tide level (m, SHVP)	0.373	0.208	-0.241	-0.855	-1.469	-1.918	
Time (hr)	6	7	8	9	10	11	12
Tide level (m, SHVP)	-2.083	-1.918	-1.469	-0.855	-0.241	0.208	0.373

(3) Calculation of inundation for each return period.

Calculation of inundation in each inundation basin was carried out for each case of North and South Basins using the above-mentioned inflow hydrograph of each return period, the tide curve in Table 5, the storage-capacity curves in Fig. 1, discharge equations (6) and (7) for the existing flap gates, and the continuity equation shown below.

$$\left(\frac{I_{t-1} + I_t}{2} - \frac{O_{t-1} + O_t}{2}\right) \Delta t = V_t - V_{t-1} \qquad (8)$$

where I_{t-1}, I_t = inflow at time t-1 and t (m^3/s); O_{t-1}, O_t = outflow through the flap gates to be calculated by the equations (6) and (7) (m^3/s); t = time interval (hr); and V_{t-1}, V_t = storage volume at time t-1 and t (m^3). The widths of the existing sluices which were used in these calculations are shown in Table 6 and the results of calculation are shown in Table 7 and Figs. 9 and 10.

Table 6 Widths of the Existing Sluices on the Sea Dike

	Total width (m)	Dimention of flap gate (BxHxNos. of openings)	
North Basin	14.5	Tambakwedi	5.0 x 3.15 x 2
		Tjumpat	1.5 x 1.5 x 1
		Kendjeran	1.5 x 1.5 x 1
		Sukolilo	1.5 x 1.5 x 1
South Basin	13.05	Larangan	1.5 x11.5 x 1
		Wonosari	3.0 x 1.5 x 1
		Kalidami	2.0 x 1.5 x 1
		Keputih	1.75 x 1.5 x 1
		Medokan	2.4 x 1.5 x 2

Note: B = width of a sluice (m)
H = depth of a sluice (m)

Elevation of ground sill of Tambakwedi sluice is -2.185 m, SHVP.

5. Inundation after Improvement of the Sluices on the Sea Dike.

(1) Improvement of the sluices.

Most of the existing sluices have insufficient capacity for drainage. They are improved as below.

The improved sluices are installed with sluice gates which can move automatically with the head difference between landside and sea. The bottom elevation of Tjumpat, Kendjeran, and Sukolilo sluices facing to the sea is -1.0 m taking the littoral drift into consideration. The bottom elevation of Larangan, Wonosari, Kalidami, and Keputih sluices is -1.5 m and that of New Tambakwedi and Medokan is -2.2 m taking account of respective conditions of the drainage channels.

We have set the three kinds of sluices, type A, B and C as given in Figs. 6, 7 and 8. Type A is applied to the first group of sluices mentioned above, type B is applied to the second group, and type C is applied to the third group, as shown in Table 8.

Table 7 Water Level of Inundation

	South Basin (m. SHVP)				
time	1/50	1/10	1/5	1/2	1/1.05
0	-0.860	-0.860	-0.860	-0.860	-0.860
1	-0.086	-0.125	-0.141	-0.160	-0.181
2	0.095	0.043	0.011	-0.036	-0.109
3	0.224	0.161	0.125	0.075	-0.007
4	0.284	0.239	0.208	0.151	0.064
5	0.312	0.276	0.253	0.202	0.103
6	0.329	0.303	0.276	0.231	0.132
7	0.340	0.317	0.295	0.252	0.149
8	0.349	0.329	0.306	0.260	0.156
9	0.358	0.339	0.314	0.268	0.161
10	0.367	0.349	0.322	0.276	0.166
11	0.375	0.358	0.328	0.282	0.169
12	0.384	0.369	0.336	0.291	0.178
13	0.395	0.380	0.345	0.301	0.194
14	0.406	0.390	0.354	0.307	0.208
15	0.417	0.400	0.364	0.314	0.222
16	0.427	0.410	0.373	0.321	0.235
17	0.437	0.418	0.380	0.326	0.244
18	0.445	0.424	0.386	0.231	0.250
19	0.451	0.428	0.390	0.333	0.251
20	0.454	0.429	0.391	0.333	0.251
21	0.457	0.429	0.391	0.333	0.242
22	0.460	0.427	0.391	0.332	0.237
23	0.461	0.425	0.390	0.332	0.231
24	0.465	0.425	0.391	0.332	0.230
25	0.470	0.425	0.392	0.334	0.233
26	0.475	0.426	0.394	0.334	0.233
27	0.479	0.427	0.395	0.338	0.244
28	0.484	0.428	0.396	0.340	0.250
29	0.487	0.428	0.397	0.341	0.251
30	0.490	0.428	0.396	0.341	0.251
31	0.490	0.425	0.394	0.339	0.247
32	0.487	0.420	0.389	0.335	0.236
33	0.484	0.415	0.385	0.331	0.225
34	0.481	0.410	0.380	0.326	0.214
35	0.477	0.405	0.375	0.322	0.202
36	0.475	0.401	0.372	0.319	0.192
37	0.475	0.400	0.370	0.318	0.186
38	0.474	0.398	0.369	0.317	0.185
39	0.473	0.397	0.369	0.316	0.187
40	0.472	0.396	0.368	0.316	0.188
41	0.470	0.394	0.366	0.314	0.184
42	0.467	0.392	0.364	0.312	0.174
43	0.463	0.388	0.360	0.308	0.158
44	0.456	0.382	0.354	0.303	0.133
45	0.450	0.376	0.348	0.296	0.106
46	0.443	0.369	0.342	0.287	0.077
47	0.436	0.363	0.336	0.278	0.046
48	0.432	0.359	0.333	0.272	0.014
49	0.429	0.357	0.130	0.268	-0.003
50	0.427	0.355	0.329	0.265	-0.012
51	0.425	0.354	0.328	0.265	-0.012
52	0.424	0.353	0.327	0.264	-0.012
53	0.421	0.351	0.325	0.261	-0.020
54	0.419	0.349	0.323	0.258	-0.039
55	0.414	0.345	0.319	0.252	-0.085
56	0.408	0.340	0.314	0.237	-0.157
57	0.401	0.334	0.308	0.222	-0.200
58	0.395	0.328	0.303	0.207	
59	0.389	0.323	0.296	0.187	
60	0.385	0.319	0.290	0.171	
61	0.382	0.317	0.286	0.161	
62	0.380	0.315	0.283	0.157	
63	0.379	0.314	0.282	0.157	
64	0.378	0.314	0.281	0.157	
65	0.376	0.312	0.279	0.152	
66	0.374	0.310	0.275	0.142	
67	0.369	0.306	0.269	0.123	
68	0.363	0.301	0.260	0.096	
69	0.357	0.292	0.251	0.066	
70	0.352	0.283	0.237	0.030	
71	0.346	0.274	0.221	-0.018	
72	0.342	0.268	0.211	-0.056	
73	0.340	0.264	0.205	-0.082	
74	0.338	0.262	0.202	-0.093	
75	0.337	0.261	0.202	-0.093	
76	0.336	0.260	0.202	-0.093	
77	0.335	0.258	0.198	-0.104	
78	0.332	0.254	0.189	-0.129	
79	0.328	0.247	0.172	-0.176	
80	0.323	0.231	0.149	-0.200	
81	0.317	0.216	0.122		
82	0.312	0.200	0.095		
83	0.306	0.177	0.065		
84	0.303	0.161	0.042		
85	0.301	0.152	0.025		
86	0.298	0.147	0.017		
87	0.297	0.147	0.017		
88	0.296	0.147	0.017		
89	0.293	0.141	0.010		
90	0.290	0.130	-0.008		
91	0.283	0.112	-0.044		
92	0.274	0.083	-0.115		
93	0.266	0.054	-0.184		
94	0.257	0.011	-0.200		
95	0.246	-0.040			
96	0.235	-0.088			
97	0.229	-0.113			
98	0.225	-0.124			
99	0.225	-0.124			
100	0.224	-0.124			
101	0.220	-0.133			
102	0.188	-0.156			
103	0.157	-0.200			
104	0.126				
105	0.094				
106	0.063				
107	0.031				
108	-0.030				
109	-0.080				
110	-0.100				
111	-0.110				
112	-0.110				
113	-0.120				
114	-0.130				
115	-0.150				
116	-0.200				

	North Basin (m. SHVP)				
time	1/50	1/10	1/5	1/2	1/1.05
0	-0.860	-0.860	-0.860	-0.860	-0.860
1	-0.116	-0.148	-0.161	-0.177	-0.192
2	0.098	0.037	-0.004	-0.056	-0.140
3	0.199	0.152	0.123	0.078	-0.020
4	0.252	0.210	0.185	0.145	0.066
5	0.280	0.243	0.217	0.175	0.102
6	0.298	0.264	0.237	0.194	0.110
7	0.307	0.280	0.253	0.205	0.111
8	0.315	0.294	0.263	0.212	0.109
9	0.322	0.305	0.272	0.218	0.106
10	0.328	0.313	0.280	0.224	0.103
11	0.335	0.320	0.288	0.228	0.097
12	0.341	0.327	0.295	0.233	0.093
13	0.348	0.334	0.302	0.239	0.097
14	0.356	0.342	0.309	0.249	0.112
15	0.365	0.352	0.317	0.259	0.134
16	0.375	0.360	0.325	0.267	0.154
17	0.381	0.365	0.330	0.272	0.160
18	0.386	0.368	0.332	0.274	0.158
19	0.388	0.368	0.333	0.273	0.152
20	0.390	0.368	0.333	0.272	0.139
21	0.392	0.366	0.332	0.269	0.126
22	0.393	0.364	0.330	0.267	0.112
23	0.393	0.361	0.328	0.264	0.095
24	0.394	0.357	0.326	0.261	0.074
25	0.395	0.355	0.325	0.259	0.061
26	0.398	0.353	0.324	0.259	0.068
27	0.401	0.354	0.325	0.262	0.089
28	0.405	0.355	0.326	0.265	0.105
29	0.406	0.353	0.324	0.264	0.106
30	0.405	0.349	0.321	0.259	0.089
31	0.402	0.344	0.316	0.253	0.058
32	0.399	0.339	0.311	0.244	0.001
33	0.395	0.333	0.305	0.234	-0.071
34	0.391	0.327	0.299	0.223	-0.159
35	0.386	0.321	0.290	0.212	-0.200
36	0.382	0.315	0.283	0.202	
37	0.378	0.310	0.276	0.190	
38	0.375	0.307	0.272	0.185	
39	0.375	0.306	0.272	0.187	
40	0.373	0.305	0.270	0.187	
41	0.369	0.302	0.266	0.181	
42	0.364	0.295	0.258	0.166	
43	0.357	0.286	0.249	0.146	
44	0.350	0.276	0.236	0.115	
45	0.343	0.266	0.223	0.075	
46	0.335	0.256	0.210	0.005	
47	0.328	0.246	0.195	-0.104	
48	0.321	0.233	0.176	-0.193	
49	0.316	0.224	0.161	-0.200	
50	0.312	0.218	0.155		
51	0.311	0.217	0.155		
52	0.309	0.217	0.155		
53	0.306	0.211	0.147		
54	0.300	0.202	0.124		
55	0.291	0.184	0.094		
56	0.281	0.164	0.037		
57	0.272	0.140	-0.049		
58	0.262	0.110	-0.171		
59	0.252	0.065	-0.200		
60	0.241	-0.005			
61	0.231	-0.075			
62	0.225	-0.111			
63	0.224	-0.111			
64	0.223	-0.111			
65	0.218	-0.134			
66	0.208	-0.193			
67	0.194	-0.200			
68	0.174				
69	0.154				
70	0.125				
71	0.090				
72	0.039				
73	-0.023				
74	-0.047				
75	-0.047				
76	-0.047				
77	-0.085				
78	-0.160				
79	-0.200				

Fig. 6

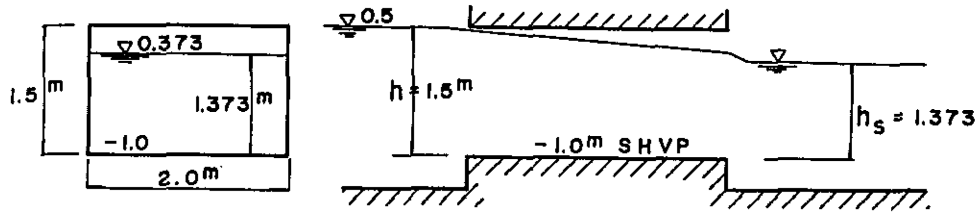


Fig. 7

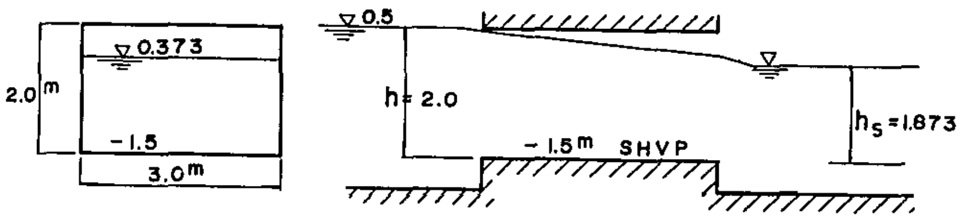


Fig. 8

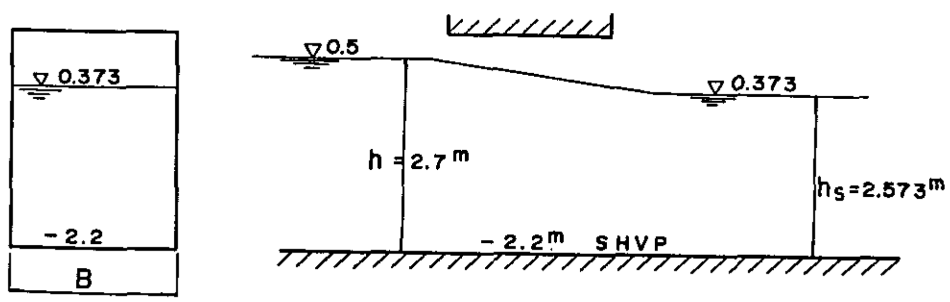


Fig.9 Variation of Water Level (South basin, Preseut condition)

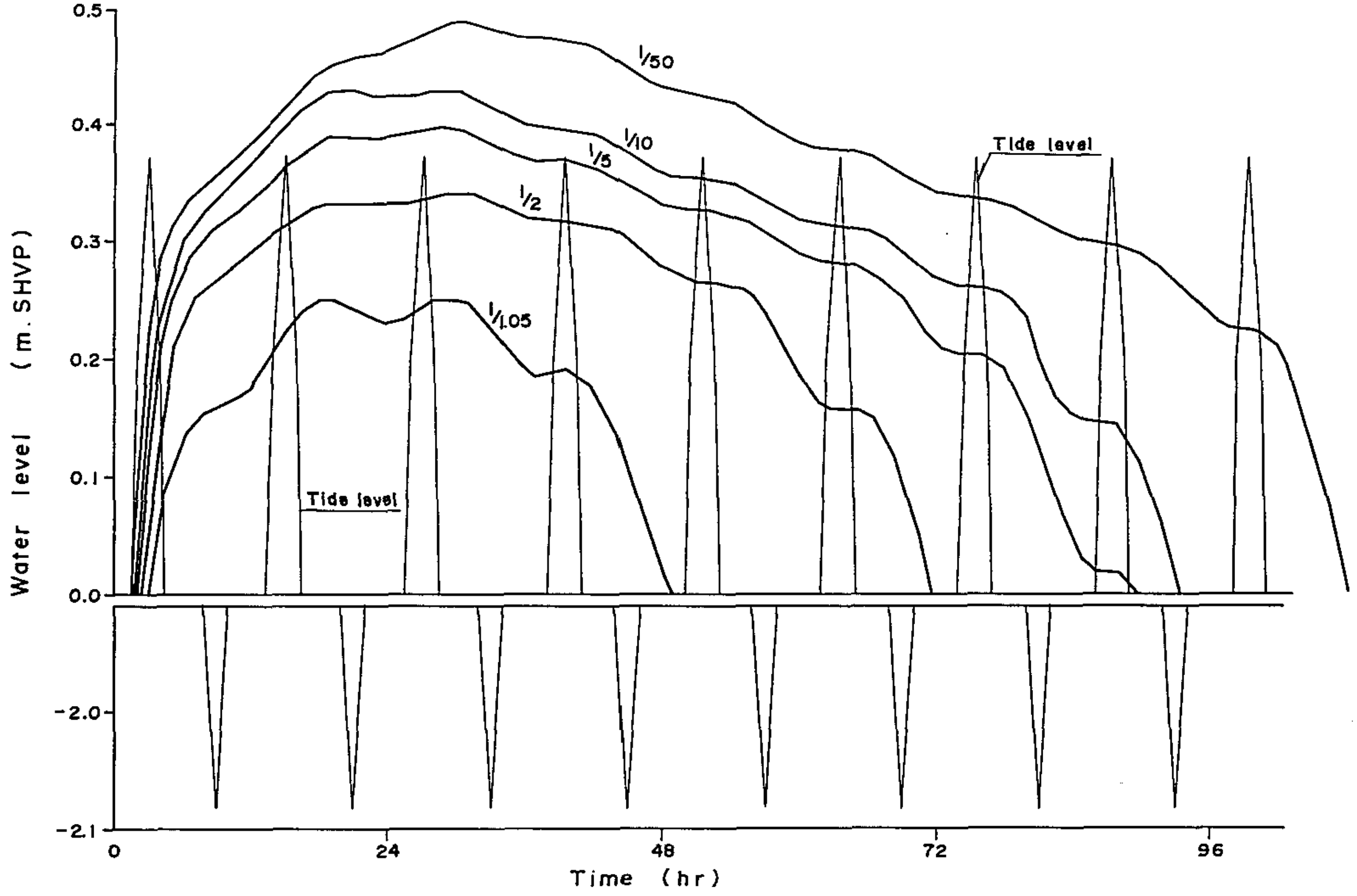


Fig. 10 Variation of Water Level (North basin, Present condition)

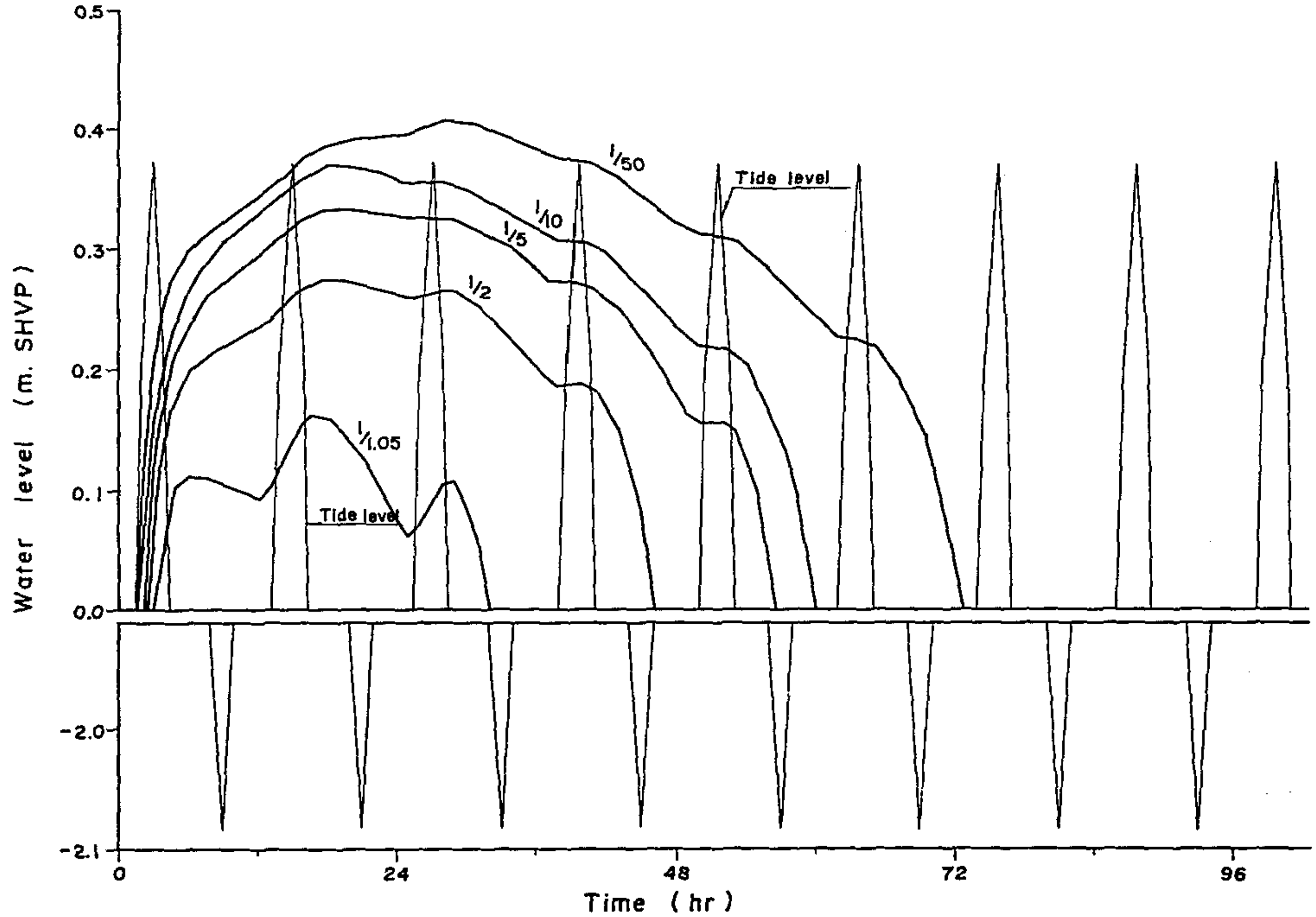


Table 8 Improved Sluices

Name of sluice	Catchment area A (km^2)	Peak runoff (m^3/s) [3]	Gate	Remarks
Tambakwedi	9.6	36.9	Existing one	[1]
New Tambakwedi	13.0	25.0	Type C_1 1 opening x 5.0 m	[2]
Tjumpat			Type A 1 opening x 2 m	[2]
Kendjeran	4.8	9.2	Type A 1 opening x 2 m	[2]
Sukolilo			Type A 1 opening x 2 m	[2]
Larangan	2.8	5.4	Type B_1 1 opening x 3 m	[2]
Wonosari	4.7	9.0	Type B_1 1 opening x 3 m	[2]
Kalidami	7.5	14.4	Type B_2 2 openings x 3 m	[2]
Keputih	5.5	10.6	Type B_1 1 opening x 3 m	[2]
Medokan	11.5	22.1	Type C_2 2 openings x 3 m	[2]

[1] f = 0.4
This sluice is used only for the Pegirian river.
The present carrying capacity is 37.4 m^3/s.

[2] f = 0.2
Newly built

[3] Peak runoff from individual drainage basin which the respective sluice serves.

If we assume that landside water flows out through a sluice with a landside water level of +0.5 m and a tide level of +0.373 m, the discharges for type A, B and C are calculated by the equation (7) where we take $\beta = 1$ because the gate shall be opened fully differing from flap gate type.

$Q = 4.0\ m^3/s$ for a type-A sluice

$Q = 8.18\ m^3/s$ for a type-B sluice

$Q = 3.74\ m^3/s$ for one meter of the width of type-C sluice

We have calculated the 5-year peak runoff of the individual drainage basin shown in Table 8 in direct proportion to the catchment area of the Kalidami basin and then, by using these peak discharge, we have found the required width or openings of the sluice, which is also shown in Table 8.

(2) Drainage of the runoff at each return period and landside inundation.

The results of calculation of drainge through the improved sluices are shown in Table 9 and Figs. 11 and 12.

Table 9 Water Level of Inundation

Time	South Basin (m. SHVP)					North Basin (m. SHVP)				
	1/50	1/10	1/5	1/2	1/1.05	1/50	1/10	1/5	1/2	1/1.05
0	-0.860	-0.860	-0.860	-0.860	-0.860	-0.860	-0.860	-0.860	-0.860	-0.860
1	-0.160	-0.186	-0.196	-0.200	-0.200	-0.182	-0.200	-0.200	-0.200	-0.200
2	0.050	-0.022	-0.058	-0.119	-0.184	0.040	-0.043	-0.100	-0.152	-0.200
3	0.204	0.132	0.096	0.044	-0.055	0.179	0.128	0.101	0.040	-0.081
4	0.267	0.219	0.191	0.127	0.035	0.235	0.200	0.171	0.125	0.035
5	0.285	0.248	0.218	0.149	0.031	0.249	0.215	0.184	0.128	0.003
6	0.272	0.247	0.207	0.109	0.005	0.230	0.198	0.150	0.095	-0.020
7	0.240	0.238	0.185	0.098	-0.015	0.196	0.157	0.130	0.076	-0.050
8	0.185	0.198	0.148	0.080	-0.050	0.158	0.140	0.120	0.040	-0.090
9	0.176	0.170	0.138	0.060	-0.095	0.150	0.128	0.092	-0.015	-0.140
10	0.160	0.155	0.115	0.035	-0.135	0.130	0.105	0.045	-0.080	-0.180
11	0.135	0.130	0.090	-0.015	-0.170	0.103	0.065	-0.005	-0.140	-0.200
12	0.110	0.110	0.055	-0.070	-0.190	0.075	0.025	-0.060	-0.180	
13	0.075	0.075	0.015	-0.125		0.055	-0.030	-0.125	-0.200	
14	0.065	0.030	-0.035	-0.170		0.035	-0.100	-0.190		
15	0.050	-0.020	-0.095	-0.185		0.005	-0.150	-0.200		
16	0.028	-0.090	-0.145	-0.200		-0.025	-0.195			
17	0.005	-0.155	-0.180			-0.060				
18	-0.025	-0.180	-0.200			-0.095				
19	-0.055	-0.200				-0.130				
20	-0.080					-0.160				
21	-0.106					-0.190				
22	-0.135					-0.200				
23	-0.157									
24	-0.180									
25	-0.200									
26										

Fig. 11 Variation of Water Level (North bosin, Gates improved)

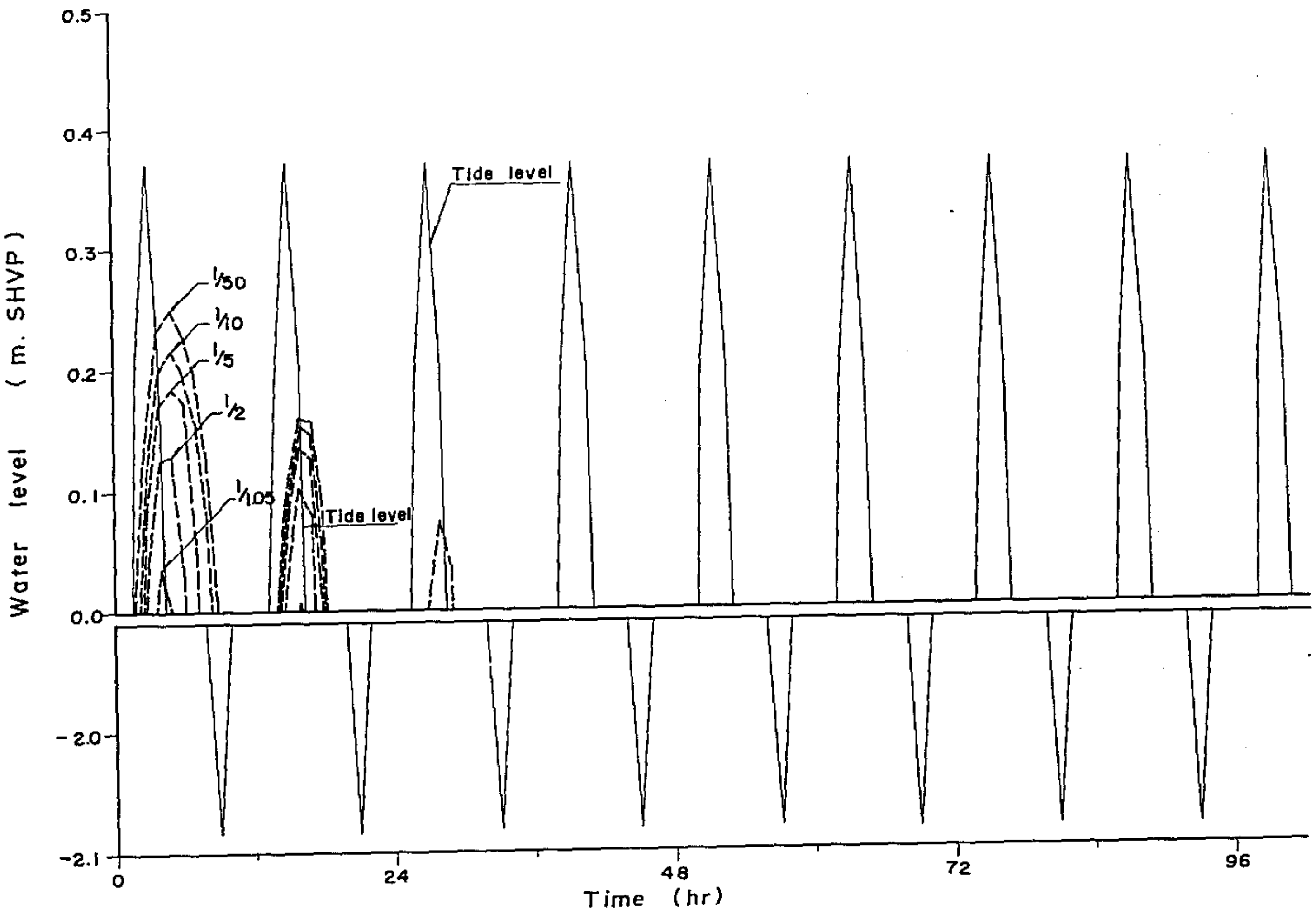
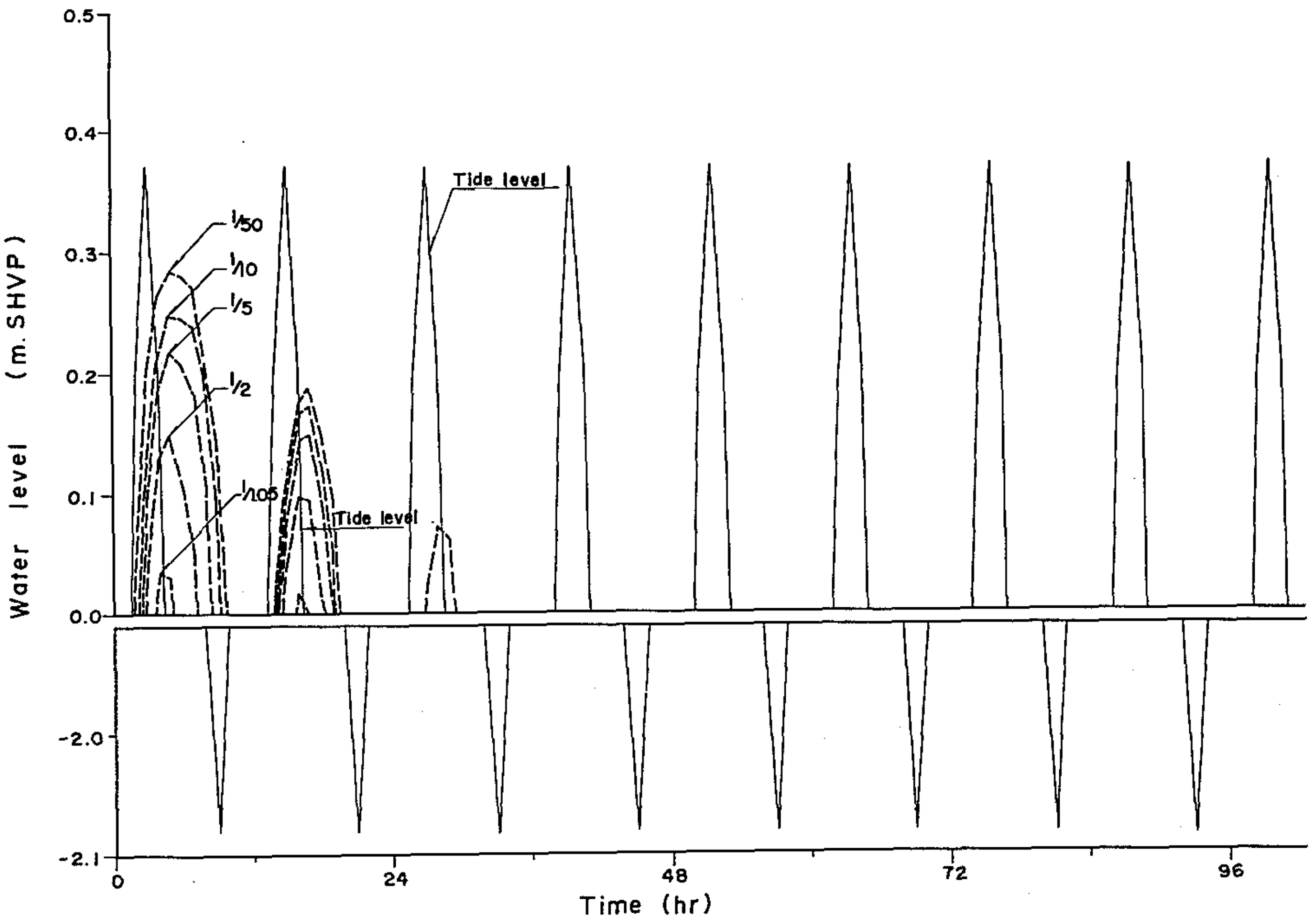


Fig. 12 Variation of Water Level (South basin, Gates improved)



CHAPTER XVII

INUNDATIONS DUE TO POSSIBLE FLOODS IN THE LOWER BASIN OF THE MARMOJO RIVER

1. General.

The lower reaches of the Marmojo river have their own dikes on the left and right sides of the river channel over the extent from Sidogede to Klubuk. The dikes, however, have been devastated, scoured on the riverside slope, lowered in the height at several parts, and is endangered against flood flows.

In this chapter, flood damages which may be caused by overtopping of flood water of the Marmojo are analyzed and reduction of damages by improvement of the Marmojo river are studied.

Examining the longitudinal profile of the lower reaches of the Marmojo river, flood flow was assumed to overtop at a section No. 45 of the right-side dike. Plan of the right-side part of the lower basin of the Marmojo river is shown in Fig. 1. The right-side dike of the Marmojo river, a rail way along the Watudakon river and a road from Kupang to Modjokerto are taken as the boundaries of the lower basin which may suffer from floods due to overtopping. This basin is called "Right Lower Basin" for the convenience of description. Contour lines in Fig. 1 were drawn on the basis of topographic maps and results of surveying of the Marmojo river and the Wonoaju river.

Fig. 2 shows longitudinal profile of the Marmojo river around the section No. 45. It is assumed that when water level reaches 17.37 m, SHVP, flood flow begins to overtop and scoures the top of the dike until the height of dike becomes 16.49 m, SHVP.

2. Discharge Hydrograph.

(1) Discharge hydrograph of the Marmojo river.

Discharge hydrograph of the Marmojo river at Klubuk was assumed in the same way as that described in Chapter XIII, Part 4.

a. Average rainfall over the drainage area.

Average rainfall over the drainage area upstream of Klubuk was calculated on the basis of the data of raingage stations at Tandjung, Kabuh Djatisari, Tapen, Gedek and Terusan.

Return period (years)	50	20	10	5	2
Average rainfall (mm)	149.4	135.4	124.1	111.8	91.6

Fig. I Right Lower Basin of the Marmojo River

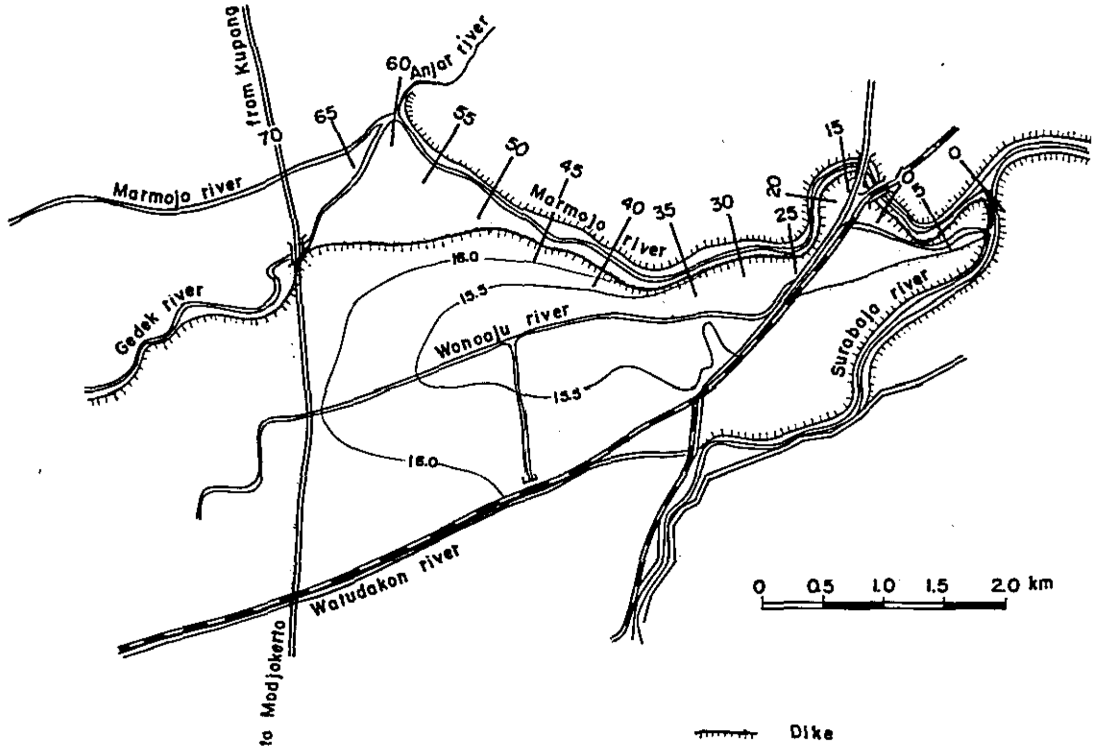
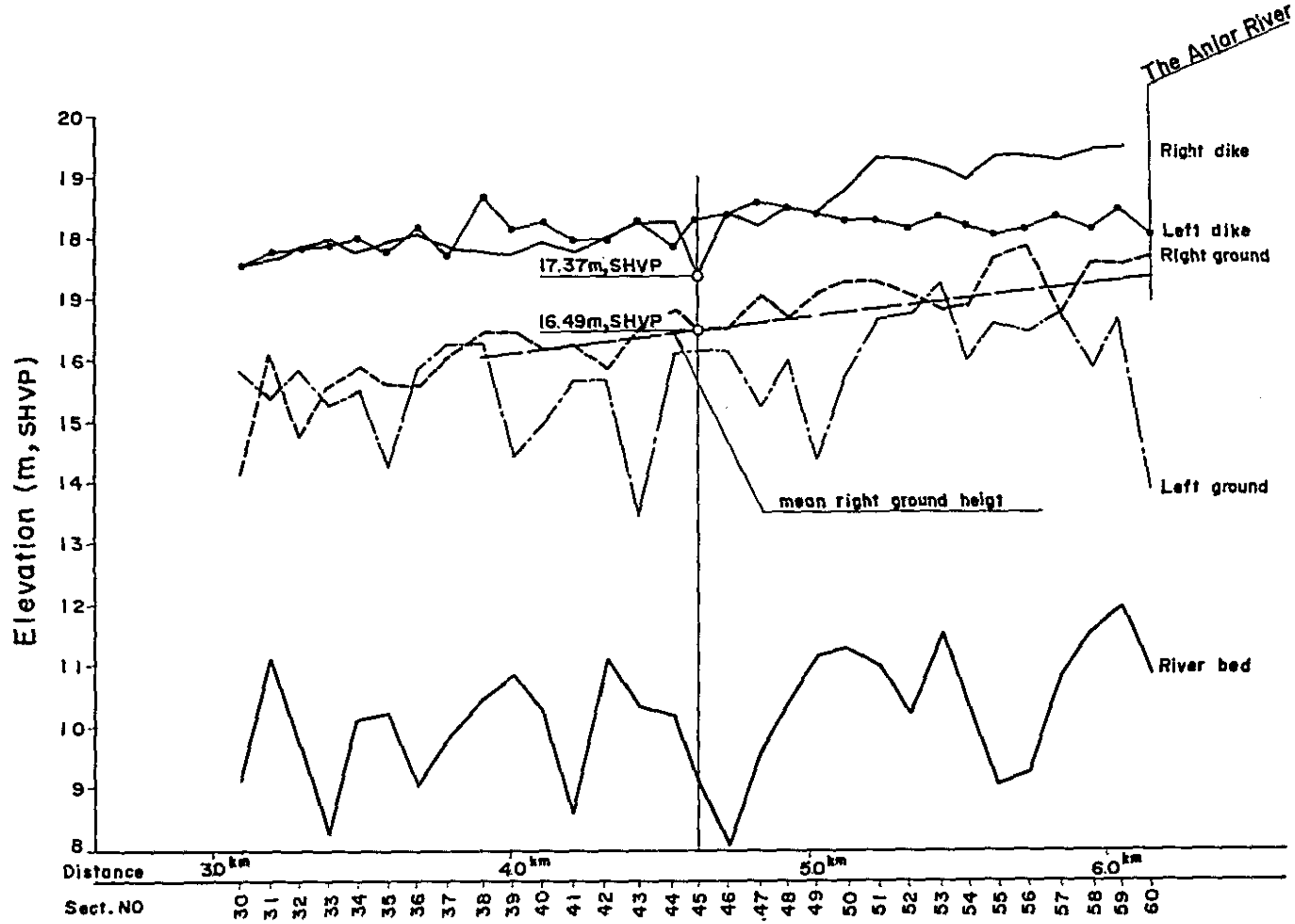


Fig.2 Longitudinal Profile of the Marmojo River Around NO.45



b. Peak discharge.

Peak discharge at Klubuk was estimated according to the following equation;

$$Q = K\sqrt{A}$$

where A = 277.1 km^2 and $K = Q_{Mer}/\sqrt{A_{Mer}}$ as shown in Table 3, Chapter XIII, Table 4.

Return period (years)	50	20	10	5	2
Peak discharge (m^3/s)	254	222	199	174	135

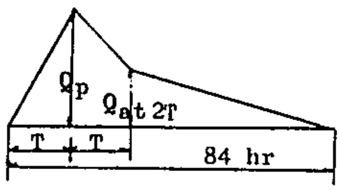
c. Time of concentration.

Time of concentration at Klubuk was estimated by Kraven's table and mean velocity in river channel as shown in Table 3, Chapter XVIII, Part 4.

Return period (years)	50	20	10	5	2
Time of concentration (hr)	6.4	6.7	6.9	7.2	7.6

d. Discharge hydrograph.

Discharge hydrograph of drainage area upstream of Klubuk was assumed as follows;



(a) Total volume of discharge hydrograph (V) was calculated for each return period as follows;

$$V = fRA$$

where f = runoff coefficient (= 0.8),
R = average daily rainfall over the drainage area,
A = drainage area upstream of Klubuk (= 277.1 km^2).

(b) Duration of flood was determined at 84 hrs or 3.5 days after some considerations.

(c) Discharge at t = 0 and t = 84 hrs are zero.

(d) Peak discharge occurs at t = T which denotes the time of concentration.

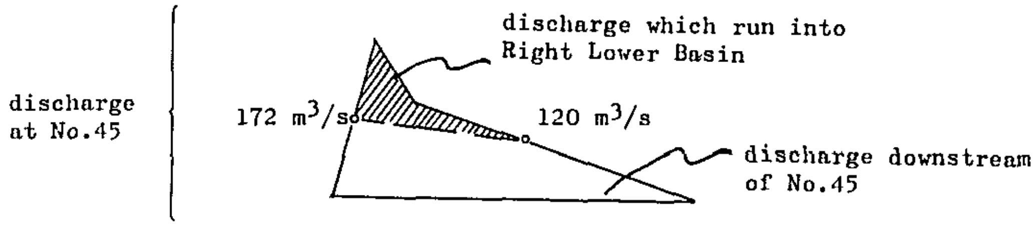
(e) Discharge at t = 2T was determined so that the total volume of hydrograph may be the same as that calculated from rainfall (V).

Result of estimation for each return period is as follows, and is shown in Fig. 3.

Return period (years)	Total volume of runoff ($\times 10^6 m^3$)	Discharge (m^3/s)			
		t = 0	t = T	t = 2T	t = 84 hr.
50	33.15	0	254	195	0
20	30.01	0	222	177	0
10	27.51	0	199	165	0
5	24.81	0	174	147	0
2	20.31	0	135	121	0

(2) Discharge hydrograph of flow which runs into Right Lower Basin.

Stage-discharge curve at the section No. 45 shown in Fig. 4 was obtained by nonuniform flow calculation. Discharges at water levels 17.37 and 16.49 m, SHVP are 172 and 120 m^3/s respectively, which means that overflow begins when the discharge of the Marmojo river has reached 172 m^3/s and finishes when the discharge has decreased to 120 m^3/s. Thus, the discharge hydrograph of the flow-running into Right Lower Basin was assumed as below;



Such hydrographs for five kinds of return period are shown in Fig. 5. Discharge hydrograph for 5-year flood is obviously negligible. Total volume of the hydrograph is as follows;

Return period (years)	50	20	10	5	2
Total volume ($\times 10^6 m^3$)	3.442	1.772	0.786	0	0

(3) Discharge hydrograph due to rainfall in Right Lower Basin.

Average daily rainfall over Right Lower Basin was taken the same as that over the drainage area upstream of Klubuk described in 2-(1)-a in this chapter.

On the other hand, accumulation curve of hourly rainfall was determined as follows, examining hourly-rainfall accumulation curves at recording-rain-gage stations in Java Island;

Fig.3 Discharge Hydrograph at Klubuk

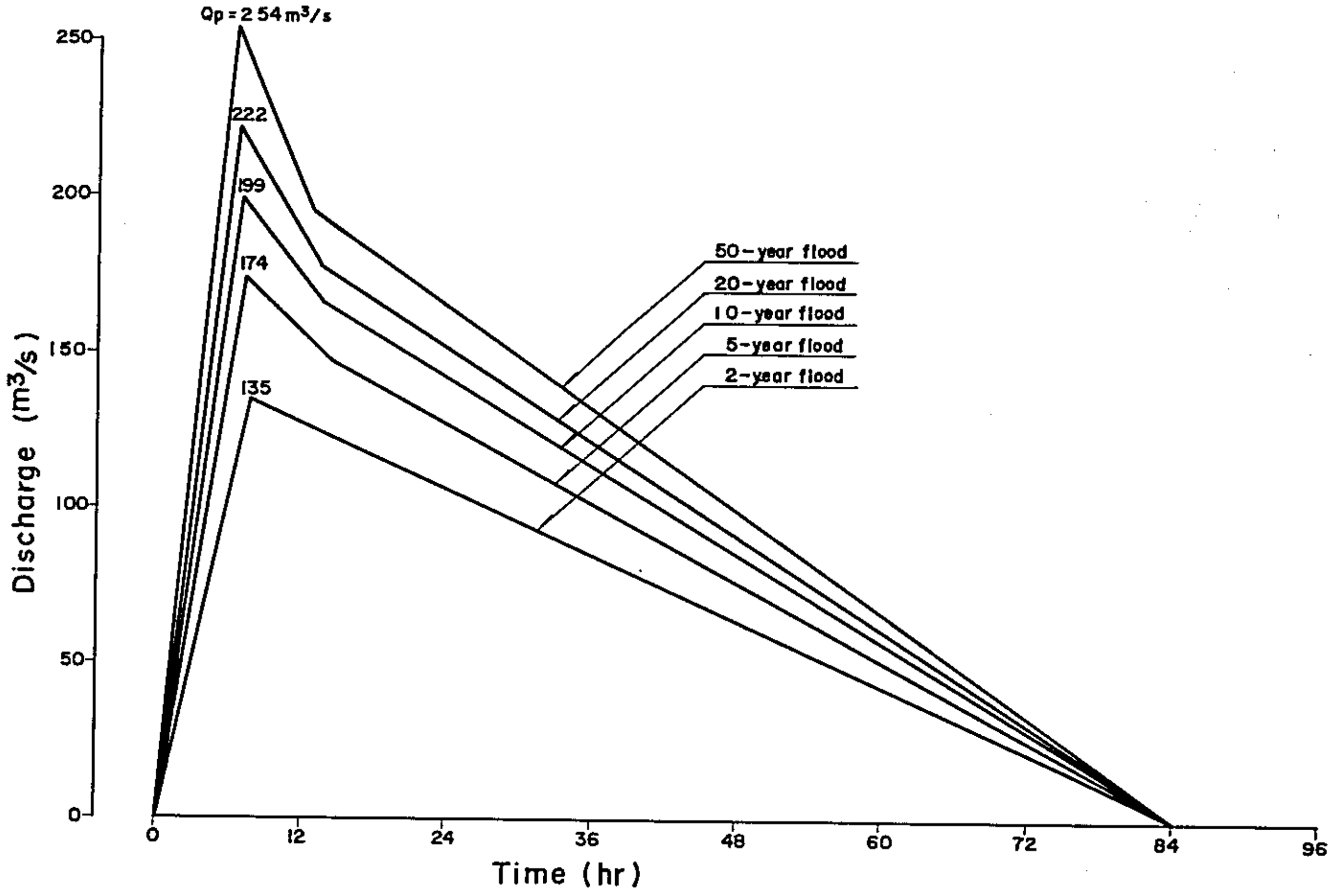


Fig. 4 Stage- Discharge Curve of the Marmojo River at NO.45

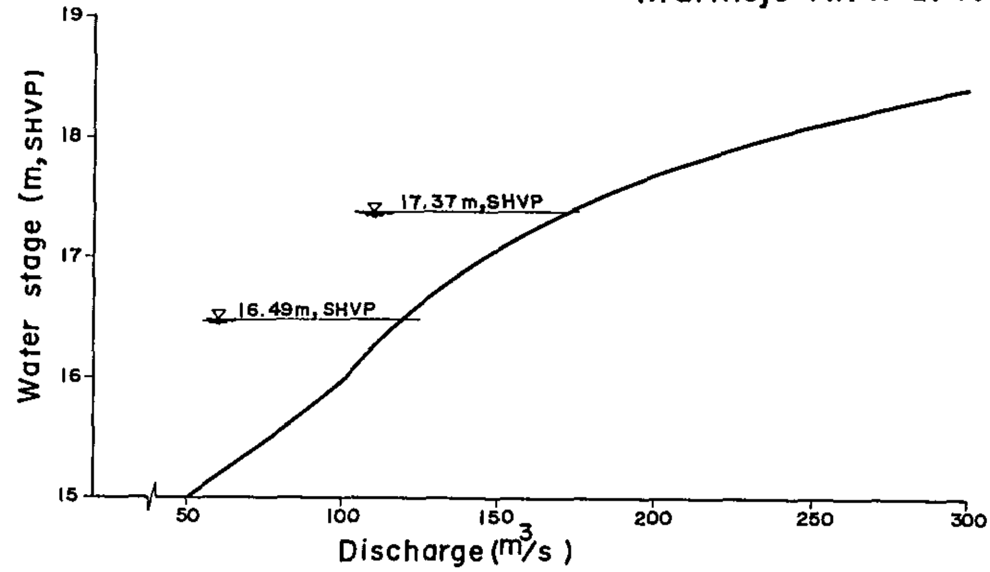
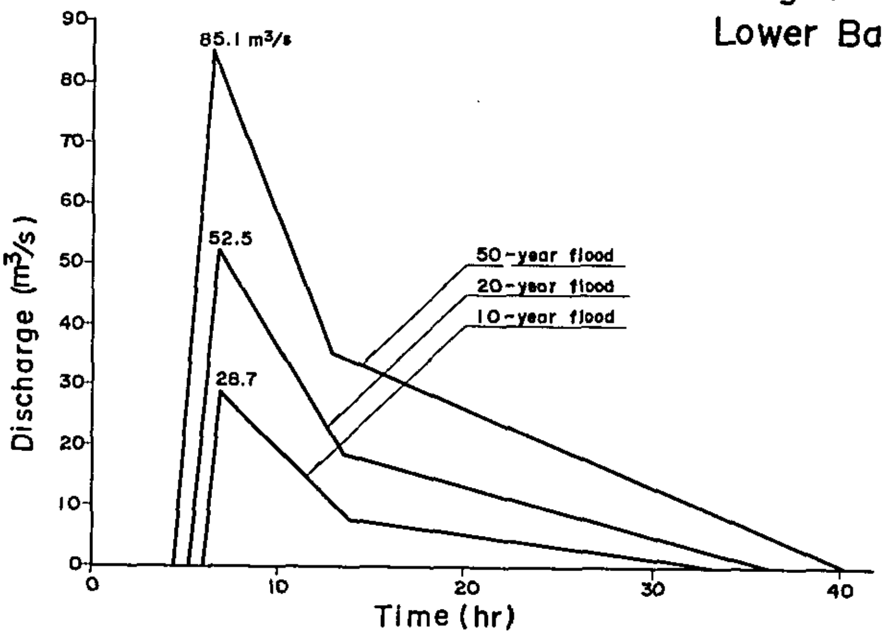


Fig. 5 Discharge Which Runs into Right Lower Basin



Time t (hr)		1	2	3	4	5	6	7	8
Accumulation curve (%)		30	56	73	83	91	96	98	100
Cumulative rainfall (mm)	50 year	44.8	83.7	109.1	124.0	136.0	143.4	146.4	149.4
	20 year	40.6	75.8	98.8	112.4	123.2	130.0	132.7	135.4
	10 year	37.2	69.5	90.6	103.0	112.9	119.1	121.6	124.1
	5 year	33.5	62.6	81.6	92.8	101.7	107.3	109.6	111.8
	2 year	27.5	51.3	66.9	76.0	83.4	87.9	89.8	91.6

Discharge at time t was calculated according to the following equation;

$$Q_i = \frac{1}{3.6} f r_i A$$

where Q_i = discharge at t = i (m^3/s),
f = runoff coefficient (= 0.8),
r_i = hourly rainfall between t = i and i - 1 (mm/hr),,
A = drainage area of Right Lower Basin (= 6.8 km^2).

The results are shown in Table 1 and Fig. 6. The total volume of each discharge hydrograph is as follows;

Return period (years)	50	20	10	5	2
Total volume ($\times 10^6 m^3$)	0.813	0.737	0.675	0.608	0.498

3. Storage-capacity curve.

Storage-capacity curve of Right Lower Basin was obtained on the basis of the contour lines in Fig. 1 mentioned in section 1. The lowest bottom of the basin is 15.0 m, SHVP in elevation. Inundated area and storage-capacity curve are shown in Fig. 7.

4. Depth of Submergence.

The Wonoaju river which is the only drainage channel of Right Lower Basin joins to the Surabaja river near Sidogede where the Marmojo river unites with the Surabajh river. Stage-discharge curve at Sidogede was obtained by nonuniform flow calculation of the Surabaja river and shown in Fig. 8. According to Fig. 8, the water level at the lower end of the Wonoaju river is 14.94 m, SHVP when the discharge of the Marmojo river is 120 m^3/s. This indicates that the drainage of Right Lower Basin is almost impossible as long as the discharges of the Marmojo river is more than 120 m^3/s because of the following reasons;

a. The elevation of the lowest bottom of the basin is 15.0 m, SHVP.

b. Length of the drainage channel from the lower end to inundated area of Right Lower Basin along the Wonoaju river is more than 2 km. Hence, the lose of head due to friction of the channel must make the drainage more difficult.

Table. I Discharge due to Rainfall in Right Lower Basin

(unit: m^3/s)

Return Period (years)		50	20	10	5	2
Time (hr)	0	0	0	0	0	0
	1	67.6	61.3	56.2	50.6	41.5
	2	58.8	53.2	48.8	43.9	36.0
	3	38.4	34.7	31.8	28.7	23.5
	4	22.5	20.6	18.8	17.0	13.8
	5	18.1	16.3	14.9	13.4	11.2
	6	11.2	10.3	9.4	8.5	6.8
	7	4.6	4.1	3.8	3.5	2.8
	8	4.5	4.0	3.8	3.3	2.8
	9	0	0	0	0	0

Fig. 6 Discharge due to Rainfall in Right Lower Basin

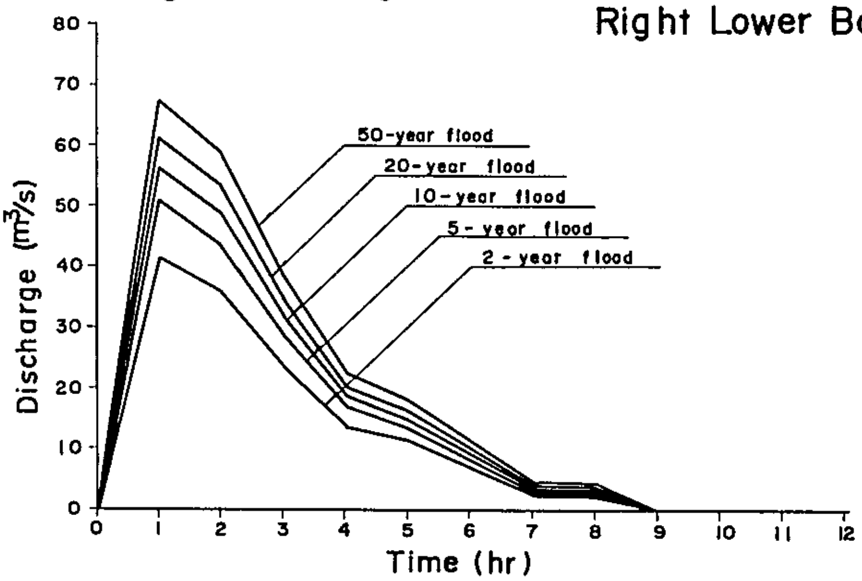


Fig.7 Storage-Capacity Curve

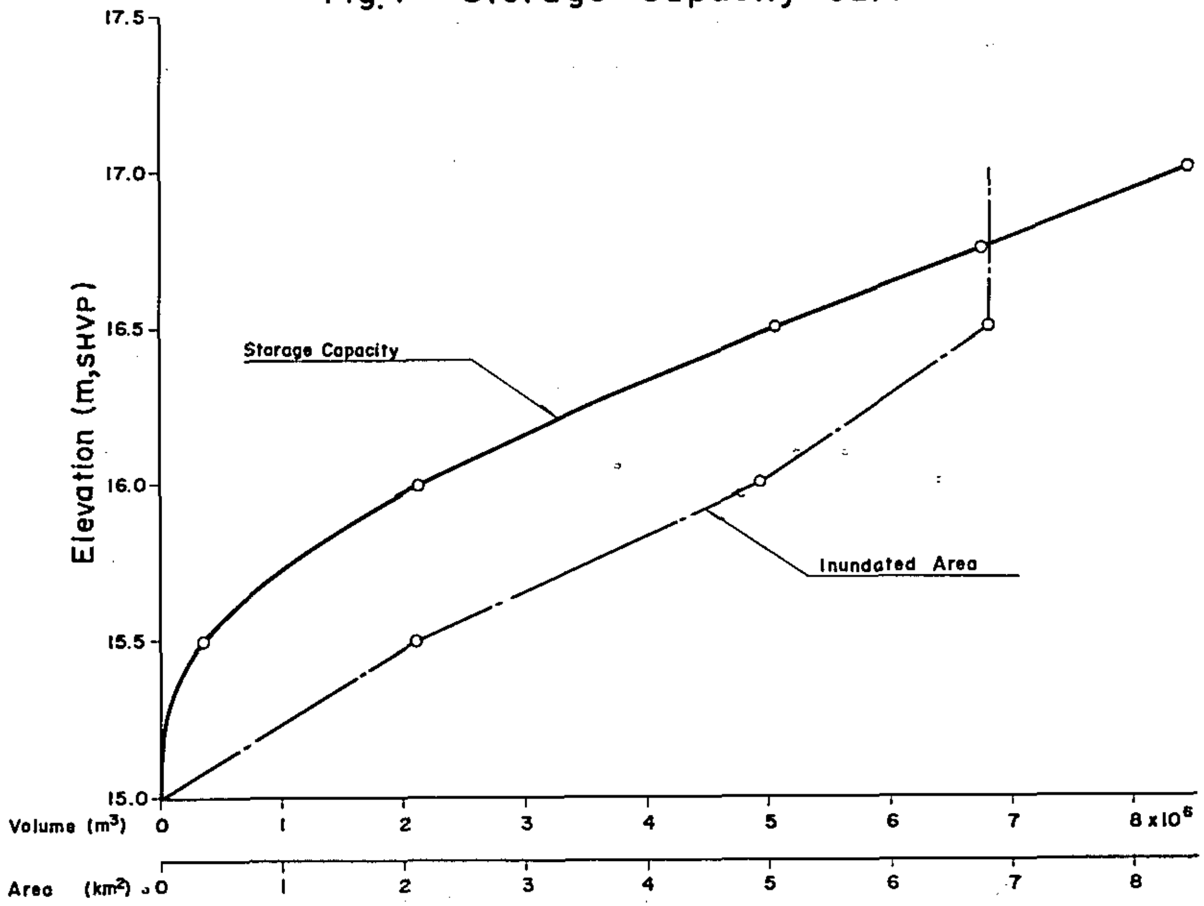
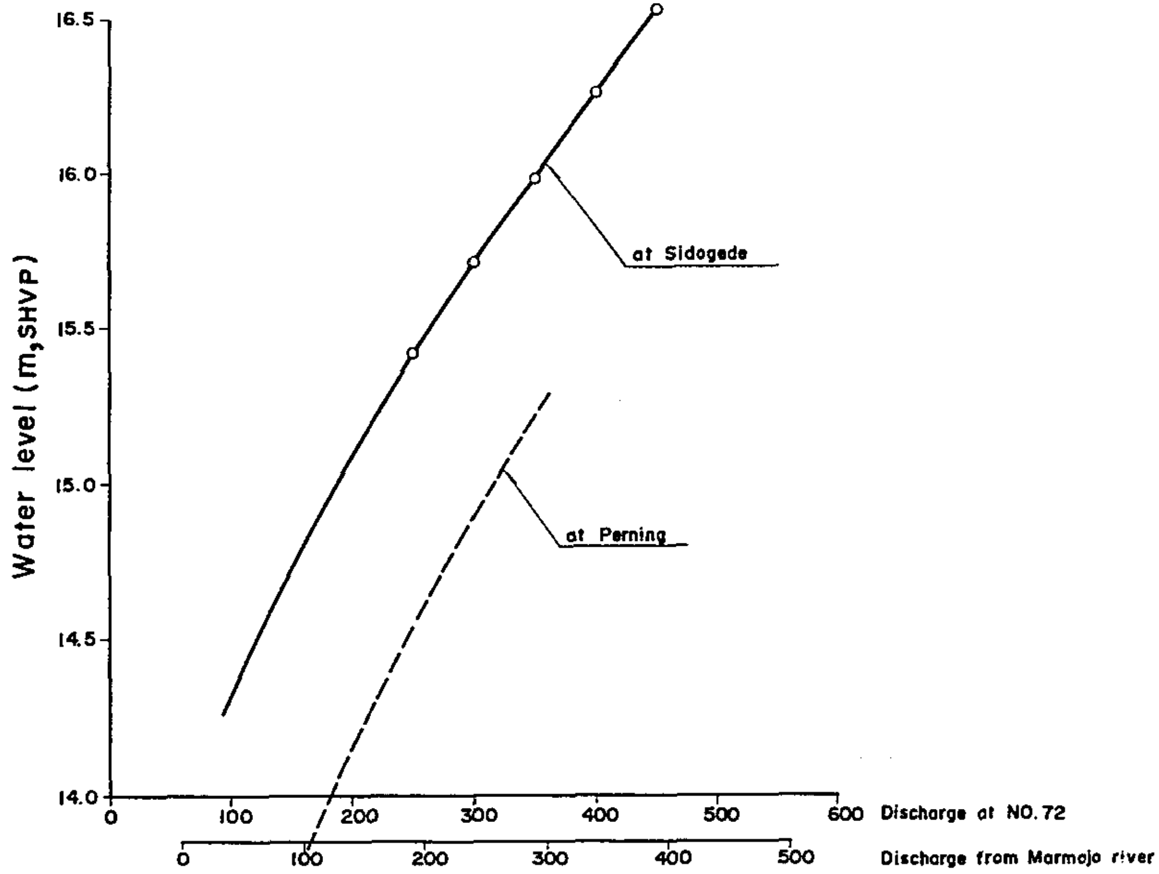


Fig.8 Stage- Discharge Curve at Sidogede



Therefore, depth of submergence and stage hydrograph in Right Lower Basin were calculated under the following conditions and assumptions:

a. Water which runs into Right Lower Basin from the Marmojo river and rain water in the basin cannot be drained by the Wonoaju river until discharge of the Marmojo river reduces to 120 m^3/s.

b. Drainage of Right Lower Basin begins when discharge of the Marmojo river reduces to 120 m^3/s and finishes in 84 hrs (3.5 days) after the beginning of the rainfall.

c. Recession curve of stage hydrograph of inundation in Right Lower Basin is straight.

d. Water surface in inundated area is level.

Inundation water level at time t was calculated from cumulative inflow up to the time t, making use of storage capacity curve (Fig. 7). Calculations were made for both cases shown below.

(1) Before improvement of the Marmojo river.

Inflow to Right Lower Basin are the water which overflows from the Marmojo river and the rain water in the basin.

(2) After improvement of the Marmojo river.

Inflow to Right Lower Basin is only the rain water in the basin.

Results are shown in Fig. 9 and the maximum depths of submergence in each case are as follows;

Case	Return period (years)	Total volume ($x10^6m^3$)	Highest water level (m, SHVP)	Maximum depth (m)	Inundation area (km^2)
Before improvement	50	4.255	16.37	1.37	6.30
	20	2.509	16.07	1.07	5.20
	10	1.461	15.85	0.85	4.10
	5	0.608	15.61	0.61	2.75
	2	0.496	15.56	0.56	2.45
After improvement	50	0.813	15.68	0.68	3.10
	20	0.737	15.65	0.65	2.95
	IO	0.675	15.63	0.63	2.85
	5	0.608	15.61	0.61	2.75
	2	0.498	15.56	0.56	2.45

Fig.9 Stage Hydrograph in Right Lower Basin