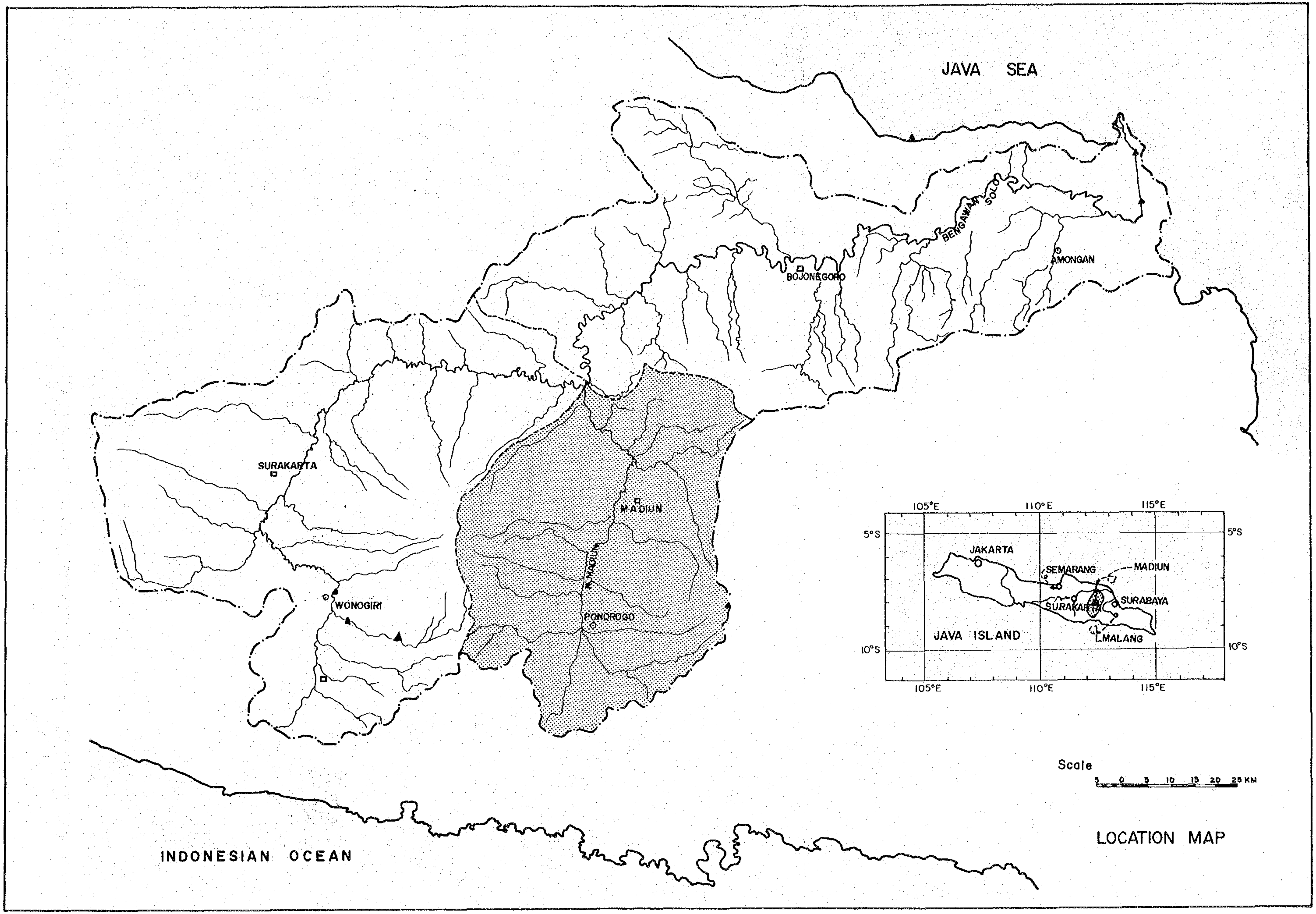
Government of the Republic of Indonesia

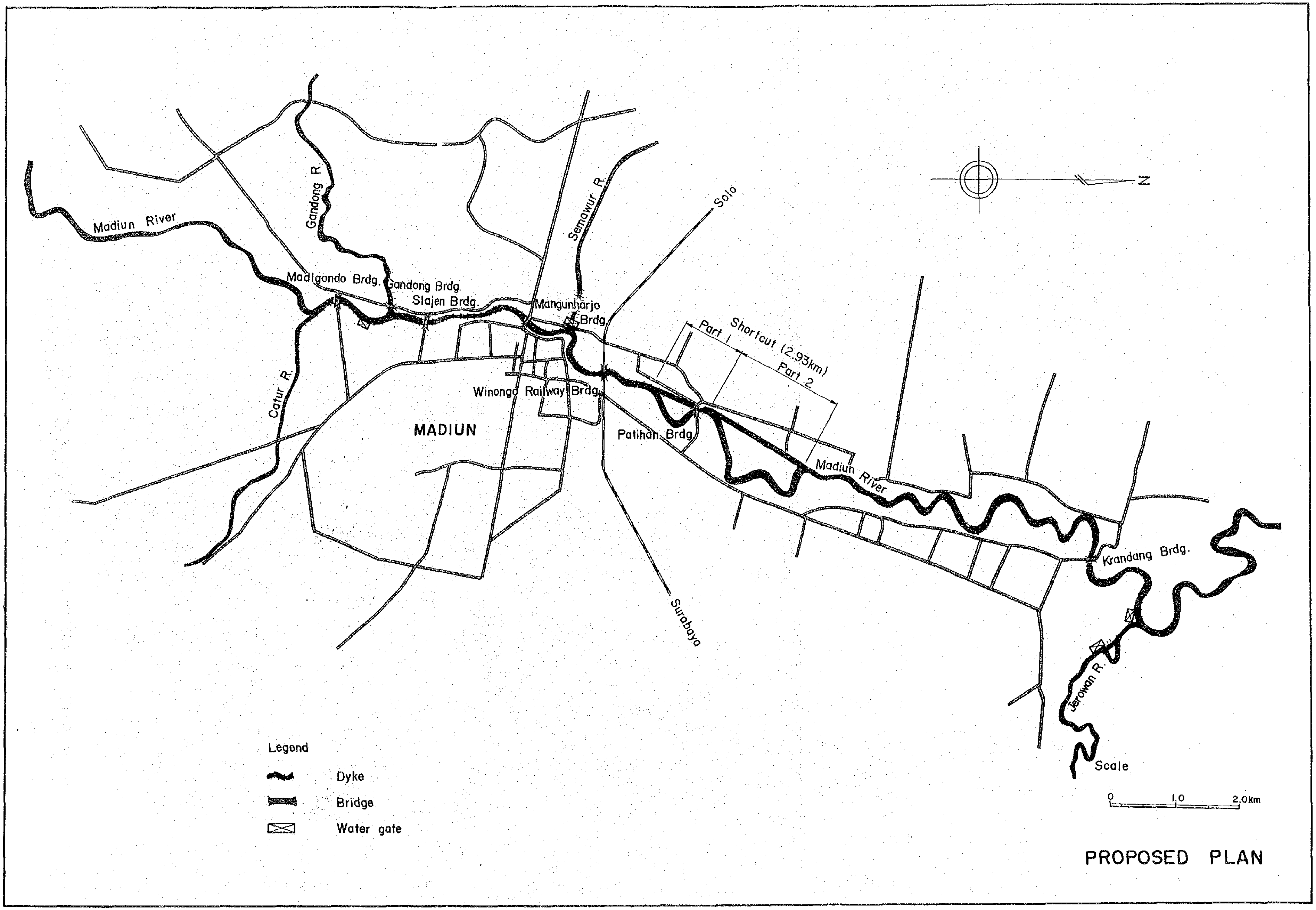
REPORT ON THE FEASIBILITY STUDY
FOR
THE MADIUN RIVER URGENT IMPROVEMENT PROJECT
(Supporting Report)

December 1980

Japan International Cooperation Agency
Tokyo



LOCATION MAP



PROPOSED PLAN

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1. INTRODUCTION

The feasibility study on the Madiun river urgent improvement project was carried out by the study team dispatched by the Japan International Cooperation Agency with the effective cooperation extended by the Government of Indonesia. The study team carried out the field study from March 20, 1980 to July 18, 1980 in the project site. During the field study, the team carried out the topographic and geologic investigations in addition to the collection of the existing data for the various study aspects. The team reviewed and analyzed the collected data and formulated the improvement plan. The studied results and the intermediate plan were summarized and described in the Interim Report.

Following the field study, home works for the project were carried out in Japan during the period from July to December 1980 reviewing the studied results of the field work.

The feasibility study on the Madiun river urgent improvement project was finalized compiling the studied results of the field and the home works. The feasibility report were prepared in consequence of the study.

The feasibility report are comprised by two volumes namely the main report and this supporting report. The outline of the improvement plan was mentioned in the main report with the principal results of the studies and the reached conclusions. The supporting report describes mainly the surveyed results and the details of the studies.

In the second chapter, the accuracy of the existing topographic data were discussed with the results of the check survey conducted by the study team. Some recommendations were also commented to use the existing topographic data for the future detailed design. The results of the additional survey along the tributaries were summarized in this chapter and made available.

The geologic and soil survey were mentioned in the third chapter. The results of the additional survey carried out at various sites were analyzed and discussed in this chapter comparing the existing geologic and soil data. The aggregate for concrete were studied and mentioned in the end of this chapter.

In the fourth chapter, hydrologic study was reported. The annual maximum flood discharge data were processed prior to the estimation of the probable flood. The hydrographic analysis was made for the flood routing study. The regulating effect of the Jerowan inundation area was examined by hydraulic method. The inundation in the downstream area was assessed by non-uniform flow method. All the studies were discussed in this chapter.

The river improvement in the middle reach inevitably brings the effect to the downstream reach. The conceivable increase of the flood discharge in the downstream reach was assessed as an effect of the urgent improvement plan.

In the urgent improvement plan, the shortcut works were proposed in the middle portion of the project area. The shortcut work make the river bed slope steeper and the traction force is increased. The scoring of the river bed and the depositing of the sediment load were estimated theoretically for the case the improvement project is realized. In spite of the estimation, the observation of the actual river bed variation is necessary to maintain the riparian structures after the project is realized. And the recommendable monitoring method was studied. These technical assessment of the project were described in the fifth chapter.

The principal subject of the sixth chapter is the detailed discussions on the construction plan. In this chapter, the discussed are the details of the construction method and the breakdown of the calculations for the required numbers of the construction equipments.

It is hoped that this supporting report is useful for the understanding of the matters described in the main report.

2. TOPOGRAPHIC SURVEY

2.1 Available Topographic Data

The topographic map in a scale of 1 to 50,000 was prepared when the country had been under the rule of Dutch. The map covers almost all over the country including the Madiun river basin.

The information shown on the map were compared with ones obtained from the site reconnaissance and the interpretation of air photo. The map was proved to maintain the sufficient consistency still now though some topographic variability was found especially in the arable land. There are some variations caused by the urbanization with the lapse of time in and around the Madiun city. However, the map is considered to keep validness because the variations in the main street, river and other are almost negligible.

The map in a scale of 1 to 10,000 was prepared by the PBS for the inundation area from just upstream from the confluence with the Catur river to Ngawi. The map was prepared based on the longitudinal and cross sectional survey in the area.

Some cross section sites were sampled from the surveyed sections and the study team carried out the check surveys at the sites. Thus the original survey were confirmed to have the allowable accuracy.

Among the bench marks drawn in the map, BM13, 15, 17, 21, 22, 24, 27, 33, 36, 38, 43 and 45 were surveyed again by the study team. The elevation obtained from the bench mark (BM75) which was used to decide the datum elevation for the Madiun river cross sectional survey carried out by the PBS in 1979 and 1980 was adopted as the datum elevation in this check survey. The checked results brought the maximum difference of 1.085 m in BM27, the minimum difference of 0.910 m in BM13 and the mean difference of 1.009 m.

The checked results indicate that the datum level reference plane of the bench mark used in this survey is one meter lower than one of the bench mark used in the river cross sectional survey, in a scale of 1 to 1,000. This also indicate that the bench mark used in this survey is the same with the bench mark established by the PROSIDA.

The accuracies of the survey in the upper reach are satisfactory but are slightly poor in the lower reach. The established bench mark and the prepared map are of use for the feasibility study but rectification is necessary to use the bench mark and the map in the future detailed study.

There are two kinds of maps in a scale of 1 to 5,000. One is the photo map prepared by the PROSIDA. The other one is the topographic map drawn from the air photo survey which was prepared also by the PROSIDA.

The photo map mentioned above was shooted in a scale of 1 to 15,000 and cover 330,000 ha. Among the covering area, the maps of the area along the Madiun river from the Madiun city to the Jerowan river were obtained by the study team. The said area is so flat as the accurate rectification is capable and the map is expected to be the precise photo map.

The topographic map in a scale of 1 to 5,000 was prepared from the surveyed air photo mentioned above for the irrigation planning.

The agricultural land were interpreted and drawn in detail. The topographic contour line was drawn with an interval of 0.5 meters in this land.

In the urban area and village, the roads were drawn with certain accuracy but structures such as houses were drawn rather simply.

The availabilities of the map were checked by the study team by measuring the distances at the sites and the satisfactory results were obtained.

The map, however, were drawn somewhat twisted around the confluence with the Jerowan river and special attention should be paid to use the map in this area.

A map in a scale of 1 to 1,000 is made available along the Madiun river by the survey performed by the PBS.

The PBS has carried out the extensive longitudinal and cross sectional survey along the Madiun river in 1979 and 1980. The survey covers the riparian area between the confluence with the Catur river and the Solo river. The primal surveyed results are the bench marks established along the river, the longitudinal profile of the river and the cross sectional profile with interval of 50 to 100 meters.

The study team investigated the established bench marks for 20 km from KM39 to KM59. The satisfactory accuracies were confirmed by the check survey for the bench marks in the left bank of the river. The accuracies of the bench marks in the right bank of the river were judged to be sufficient except for KM52, KM47 and KM45.

The study team measured the differences between the four corresponding bench marks in the left and right banks. The obtained differences are as follows:

No. of BM		Difference (m)
Right	Left	
KM40KA	KM40KI	+ 0.008
KM46KA	KM46KI	+ 0.025
KM54KA	KM54KI	+ 0.011
KM59KA	KM59KI	- 0.013

The measured results indicate that the altitude of the bench marks in the right bank are generally higher than ones in the left bank by 10 to 20 mm. It is supposed that the differences are attributable to the compensation of eross to obtain the most probable values.

The bench marks are considered to be of use for the future detailed study except for the three mentioned above.

The locations of the investigated bench marks and others are shown in Fig. 2.1.

The prepared cross sections were resurveyed by the study team for the sites of PR20, PR61, HI129, HI224. The distances were measured by the electro optical distance meter and the elevations were surveyed from the relevant bench marks.

No significant difference could be found between the results of both cross sections.

The checked cross section sites are shown in Fig. 2.2.

2.2 Bench Mark

The altitudes in the whole Solo river basin are based on the standard datum of levelling obtained from the mean sea level of the Surabaya harbour.

The study team dispatched by the OTCA, Japan established the network of the bench marks in 1975 in their study on the BENGAWAN SOLO RIVER BASIN DEVELOPMENT PROJECT. The network covers the Madiun river basin. The results of works were summarized in the report; APPENDIX I LIST OF BENCH MARK. The bench marks have been maintained by the PBS.

The PROSIDA established the new bench network in the Madiun river basin including the bench marks established by the OTCA. The standard datum of levelling is originated in the same source with ones established by the OTCA.

The network is shown in Fig. 2.3 and the features of the bench marks are summarized in Table 2.1.

According to the information the elevation given to BM75 by the PBS is lower by 1 meter compare with ones of BM74 and BM76.

The new bench marks were established by the PBS based on the elevation of BM75 in order to carry out the survey for the map in a scale of 1 to 1,000 in 1979 and 1980.

The elevation of new bench marks were compared with the corresponding bench marks of the PROSIDA's network. And the following results were obtained:

No. of bench mark		Difference (m)
PROSIDA	PBS	
BM76	KM59KA	- 0.975
BM75	KM50KA	- 0.993
BM74	KM41KA	- 0.993
BM38	KM 1KA	- 1.012
BM72	KM24KI	- 0.977
BM73	KM31KI	- 0.953

In this table, the PROSIDA adjusted the elevation of the bench mark, BM75 by 1 meter.

The mean of the difference is -0.984. Accordingly, it is concluded that the elevation of BM75 in the report prepared by OTCA should be adjusted by 1 meter or the elevation proposed by the PROSIDA should be adopted in the future use.

2.3 Additional Topographic Survey

The longitudinal and cross sectional survey were carried out in the proposed short-cut sites. The center lines of the short cut were plotted on the photo map in a scale of 1 to 5,000. The center lines in the map were set at the sites and the longitudinal and cross sectional survey were conducted. The cross sectional survey were carried out for around 200 meters in an intervals of 200 to 400 meters. The drawings of the longitudinal and cross sectional profiles were prepared as the survey results. The bench mark used in the survey for the map in a scale of 1 to 1,000 were used in this additional survey to maintain the consistency between both results.

The longitudinal and cross sectional survey were conducted in and along the tributaries such as the Catur, Gandong and Jerowan rivers. The center lines of the tributaries were plotted on the map in a scale of 1 to 5,000. The plotted center lines were set at the sites and the survey were carried out. The same specifications as the survey in the short cut sites were applied in the survey in the tributaries.

The drawings of the longitudinal and cross sectional profiles were prepared as the survey results.

The locations of the surveyed cross section in the short cut and the tributaries are shown in Fig. 2.4 and 2.5.

The numbers of houses to be relocated due to the short cut and the heightening of the dykes were counted in the said photo map applying the stereo scopy. The counting were made for each shortcut site and the dyke. The obtained numbers were checked and reviewed at the sites. In this manner, an accurate number of the houses to be compensated were obtained.

Table 2.1 Available Bench mark

(Quoted from the work of PROSIDA)

NAME	ELEVATION	NAME	ELEVATION	NAME	ELEVATION
BM 38	45.177 m	IM.0	78.288 m	IM.O(BASE)	78.288 m
BM 72	47.194	IM.2	79.564	INITIAL POINT	77.888
BM 73	50.073	IM.6	63.375	NO.2	76.271
BM 74	53.858	IM.7	53.886	NO.4	73.227
BM 75	59.127	IM.8	94.852	NO.6	72.544
BM 76	69.204	IM.9	84.951	NO.8	72.241
BM 77	86.239	IM.18	52.848	NO.10	70.595
BM 78	113.738	IM.19	51.019	NO.12	70.375
BM 80	113.128	IM.20	46.263	NO.14	70.469
Db.5	66.835	IM.70	60.799	NO.16	70.058
Db.4	73.775	IM.162	70.248	NO.18	70.339
Db.9	53.500			NO.20	69.284
Db.12	53.500			NO.22	
IDA.1	84.187			NO.24	68.321
IDA.2	102.231			NO.26	68.099
IDA.5	70.813			NO.28	66.467
				IDA.5	70.811
				IM.18	52.863

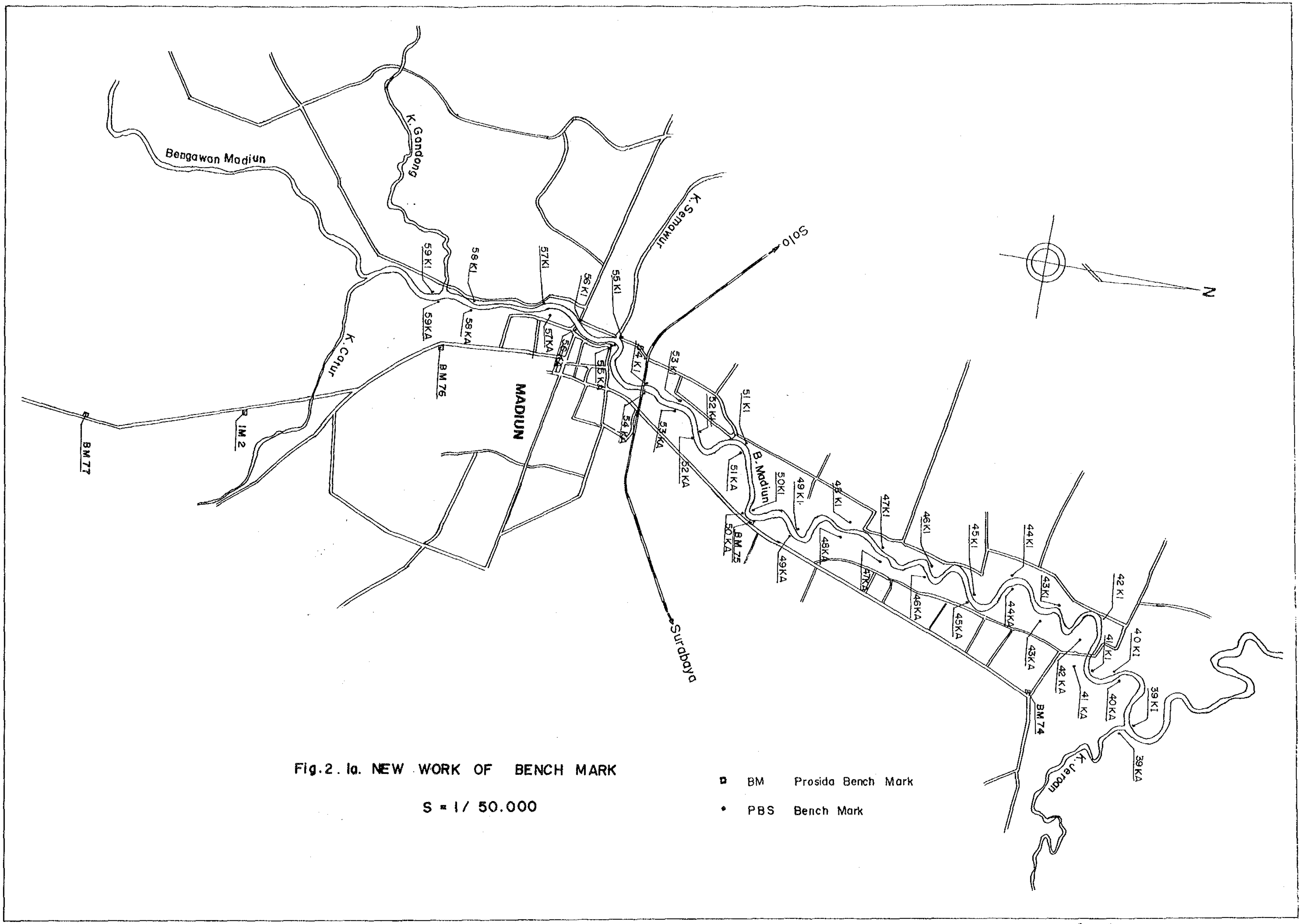


Fig. 2. Ia. NEW WORK OF BENCH MARK

S = 1/ 50.000

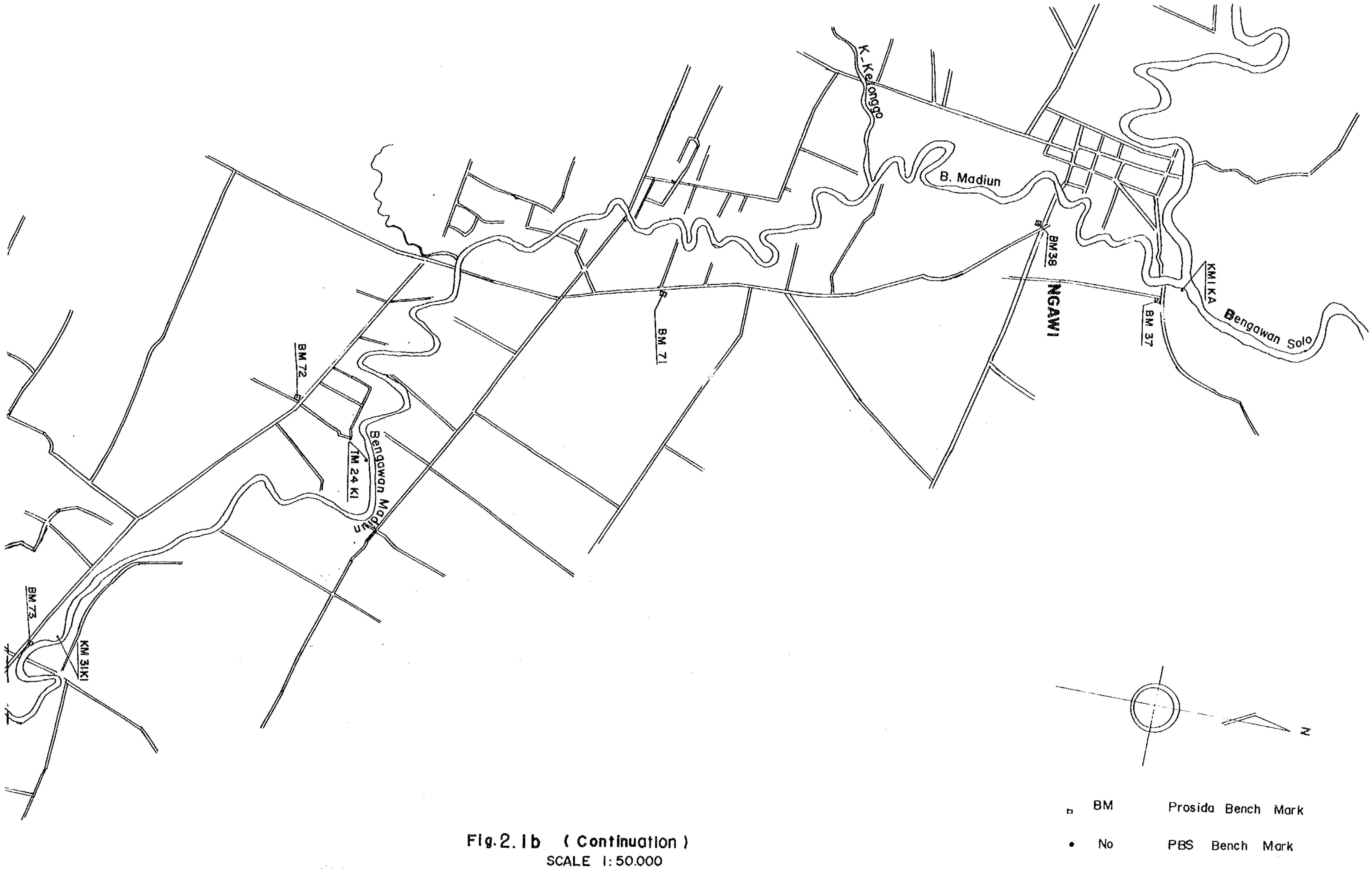


Fig. 2.1b (Continuation)
SCALE 1:50.000

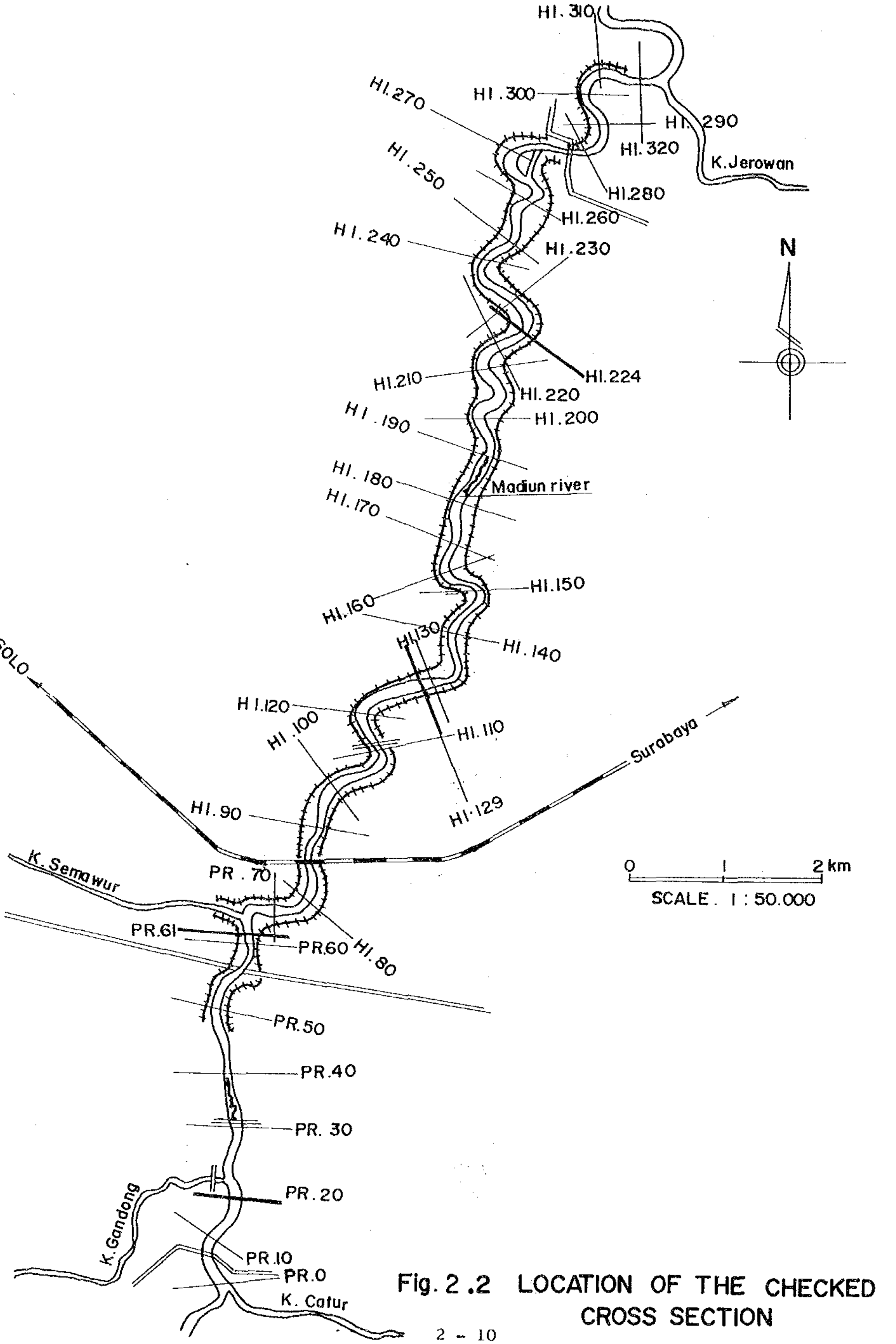


Fig. 2.2 LOCATION OF THE CHECKED CROSS SECTION

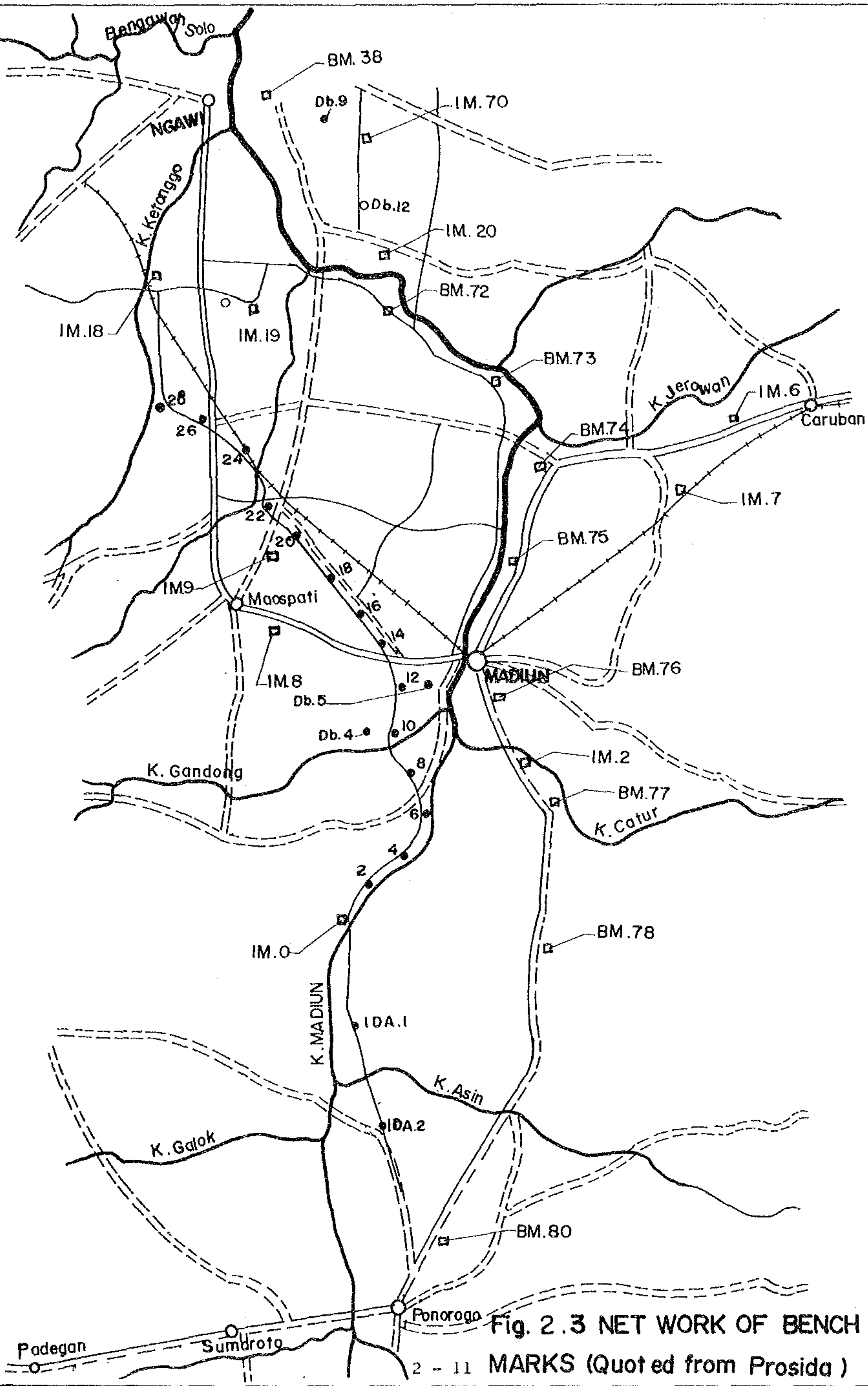


Fig. 2 .3 NET WORK OF BENCH MARKS (Quot ed from Prosida)

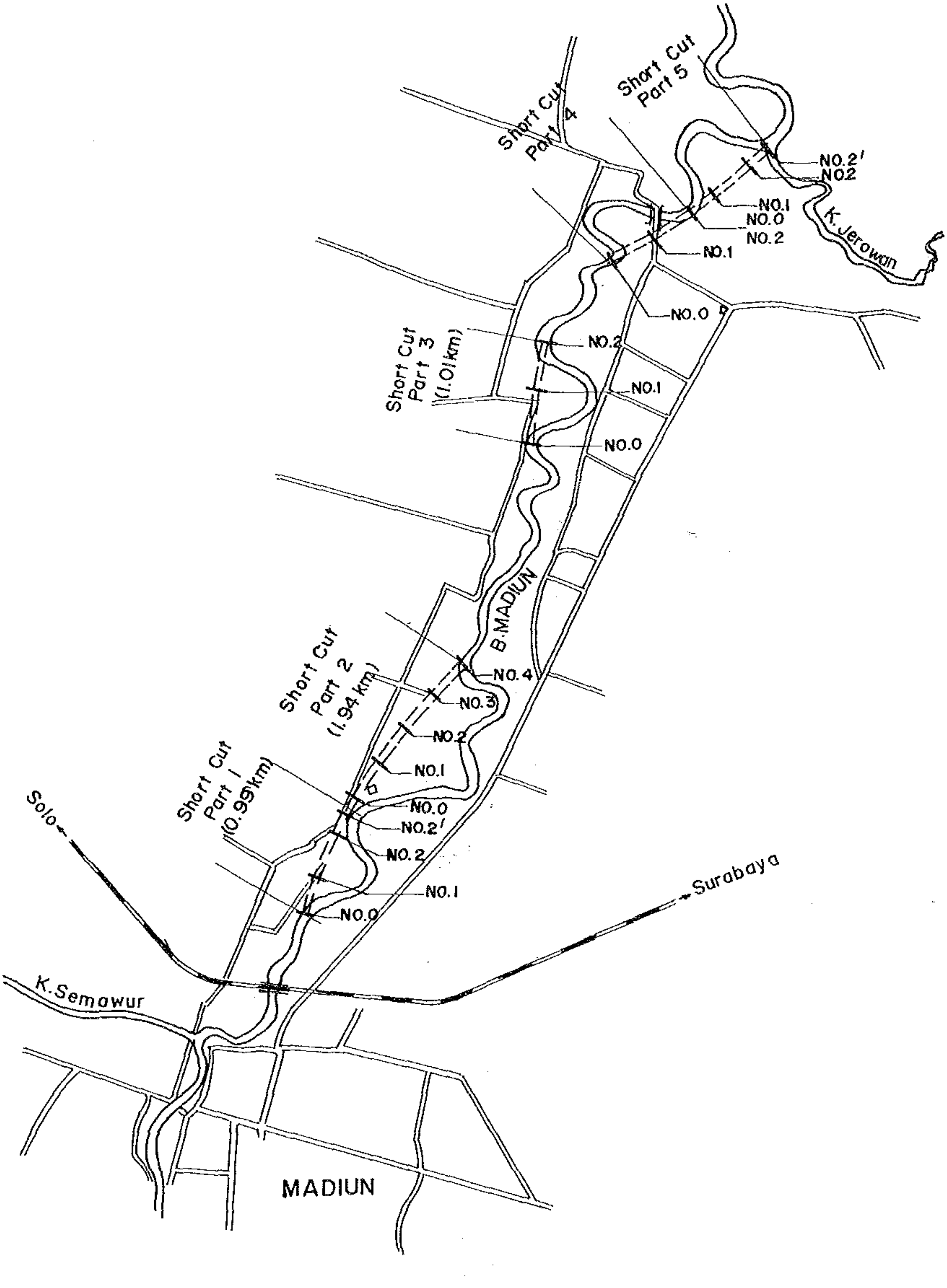


Fig.2.4 LOCATION OF THE SURVEYED CROSS SECTION
(Short Cut)

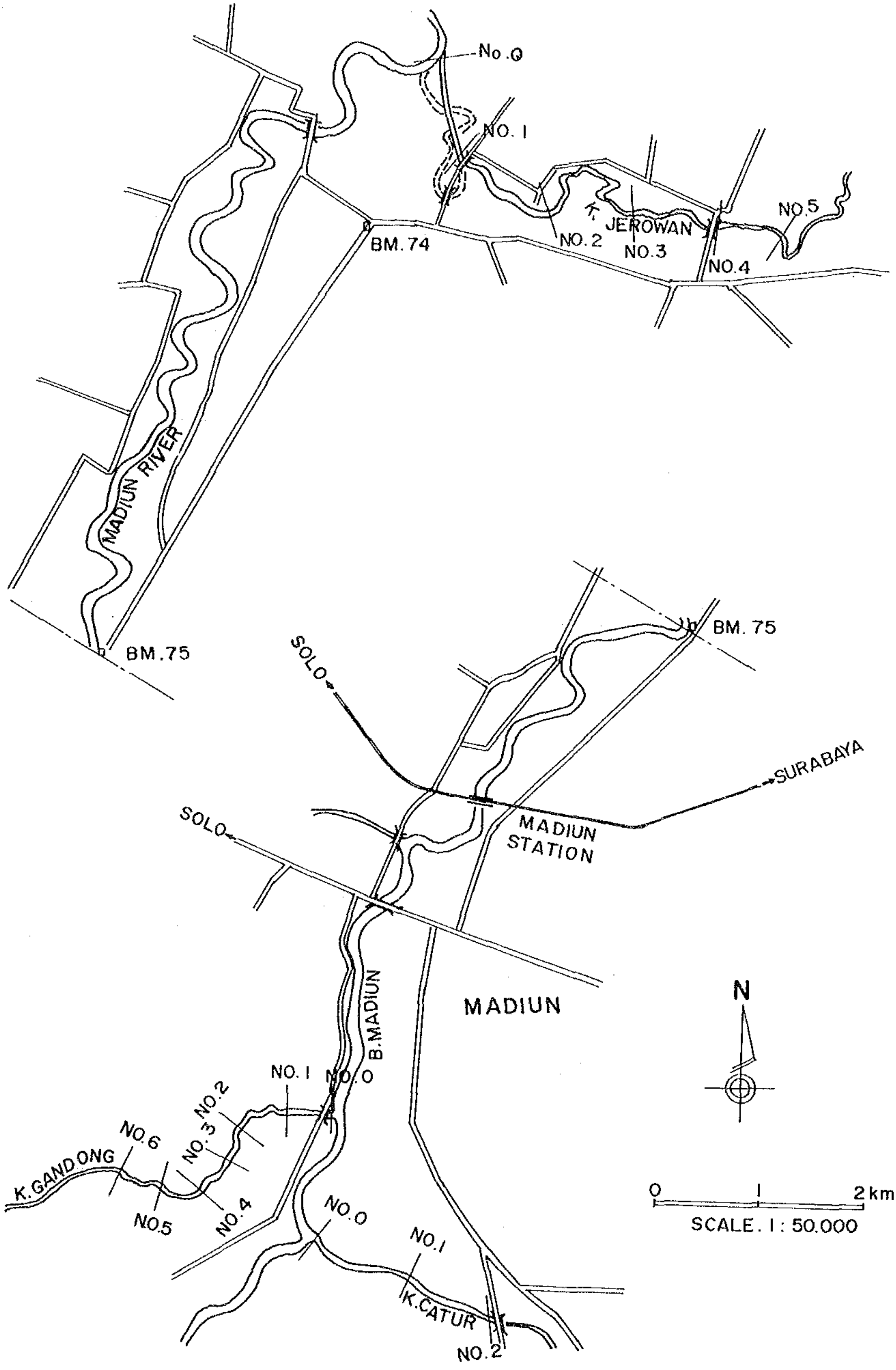


Fig. 2.5 LOCATION OF THE SURVEYED CROSS SECTION
(Tributari)

3. SOIL AND GEOLOGICAL STUDY

3.1 General

The Madiun river basin is surrounded by mountains and hills; The surroundings are mount Lawu, altitude of which is 3,265 meters above sea level, in the west, mount Wilis, 2,169 meters in the east, several mountains, peaked 500 to 600 meters, in the south and hills in the north.

The mountain slope of mount Lawu and mount Wilis are composed of quarternary volcanic products such as mud flow, pumice flow, tuff breccia, lava and so on. The southern mountain area at the border of the Madiun basin is composed of tertiary marine deposits, such as limestone, sandstone, mudstone, etc. and also the northern hilly part is composed of quarternary and tertiary marine deposits which are built up by anticline and syncline structures in geologic term.

The Madiun plain itself is covered by the deposits recently carried by the Madiun river. The river deposit covers alluvium, pleistocene deposits and volcanic products produced by mount Lawu and Wilis at the feet of mountains.

The project area belongs to the Madiun plain and its geology and soil are composed of alluvium deposits in the surface and pleistocene ones, such as sand and hard clay thereunder.

The general geology of the Madiun river basin is illustrated in Fig. 3.1, 3.2, and 3.3.

The details of the geology and soil are discussed in section 3.3.

3.2 Available Geological and Soil Data

The geological investigations on foundation of check dams were carried out by the P.B.S. and the data are made available.

These geological invetigations were carried out in the outside of project area but are useful to get general idea of geology in the Madiun basin. Soil investigation were performed to obtain the information on the fill material for dykes and the sampled sites were located close to the project area. And the data are supposed to be useful for the project. The details of the collected data are given Fig. 3.4.

The locations of the soil investigation in the project area and the prepared reports are shown in Fig. 3.4. The previous soil investigations described in the said reports are conducted for two proposes. One of them was to study the fill materials to reconstruct the broken dyke. The study was carried out by the P.B.S. The other one was made by Gajahmada University for the study on the floodway.

The former study were described in the report No.1 to 13. The investigation were performed by means of Test pit, Dutch cone penetration test and Auger boring. The samples from the test pits were examined in the laboratory. The usefull information of subsurface soil condition were obtainable thereby. The latter one was investigated along the new proposed floodway and described in the report No.14. According to the report, the same method described above were adopted to the investigation. The data obtained from both were used in this study for the analysis of the soil conditions in the project area and the properties for fill materials. They are summarized and shown in Fig. 3.5, 3.6, 3.7, 3.8, 3.21, 3.22, and 3.23.

According to these previous investigation, it is considered that the soil type is composed of clay, sandy clay, dense sand and sand in the project area. Clay of grain size distribution more than 70% passing of 0.074 mm is predominant. Sand of grain size distribution less than 30% passing of 0.074 mm were found in only seven samples, which is deemed to have been sampled at the terrace or near the river bed along the Madiun River.

The natural moisture contents are almost in the range of 20% and 40%. And they are near and less than the plastic limit for clay indicating that they are relatively hard clay. The unit weights in site are almost in the range of 1.6 g/cm^3 and 1.9 g/cm^3 and the specific gravities are almost in between 2.4 and 2.7. The figure is rather small for soils distributed in the volcanic area.

Clay soils of this area are classified into the categories of CL, ML, CH, and MH from their plasticity chart and grain size.

Compaction test proved that the maximum dry density are in between 1.3 g/cm^3 and 1.7 g/cm^3 and ones of the optimum moisture content are in between 20% and 25%. The optimum moisture content are between more 10% more than natural moisture content and less 10% less than that. Concerning clayey soils, optimum moisture contents are less than natural moisture contents.

Data of cohesion (C) and friction angle (F) obtained by the results of direct shear thest distributed in the wide range as shown in Fig. 3.23.

3.3 Additional Geological and Soil Survey

The short cut of the meandering river channel was proposed as the alternative improvement plans. In this connection, the relocation and/or hightening of bridges were required as well as the construction of new shortcut channels by excavation and construction of new dykes thereof by embankment.

Therefore, the soil and geological conditions of the bridge foundations including one of National Railway Bridge, the conditions along proposed shortcut sites and embankment sites were studied as the additional survey work. The properties of the excavated materials derived from the proposed short cut sites were also studied to examine the possibility of the material to utilize for the embankment. As for the additional soil and geological survey boring, auger boring, test pit with sampling, Dutch cone penetration test and Sweedish sounding were carried out. And also soil samples were carried into the soil laboratory of the P.B.S. for the tests.

The quantities of the investigation were listed in Table 3.1.

The result of the investigations and laboratory tests are discussed for each site as follows:

Proposed Shortcut Site Part - 1

The soils are Brown Clay (C6) and Clay (C) with a little Terrace Sand (S_t) and Loose Sand (S_1). The ground water surface is found at the depth of 3 m to 4 m from the ground surface. In the proposed relocation site of the Patihan bridge brown clay of 2 meters thick are distributed beneath ground surface. A few sandy soils are included therein and they cover clay with 5 m thick.

Dense Sand (S_d), in which number of Standard penetration test (S.P.T) is more than 30 and qc of Dutch cone penetration test is more than 100 kg/cm^2, appears in the depth of 8 m in the left and right side of the proposed short cut sites.

Proposed Shortcut Site Part - 2

The soils of the short cut site were identified to be Brown clay, clay and Loose Sand with a little Terrace Sand at the both, ends. The ground water surface is found at about 3 m in depth in the layer of clay.

Proposed Shortcut Site Part - 3

The proposed shortcut site are covered by Brown Clay, Clay and Terrace Sand for around 4 m depth. The ground water surface is found at 1.5 m to 3 m in depth.

Proposed Shortcut Site Part - 4

The proposed shortcut site are covered by Brown Clay, Clay and Terrace Sand. The ground water surface is found at 2 m to 3 m in depth. Soil of the proposed relocation site of the Krandang bridge is identified to be Brown Clay of 2 m to 3 m thick covering clay.

Dense Sand appears at 8 m in the left side and 14.5 m in the right side in depth. The number of S.P.T and qc of Dutch cone penetration test thereof are 35 and 100 kg/cm^2 respectively.

Proposed Shortcut Site Part - 5

The soils of the proposed shortcut site are covered mainly by Brown clay and clay with a little Terrace Sand and Loose Sand. The ground water is found below the ground surface by 1 m - 2. m.

Soil condition along the existing railway bridge is investigated. The developed soil cross section shows that the depth of the river bed sand (S_r) is approximately 5 m and it is in loose condition because the number of S.P.T. is in the range of 3.

Beneath the river bed sand, gravel is distributed for around 1.6 meters. And dense sand and clay are distributed below gravel.

The locations of the investigated sites are shown in Fig. 3.9.

The soil cross section along the proposed shortcut sites and the bridge sites are developed and shown in figures from Fig. 3.10 to Fig. 3.18, and their composition of soils are also shown in Legend of Cross Section (A).

As shown in the sketch of typical cross section in Legend A "Terrace Sand" (S_t) with the thickness of 0 m to 3 m or Brown Clay (C_b) with the thickness of 1 m to 3 m cover clay (C), Loose Sand (S_l) and Dense Sand (S_d). The water surface of the subsurface acquifer is found at the depth of 1.5 m to 4 m from the ground surface.

3.4 Soil Test And aggregates

Soil tests were carried out for 60 samples. The tested were the natural moisture content, specific gravity, atterberg limit and grain size analysis. And uniaxial and triaxial compression test were also performed.

Surveys for the concrete aggregate were carried out in this study.

<u>Soil Test</u>

The results of soil tests were summarized and shown in the attached fitures, Fig. 3.19 to Fig. 3.23.

Most of the samples are taken within the depth of 5 m from present ground surface at the proposed short cut sites and are materialized for soil tests.

They belong to Brown Clay (C_b) and Clay (C), except few terrace Sand (S), Loose Sand (S_1) and Dense Sand (S_d).

The results of grain size analysis shown in Fig. 3.20 proved that Clay with more than 80% passing of 0.074 mm grain size were predominant.

The natural moisture contents were almost in between 30% and 47% and the result is a little higher than the data from the previous studies.

Specific gravity are in between 2.4 and 2.6, and unit weight are though only 8 samples are tested, in between 1.7 g/cm^3 and 2.0 g/cm^3. These properties almost coinside with the results of the previous studies.

Clay soils are classified in CH, MH, and CC from plasticity chart and grain size. The result is a little different from ones in the previous study of CL, ML, CH, and MH. A reason of this is considered in the difference of clay content less than 0.005 mm grain size between both studies. The difference becomes obvious if the grain size accumulation curves in Fig. 3.8 and Fig. 3.20 are compared.

Liquidity index (I_L) indicates relative hardness of clay soils. If the I_L of clay is small, the clay is considered to be hard in the natural condition. The obtained I_L shown in Fig. 3.19 is almost in between 0.1 and 0.5. The clay is considered to be hard and stable in the natural condition.

The result of compaction test shows that the maximum dry density are in betwee 1.3 g/cm^3 and 1.6 g/cm^3, and ones of optimum moisture content in between 20% and 25%. This result coinsides with the data of previous studies. Compared with natural moisture content, the optimum moisture content scattered less 15% to more 5% than the natural moisture content.

Results of cohesion (C) and friction angle (F) obtained by triaxial test on U.U. condition are plotted in Fig. 3.23 together with the results of the previous study obtained by direct shear test. The plots distribute in the wide range.

On the other hand, the results of the site tests marked that number of S.P.T. is almost in between 13 and 30 though rather high value are obtained, and qc of Dutch cone penetration test is recorded between 10 and 20 kg/cm^2 at the same depth where the materials for laboratory test were sampled.

Cohesion (C) inferred from these values, are supposed to be in between 0.8 kg/cm^2 and 2.0 kg/cm^2, if friction angle is neglected. Therefore, it is difficult to define the unique values for C and F throughout the project area. And the specific value should be applied to each site.

The information and conclusion on the fill material derived from the data obtained by soil test and the previous work were summarized as described below.

Fill materials obtained from the proposed short cut are mainly clay soils with unified classification system of CH, CL, MH and ML, including a little S_1 and considered to be barely allowed to use for embankment.

As proposed height of embankment is about 3 m no problem may occur in the embankment of the proposed slope if they are compacted well. The embankment of 3 meters height will cause no problems in the foundation of embankment according to the obtained figures of S.P.T., qc of dutch cone and the results of soil test.

The natural moisture content of the soils derived from the shortcut sites are rather high. And the material derived from the proposed short cut should have to be dried until it come near to the optimum moisture content before it is used for embankment and compaction.

No concrete figures for C and F to analyse the slope stability could be obtained in this study as stated before.

However, the results of soil tests, the previous data, number of S.P.T., qc of Dutch cone and the present condition of the existing dyke imply that C may be estimated about 0.25 kg/cm^2 and F may be about 10%.

For this matter, further study are recommendable in the detailed design stage.

According to the grain size analysis, the soil contains silt and clay more than 50%. And the permeability is estimated less than 10^{-5} cm/sec.

<u>Aggregate</u>

The river gravel and sand useful for the concrete aggregates were investigated and some data were collected. As for the fine aggregates the riverbed sand have been used and coarse aggregates have been obtained from the crushed river gravel for the actual construction works. It is considered there are no problem to use river bed sand within project area or near for the fine aggregate.

There are no deposit of gravel in the river course of the project area. And there are no crop out of rock near the project site. The most suitable source for the coarse aggregate were judged to be the river gravel in the tributaries which originated in the mountain composed of volcanic products of the both east and west sides of Madiun Basin. The Catur, Gandong and upperstream of the Jerowan river are investigated and their results are shown in Fig. 3.24.

The gravel useful for concrete aggregates are deposited along the Catur and Gandong rivers. The Gravel of the Catur river are larger in size and the estimated quantity is sufficient.

It is preferable to exploit them because it is easy to access to river bed from the existing road.

The available quantity of gravels aong the Catur river is estimated more than 50,000 m^3, the estimated required volume for the project.

Most of the river gravel is composed of andesite with some pumice.

No test for concrete aggregates is carried out but the existing data of the gravel in Lembean along the Gonggong river were collected as shown in Fig. 3.25 and Table 3.2.

The gravel have been used for the reconstruction of the Dam Jati.

The grain size of fine aggregates shown in Fig. 3.25 is the result of natural river bed sand test. The grain size of coarse aggregates in the figure is the test result of crushed gravel used for concrete coarse aggregates at Dam Jati. The tested gravel is considered to be almost same to one in the Catur River.

Table 3.1 <u>Quantity of Geological and Soil Investigation</u>

	location	Part 1 (A)	Part 2 (B)	Part 3 (C)	Part 4 (D)	Part 5 (E)	Railway bridge	Total
Boring	N. of boring	1	0	0	1	1	1	4
	Depth	20 m	-	-	20 m	10 m	11 m	66 m
	Sampling and Soil test	4	-	-	5	3	-	16
Auger Boring	N. of Auger	2	2	2	1	2	0	9
	Depth	7 m 7 m	7 7	7 7	7	7 7	-	63
	Sampling and Soil test	5 4	5 4	4 3	6	2 3	-	36
Test pit	N. of test pit	2	2	1	1	2	0	8
	Depth	3.2 m 3.0	3.4 3.0	3.6	4.0	3.4 2.0	-	25.6 m
	Sampling and Soil test	2 2	2 3	3	2	3 1	-	18
Dutch cone	N. of Dutchcone	2	2	1	3	2	2	12
	Depth	11.6 m 10.2 m	17.6 14.0	7.6	19.4 15.2 10.2	16.8 9.4	12.4 m 18.4 m	162.8 m
Sweedish sounding	N. of sweedish Sounding	0	0	0	2	3	0	5
	Depth	-	-	-	8.6 m 9.3 m	7.3 m 9.6 m 9.7 m	-	44.5 m

Table 3.2 Property of Concrete aggregates in LEMBEAN

	Fine aggregate		Coarse aggregate
	Q 4.7t mm	Q 4.76 mm	
Saturated apprarent specific gravity	2.66 - 2.72	2.50	2.58
Dry apparent specific gravity	2.54 - 2.63	2.40	2.52
Water absorption	4.6 - 3.2	4.2	2.5
True specific gravity	2.89 - 2.88	2.66	2.80
Soundness by sodium sulfate method (percent loss)	6.7%	-	0.6 %

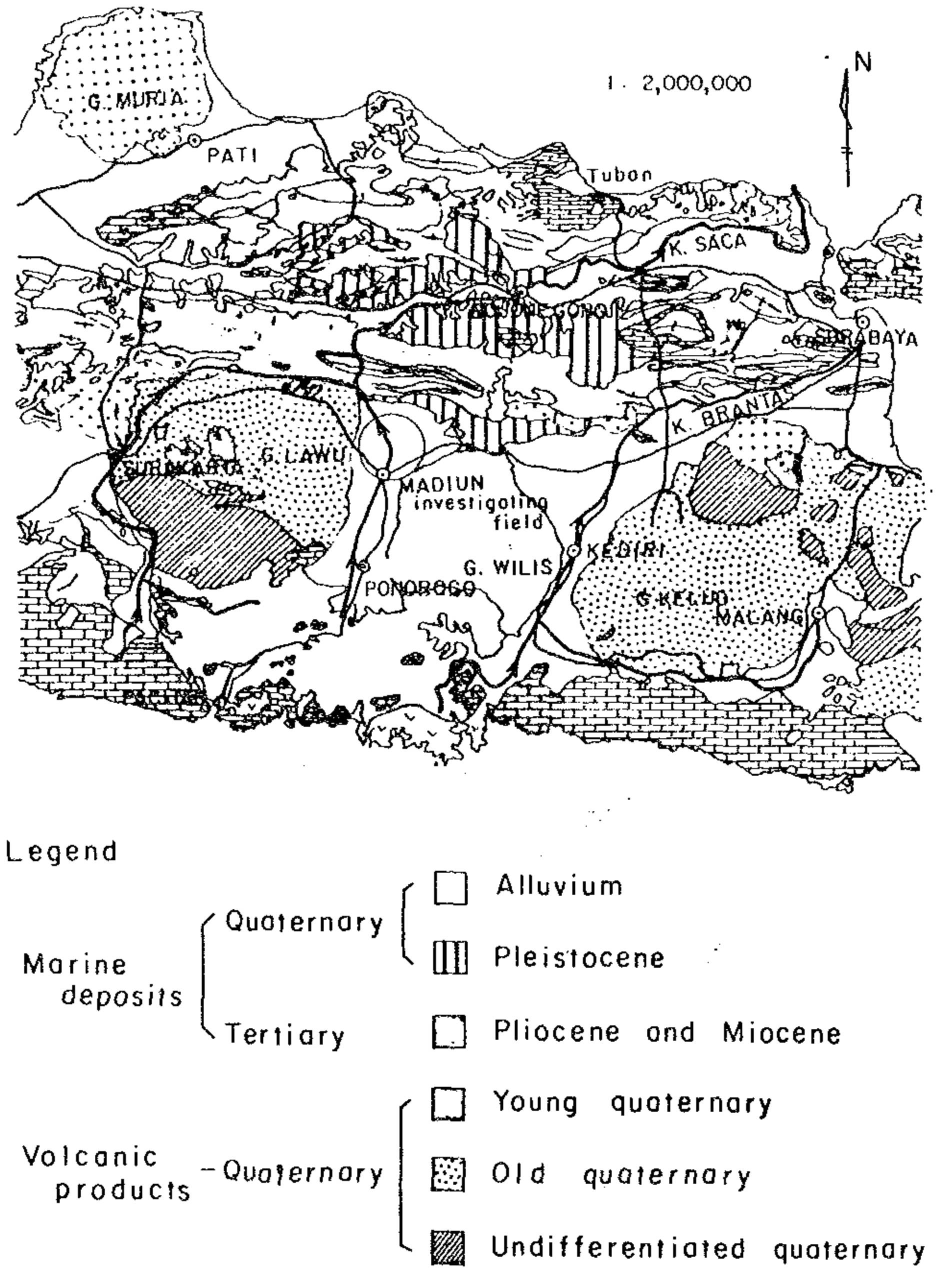


Fig. 3.1 GEOLOGICAL MAP OF EAST JAWA

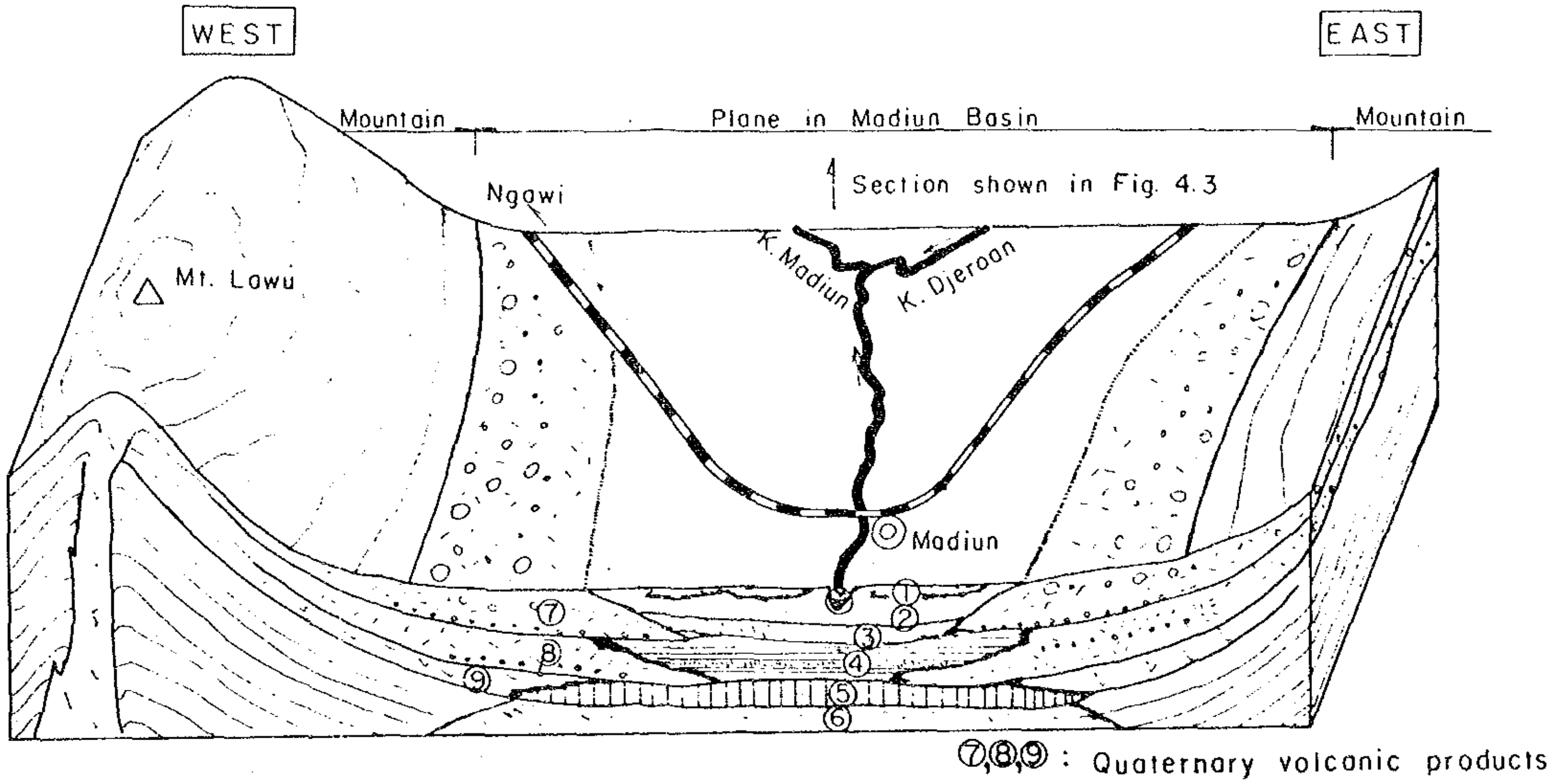


Fig. 3.2 SCHEMATIC BLOCK DIAGRAM OF GEOLOGY IN MADIUN BASIN

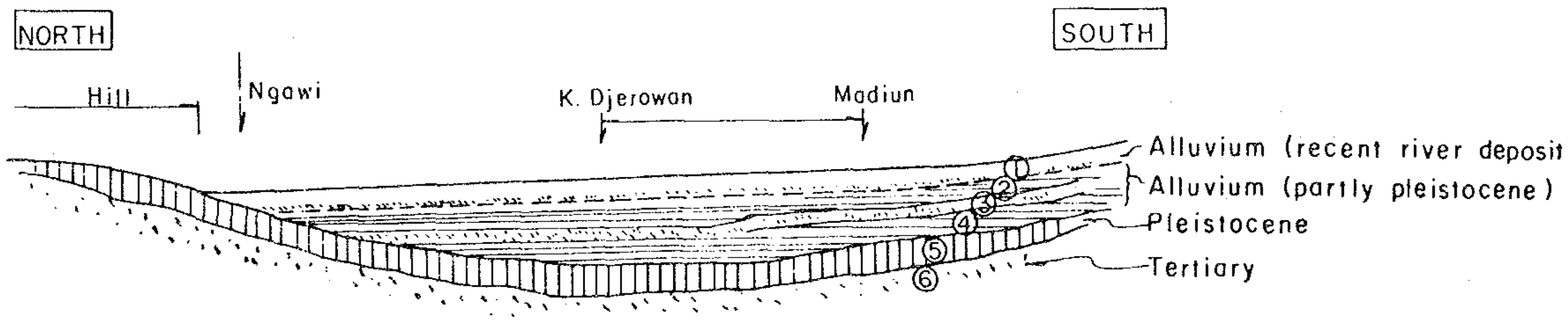


Fig. 3.3 SCHEMATIC SECTION OF MADIUN BASIN

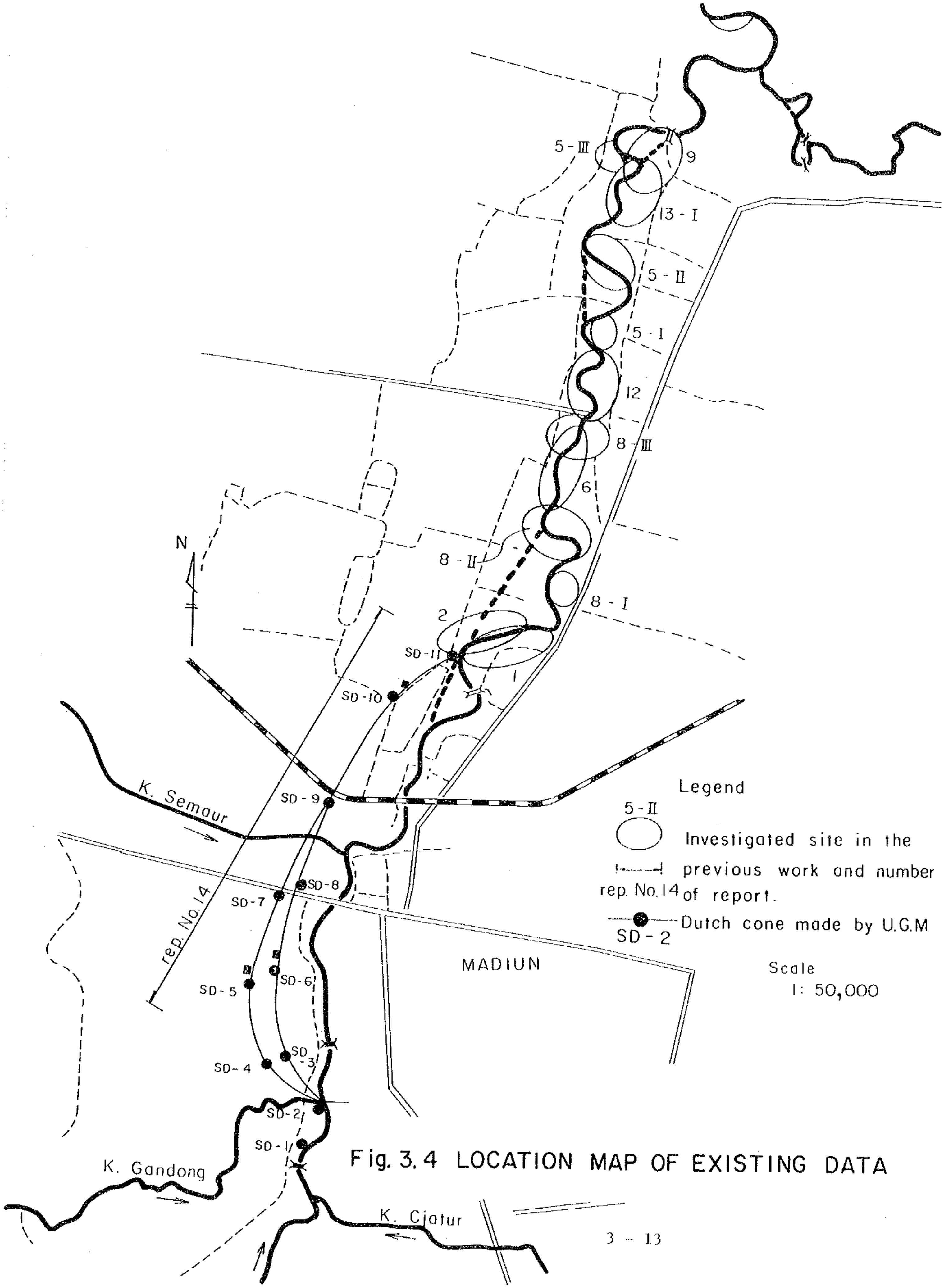


Fig. 3.4 LOCATION MAP OF EXISTING DATA

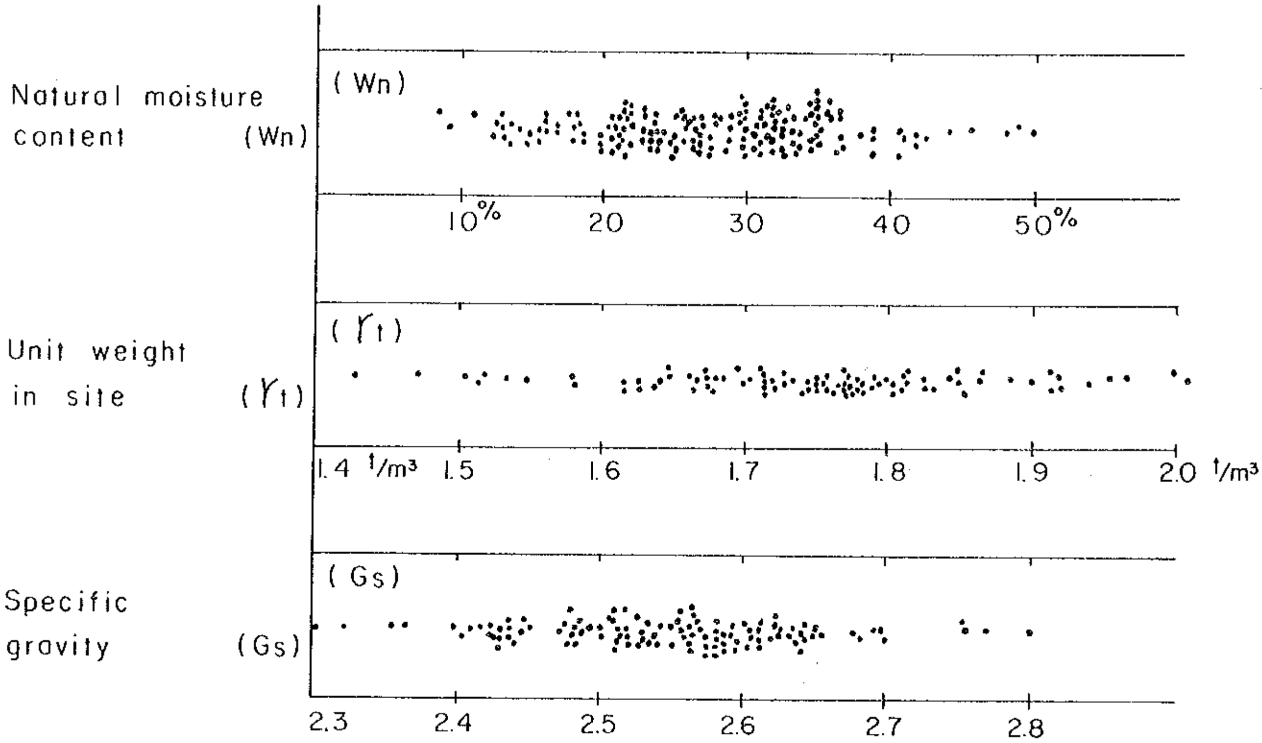


Fig. 3.5 BASIC SOIL PROPERTY (PREVIOUS WORK)

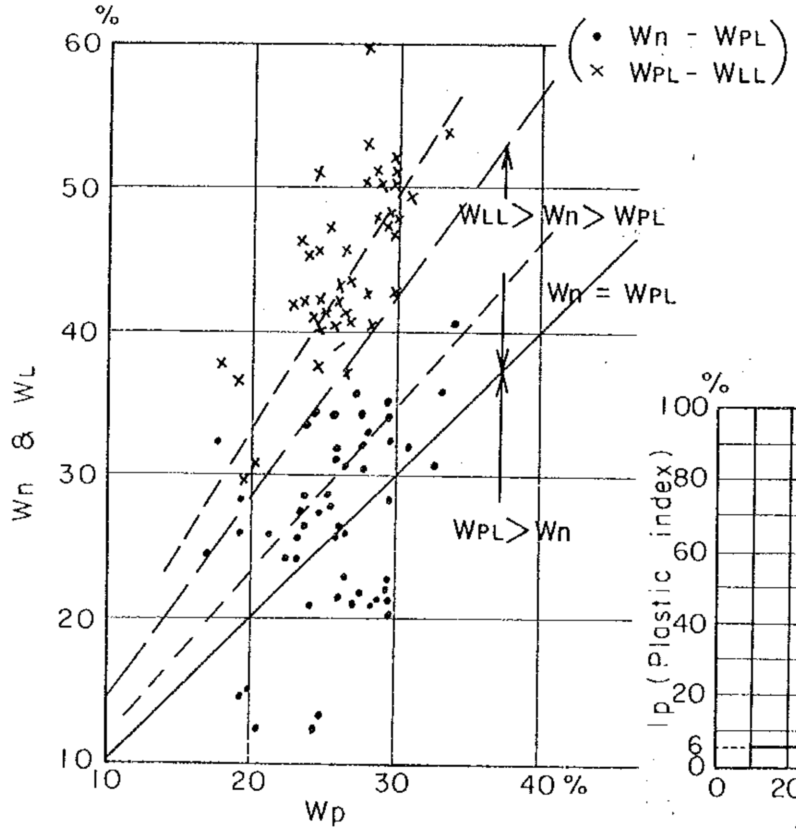


Fig. 3 6 CONDITION OF Wn WITH Wl AND Wp

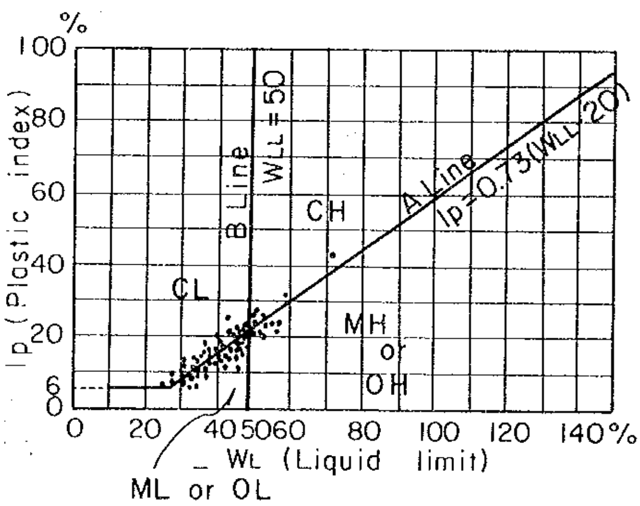


Fig. 3. 7 PLASTICITY CHART

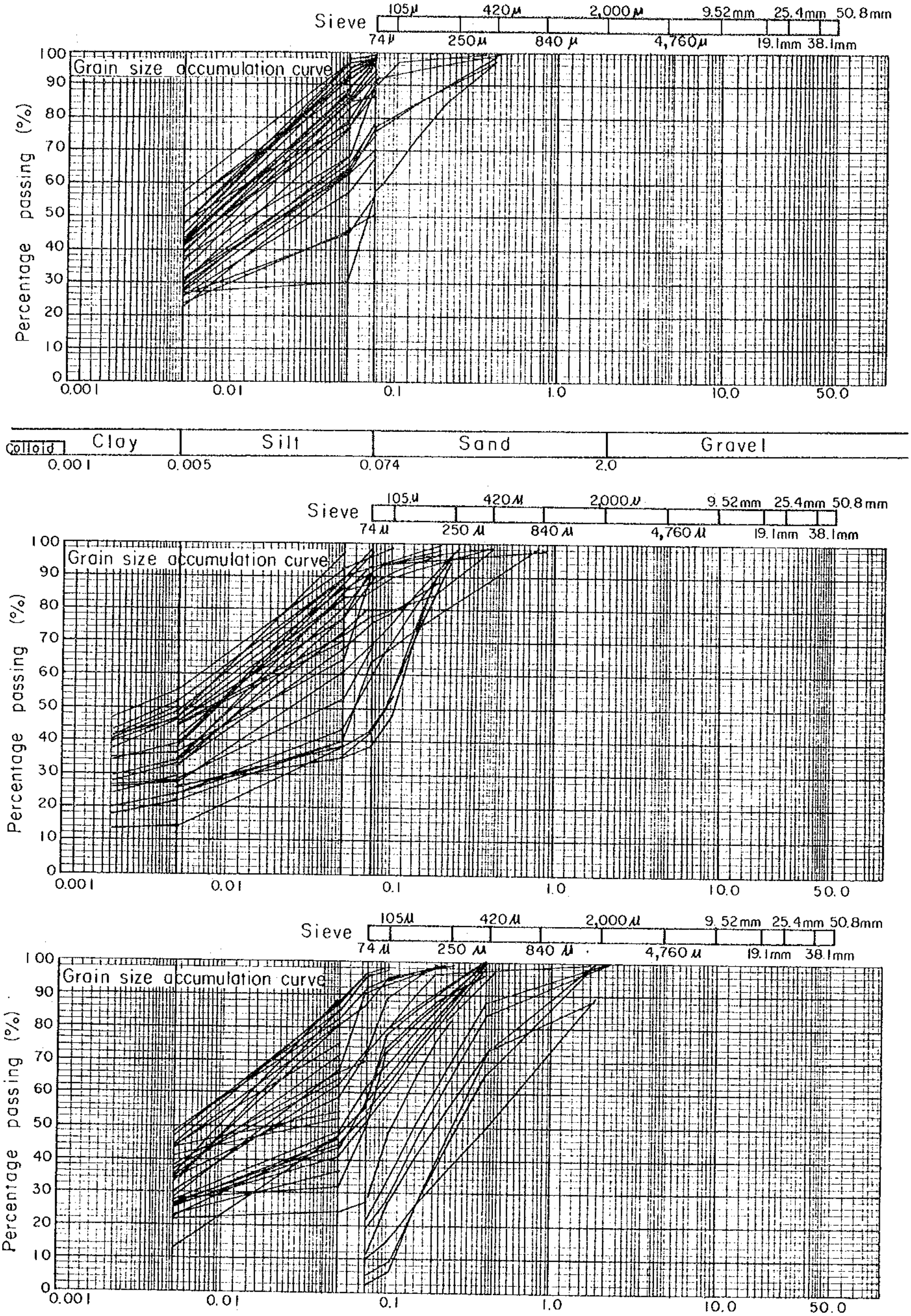


Fig. 3. 8 GRAIN SIZE OF SOIL (PREVIOUS WORK)

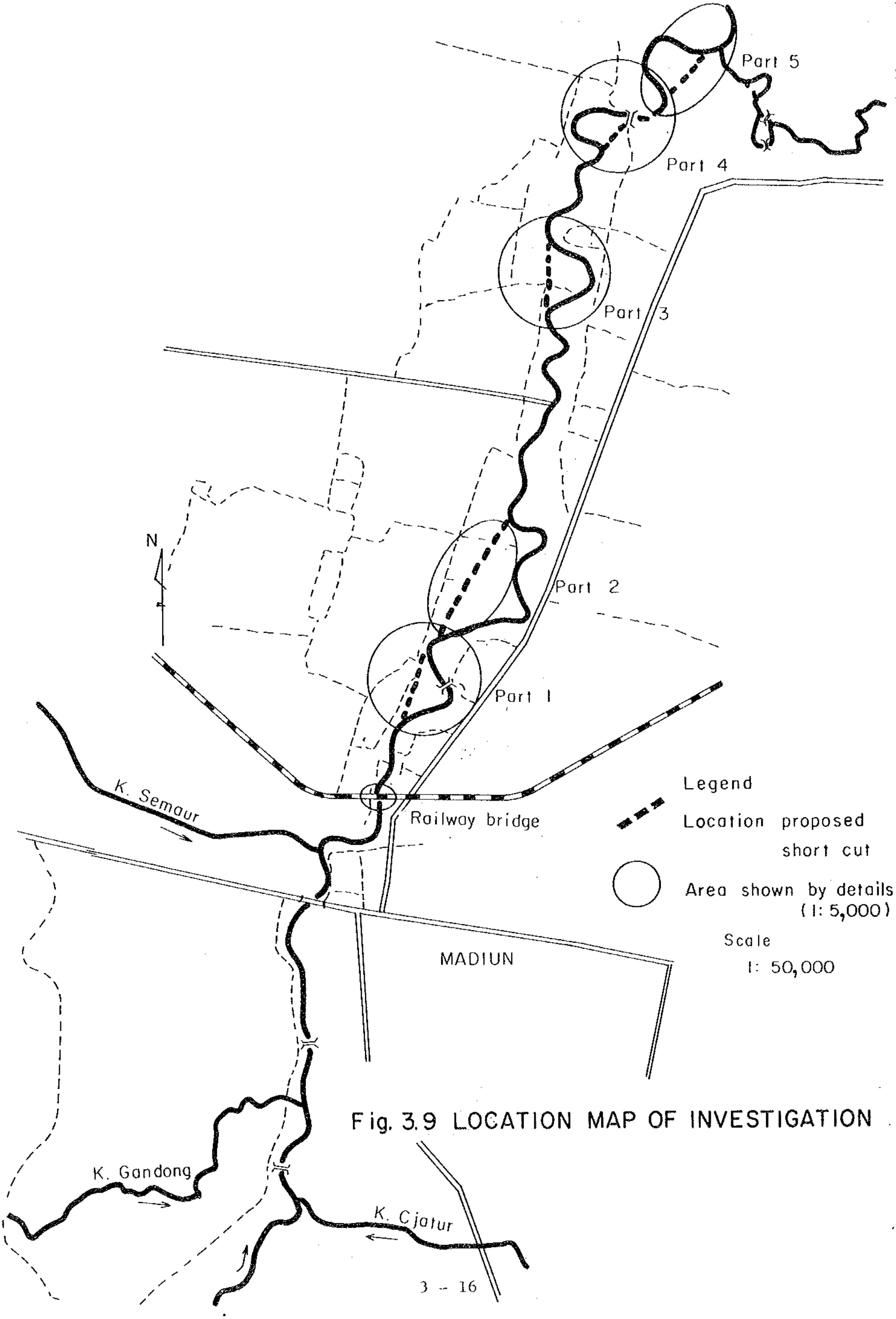


Fig. 3.9 LOCATION MAP OF INVESTIGATION

Legend of Cross Section (A)

	Soil name	Description	N. of S.P.T appro.	qc of Dutch cone (kg/m^3) appro.
S_r	River Bed Sand	grey - dark grey fine - coarse sand very loose. distributed along recent river bed.	30 max 3 min	50 max 4 min
S_t	Terrace Sand	brown fine sand with much clay. contains sand, clayey sand, sandy clay. deposited by flood and then distributed at the surface near the river.	"	"
G	Gravel	rounded gravel and coarse sand. ∅ is between 10 - 100 m/m. distributed below river bed sand.	50 min	-
C_b	Brown Clay	brown clay. sometimes contains sandy soil and also broken brick frogments due to be deposited by flood.	50 max.	25 max 7 min
C	Clay	clay with colour of dark grey, bluish grey white, yellow grey and black, contains silt and silty clay. gravel of coral relics is found. (∅ = 10 - 30 m/m)	32 max 15 min	35 max (50) 10 min
S_l	Loose sand	grey - yellow brown fine sand with clay	35 max 20 min	50 max (70) 25 min
S_d	Dense sand	grey - yellow brown fine - coarse sand	30 min	100 min (50)

LEGEND (B)

Plane

B-1 Boring

Sw-1 Swedish sounding

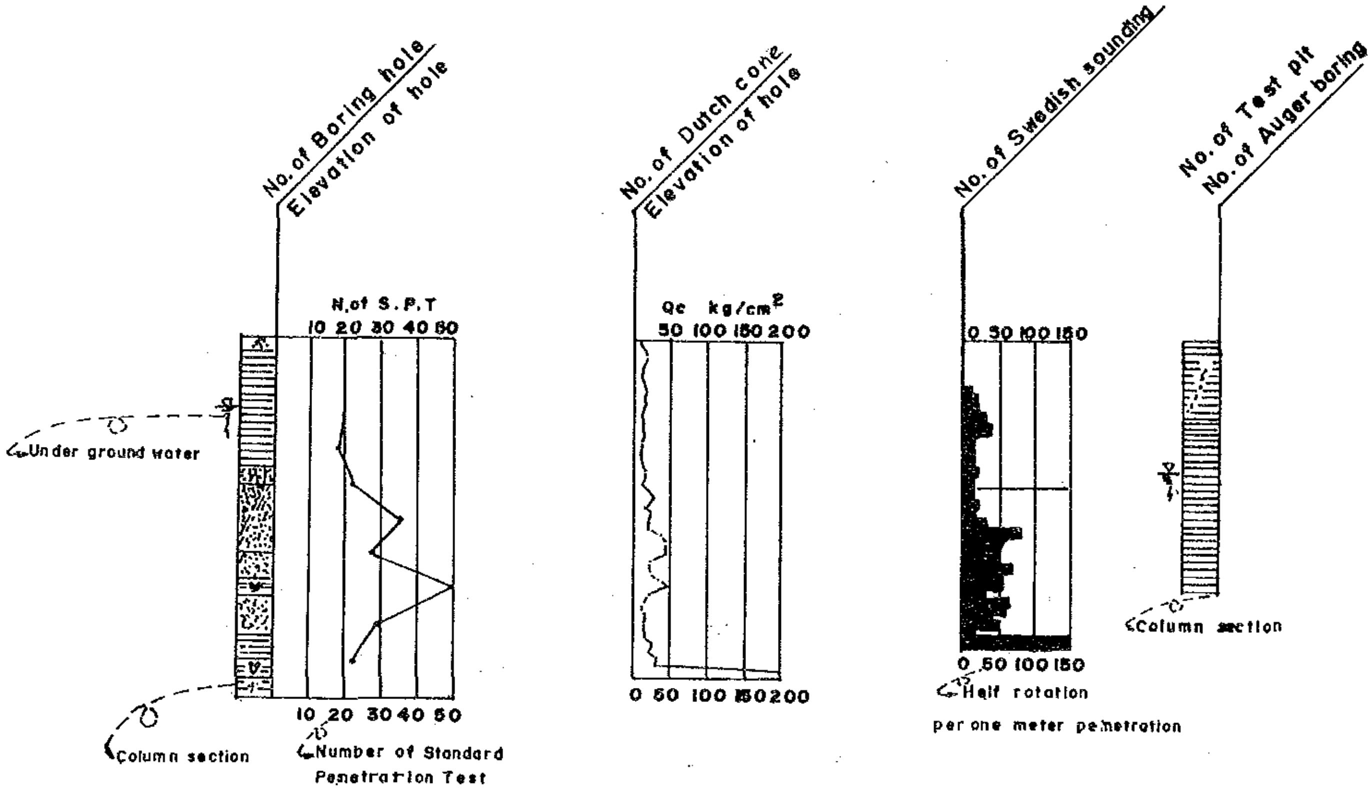
SD-1 Dutch cone

Tp-1 Test pit

Ab-1 Auger boring

Dutch cone (previous work)

Cross section



Clay

Silt

Sand

Clayey

Silty

Sandy

Mixed with clay

Mixed with silt

Mixed with sand

Gravel of coral relic

Boundary of soil or geology

Under ground water surface

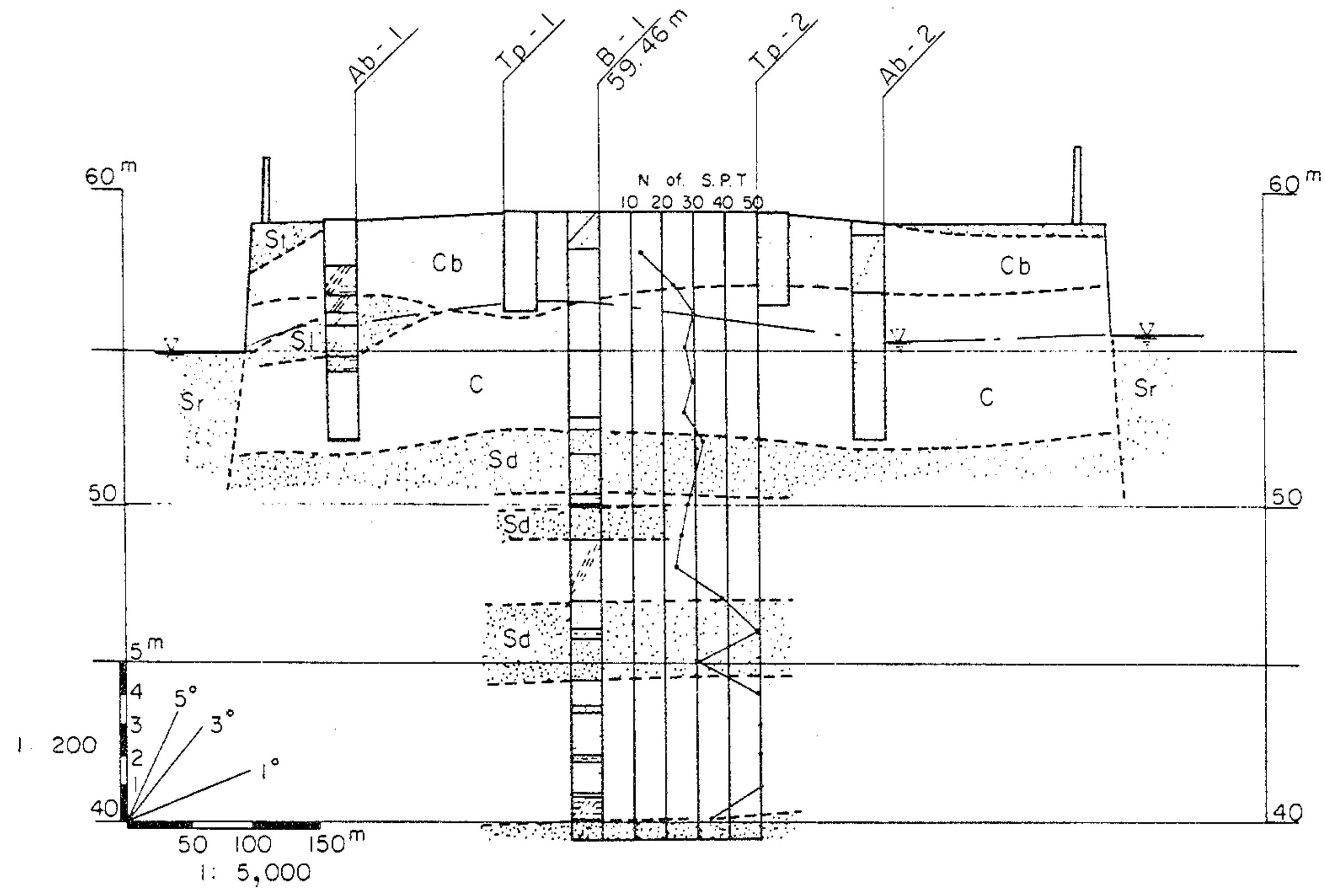


Fig. 3.10 SOIL CROSS SECTION ALONG THE PROPOSED SHORT CUT SITE (PART 1)

Part 2 (B)

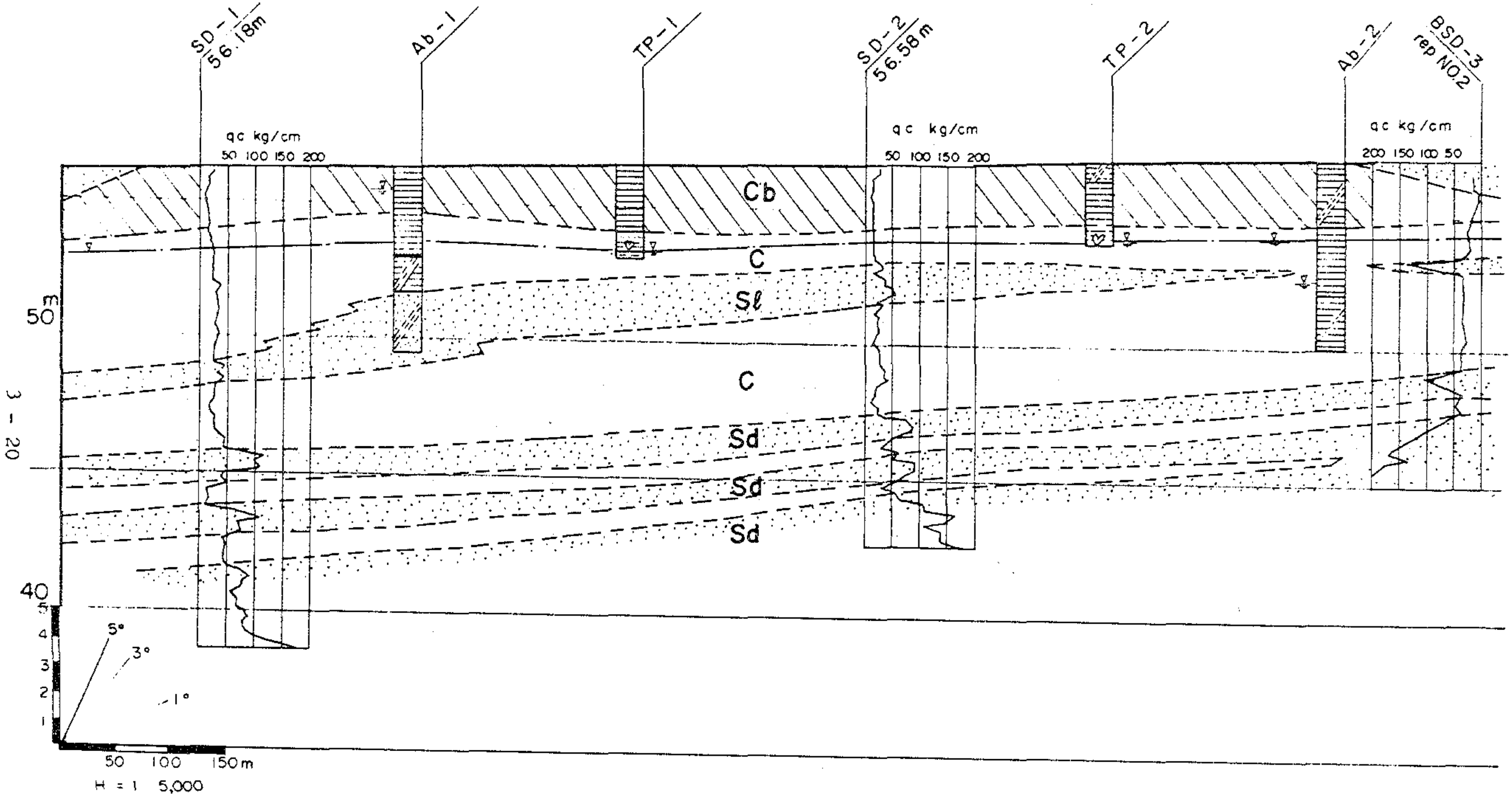


Fig. 3.11 SOIL CROSS SECTION ALONG THE PROPOSED SHORT CUT IN PART 2

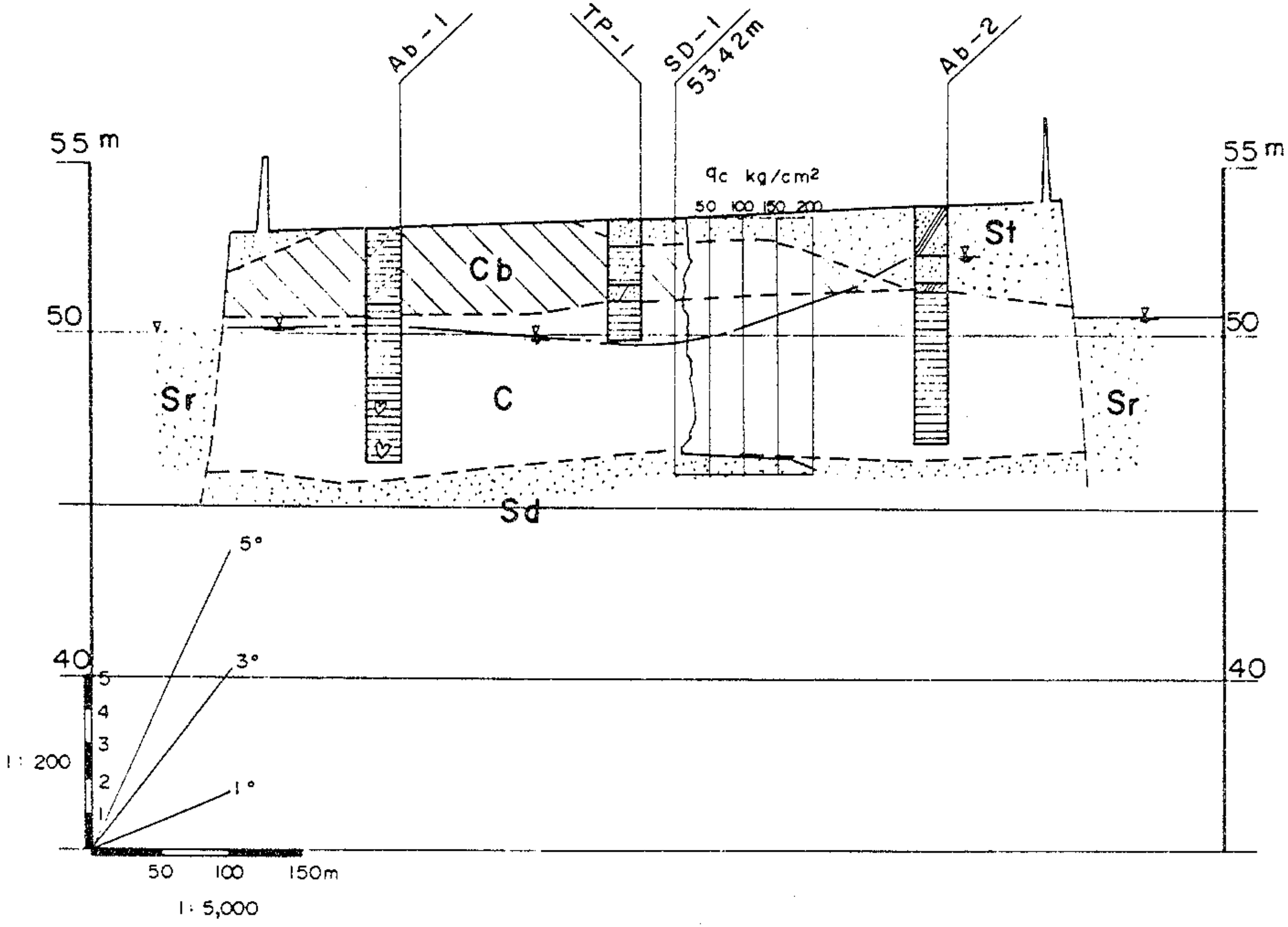


Fig. 3.12 SOIL CROSS SECTION ALONG THE PROPOSED SHORT CUT IN PART 3

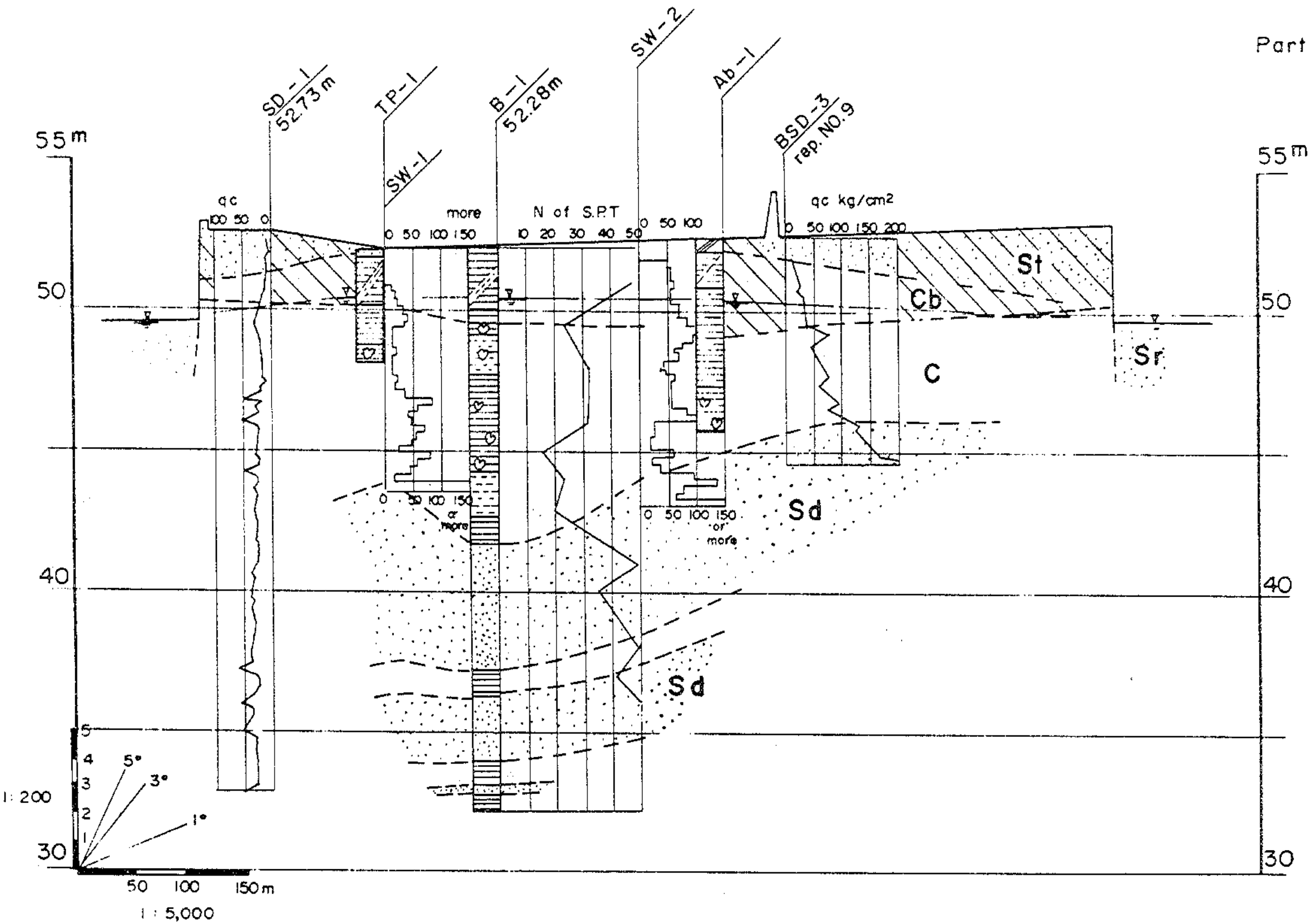


Fig. 3.13 SOIL CROSS SECTION ALONG THE PROPOSED SHORT CUT IN PART 4

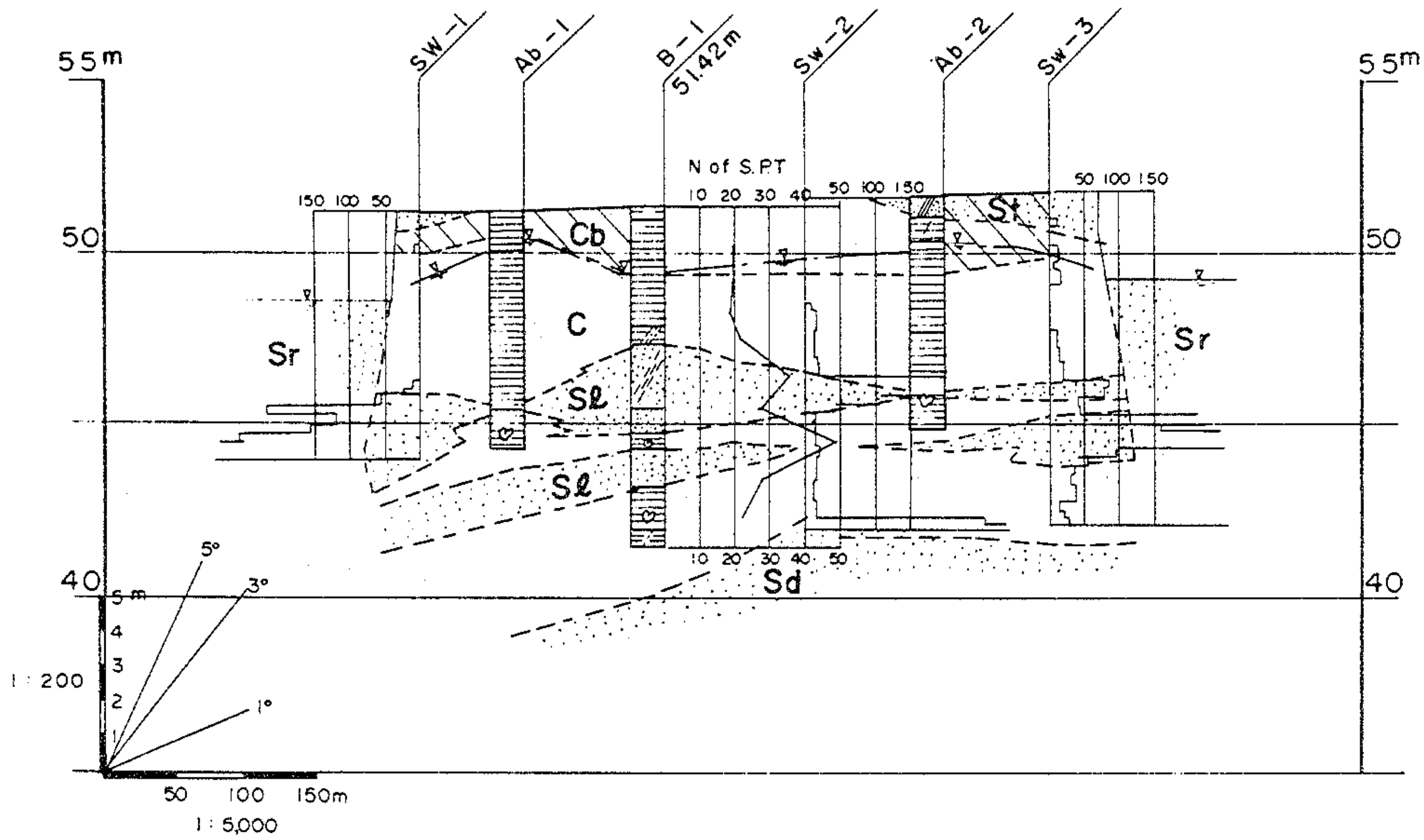


Fig. 3.14 SOIL CROSS SECTION ALONG THE PROPOSED SHORT CUT IN PART 5

Part 1 (A)

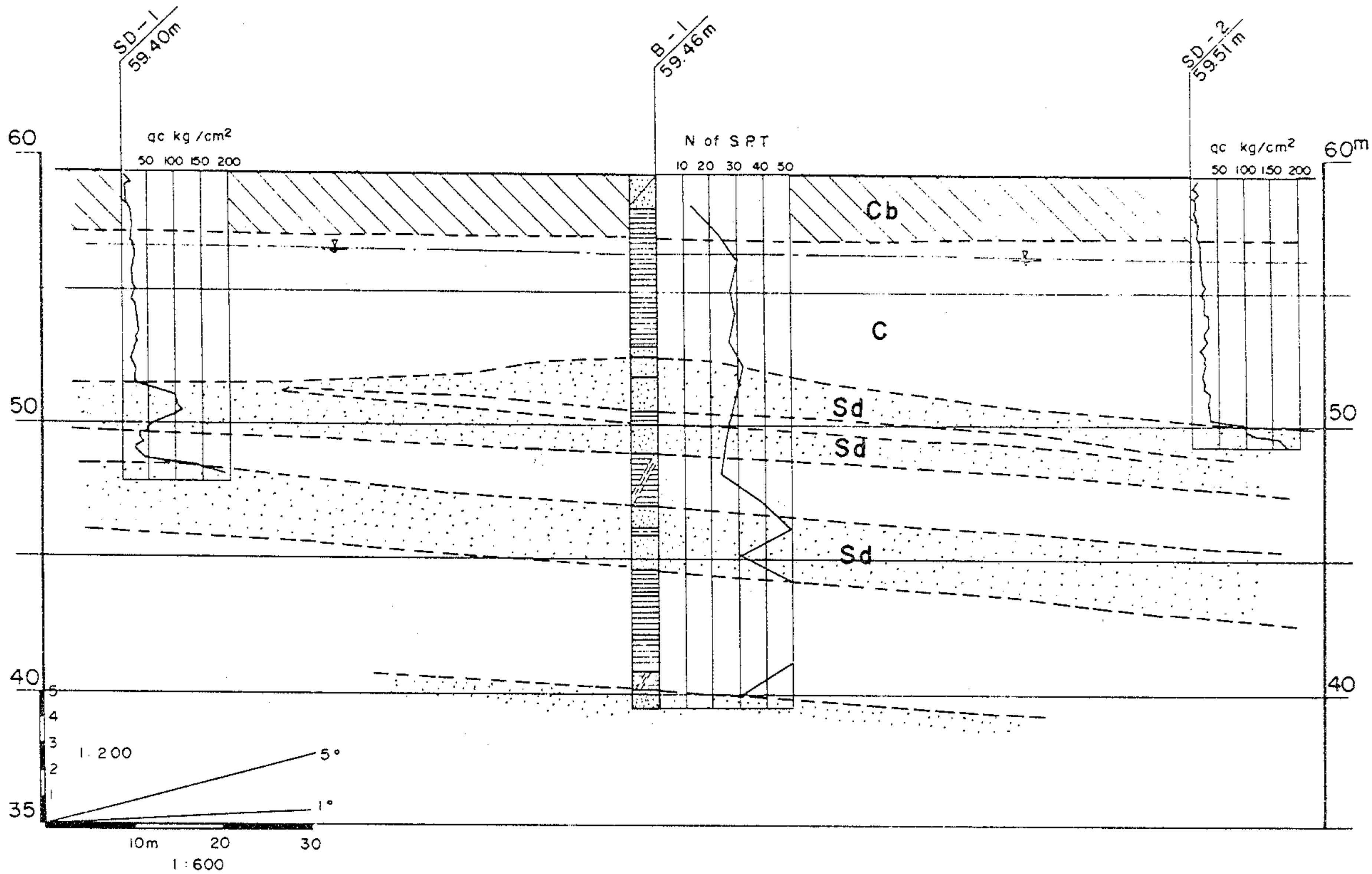


Fig. 3. 15 SOIL CROSS SECTION ALONG THE PROPOSED NEW PATIHAN BRIDGE IN PART I

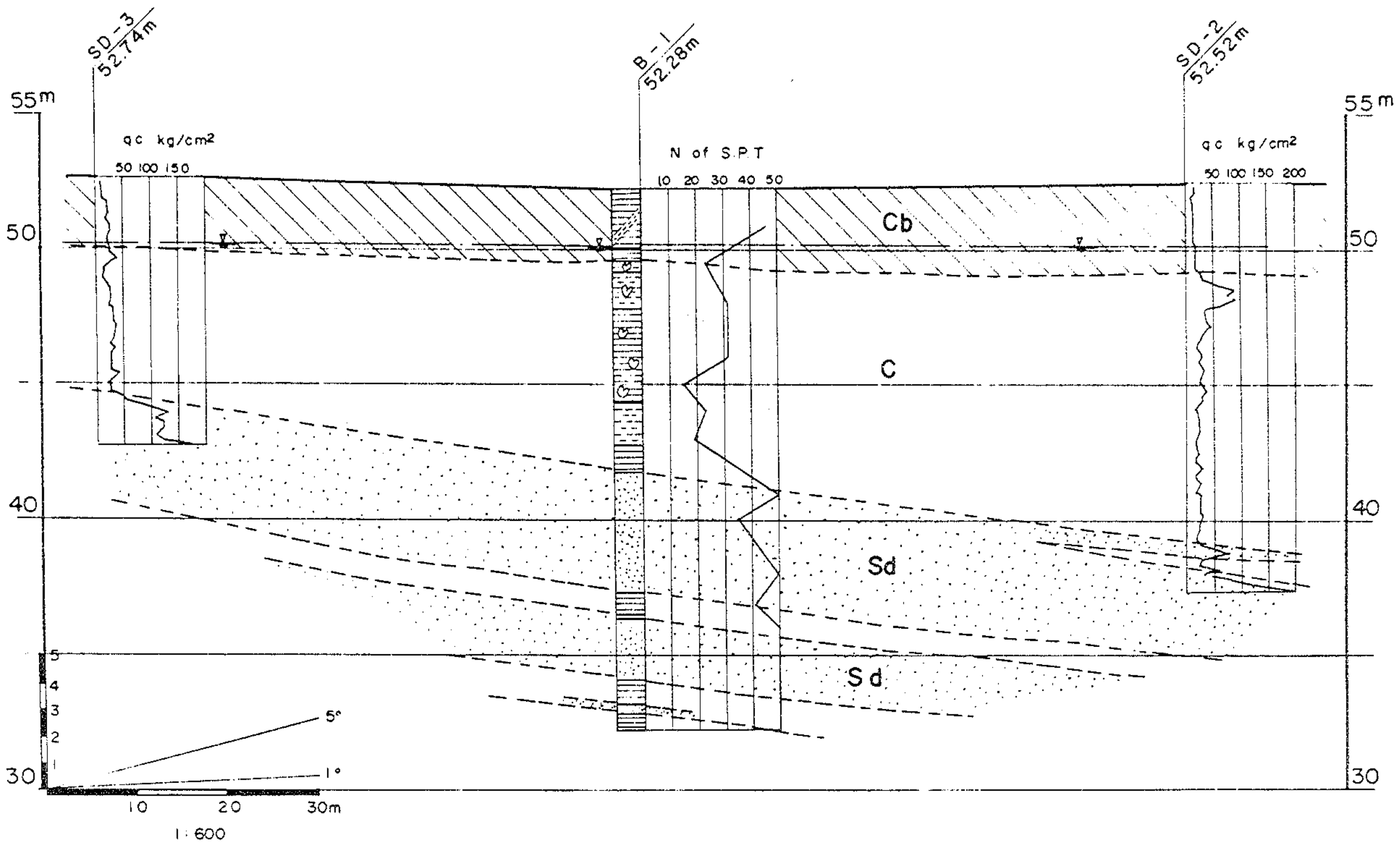


Fig.3.16 SOIL CROSS SECTION ALONG THE PROPOSED NEW KRAMDANG BRIDGE IN PART 4

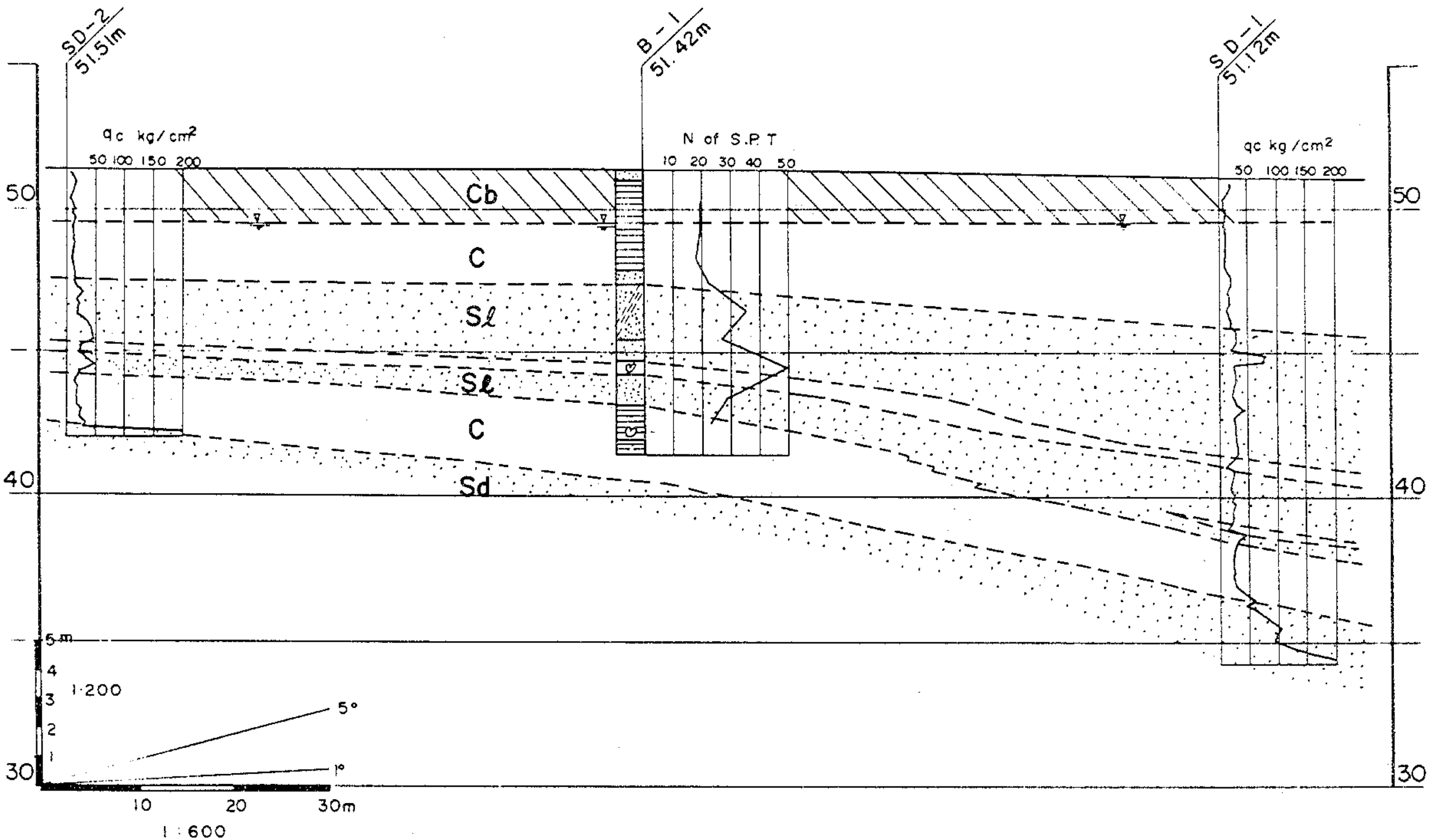


Fig. 3. 17 SOIL CROSS SECTION IN PART 5

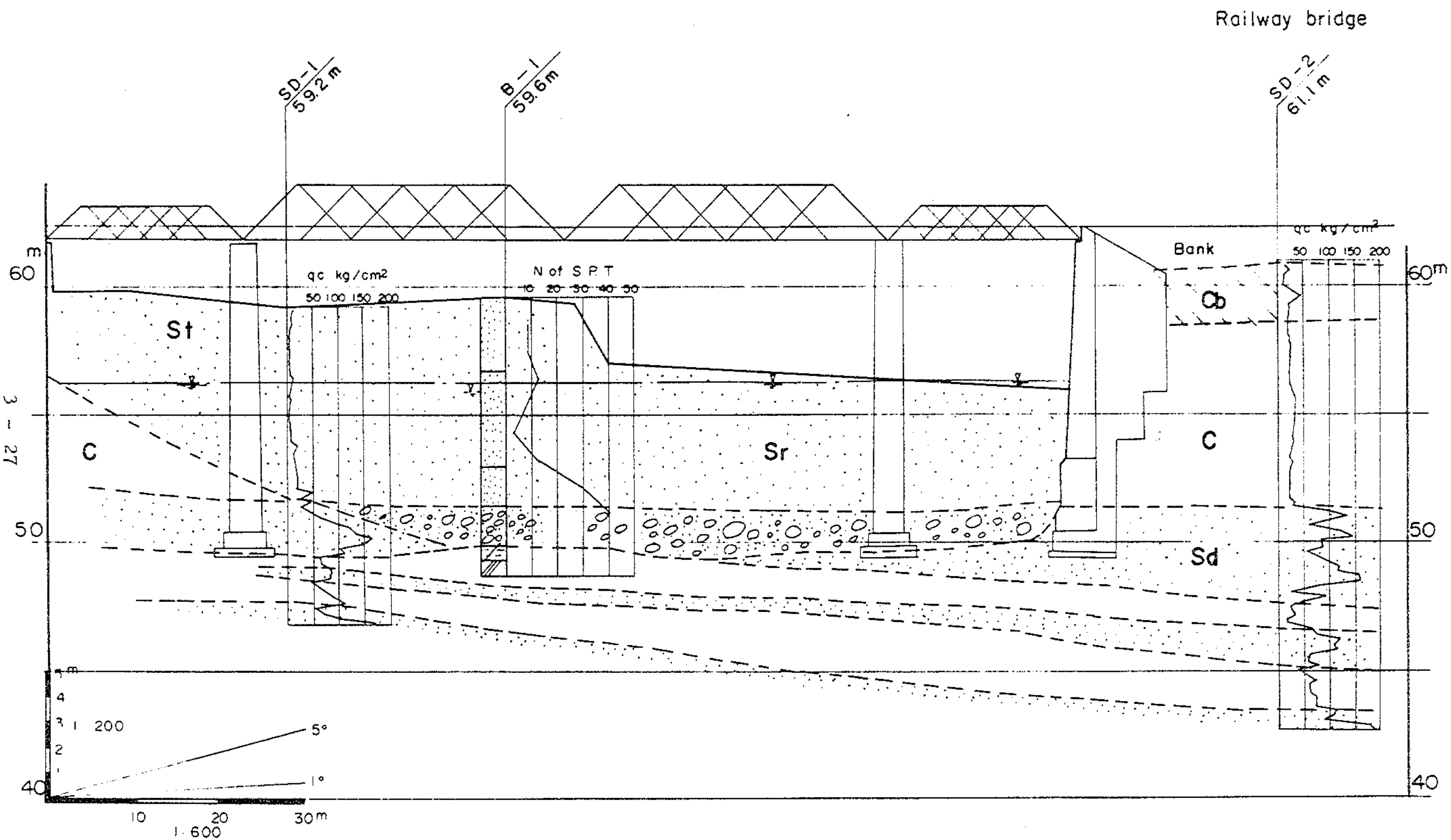


Fig. 3. 18 SOIL CROSS SECTION ALONG EXISTING NATIONAL RAILWAY BRIDGE

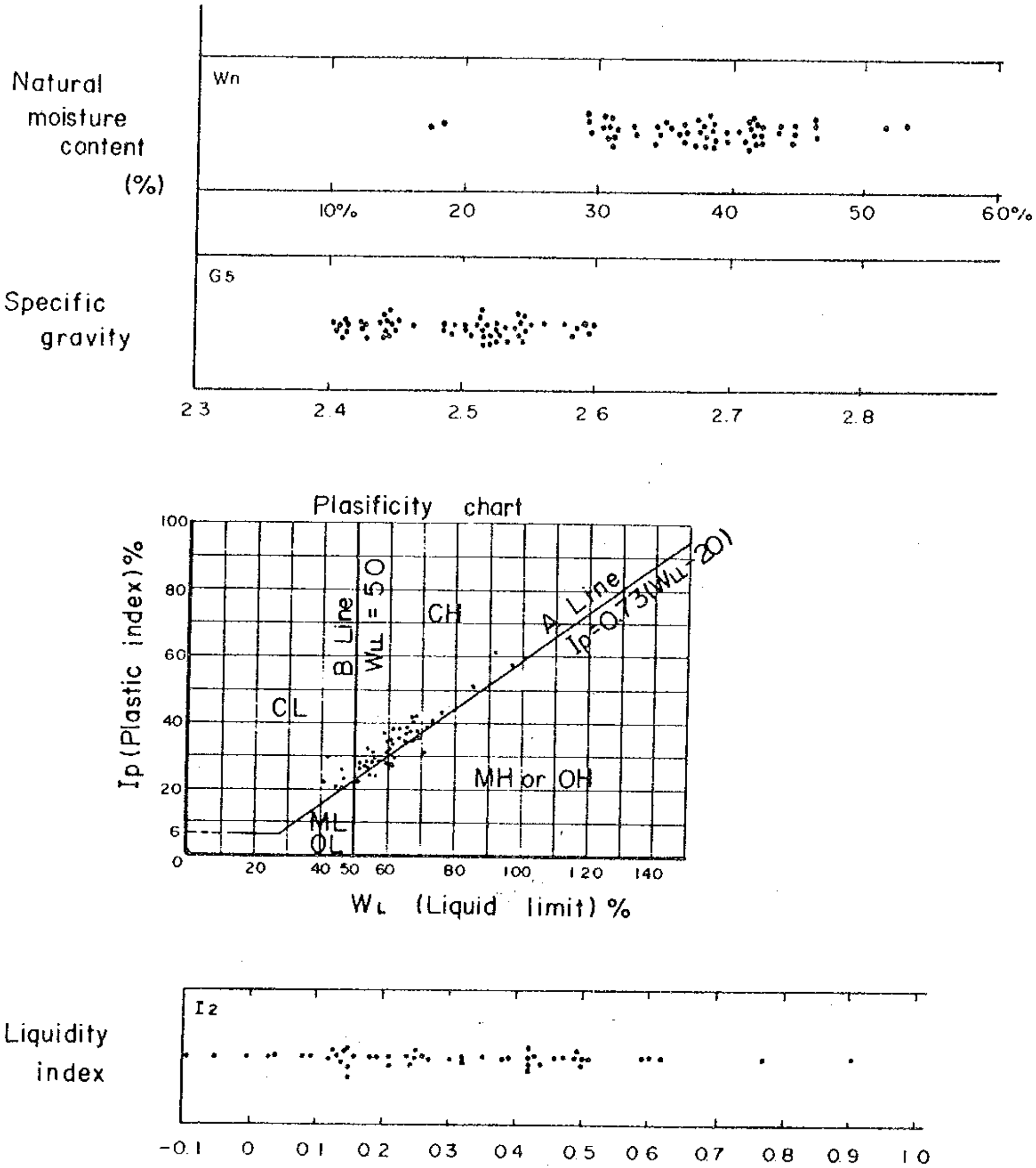


Fig. 3.19 BASIC SOIL PPOPERTY

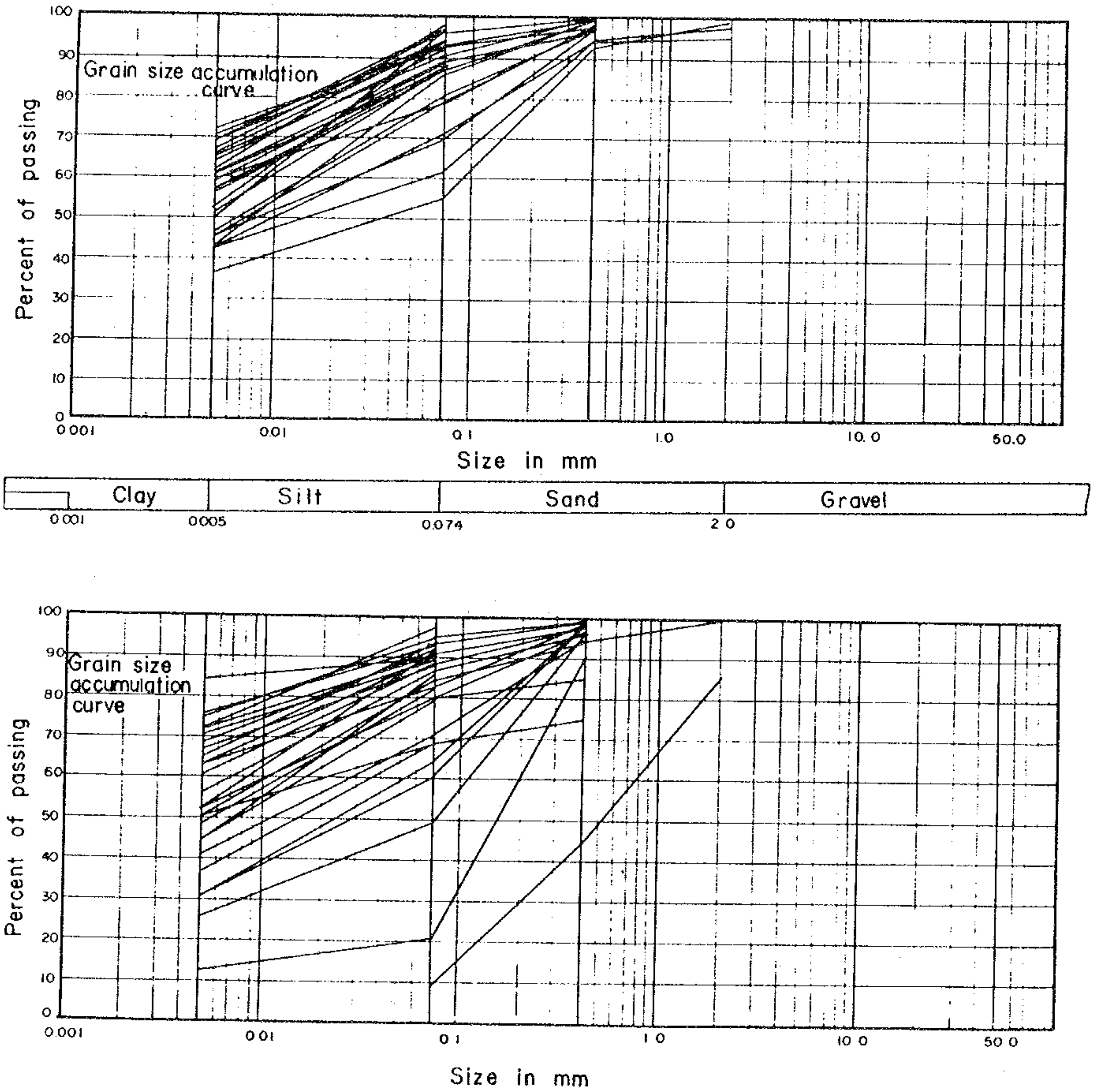


Fig. 3. 20 GRAIN SIZE OF SOIL

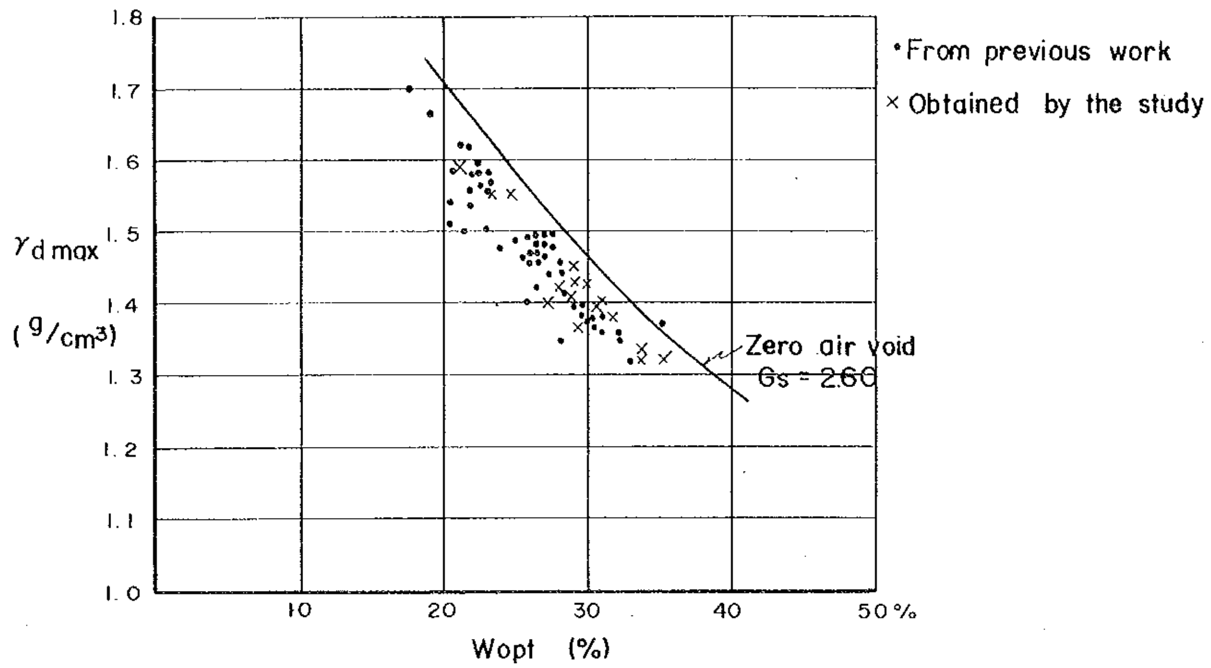


Fig. 3.21 RELATIONSHIP BETWEEN Wopt AND γdmax OF COMPACTION TEST.

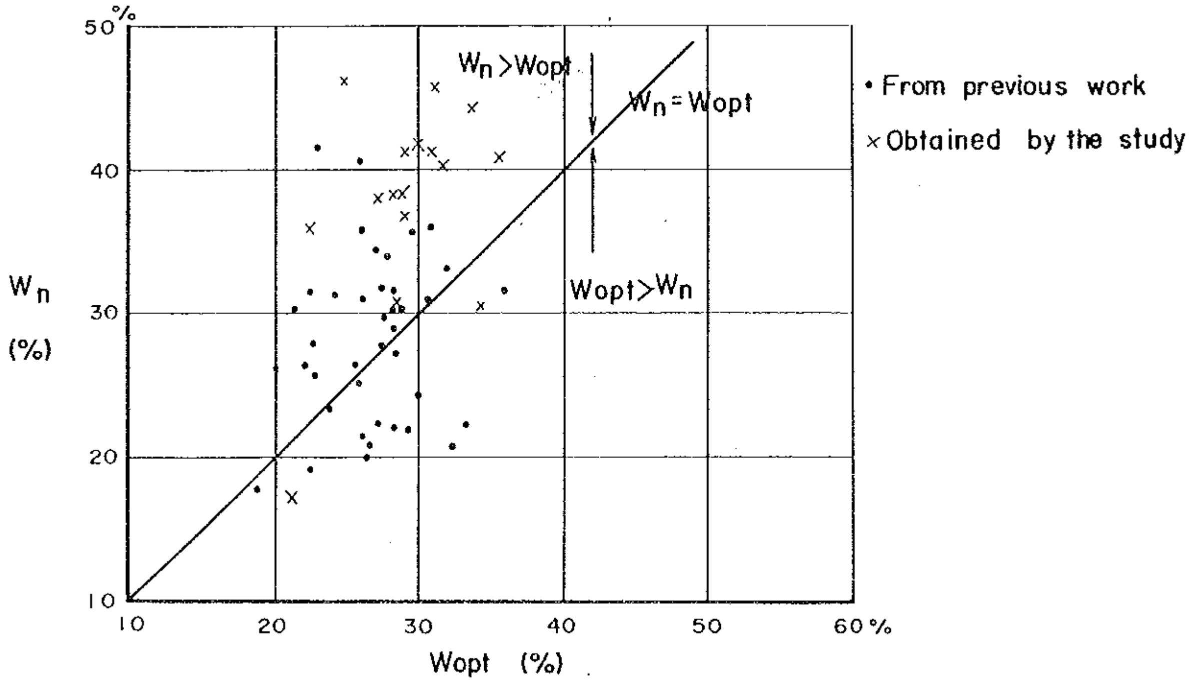


Fig. 3. 22 RELATIONSHIP BETWEEN Wopt AND Wn

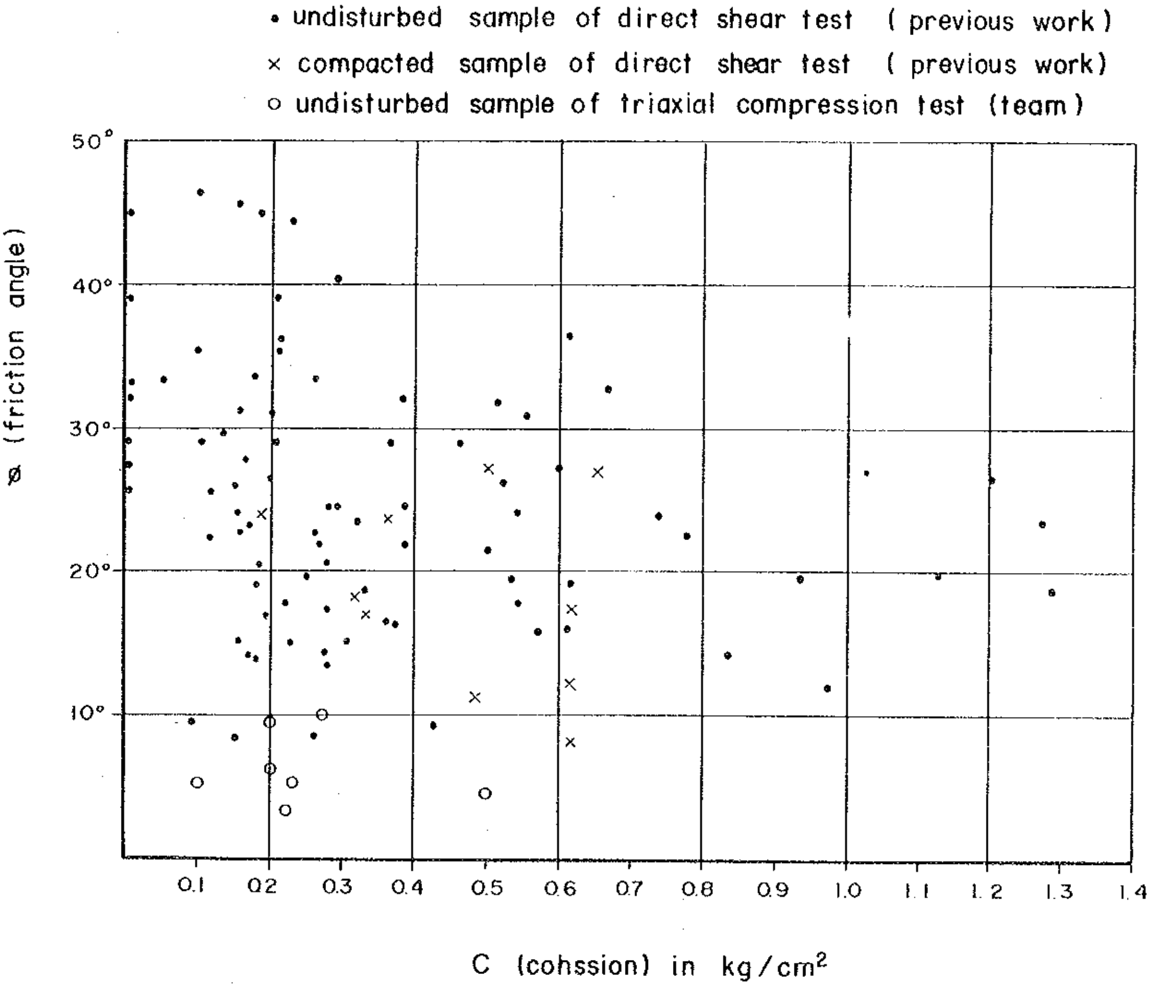
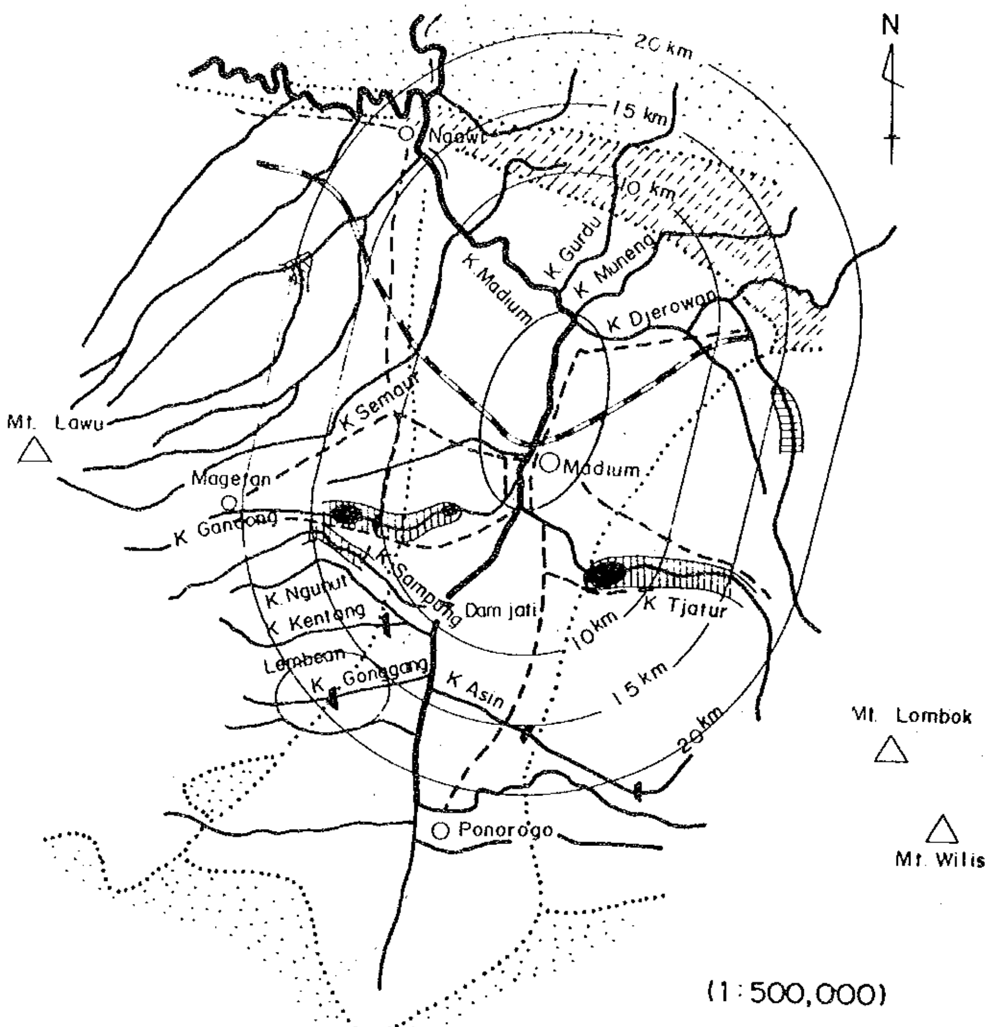


Fig. 3.23 C AND O



Legend

Project area

Existing gravels along the river usable as concrete coarse aggregate

Existing guarry site for coarse aggregate

Existing check dam

Geology

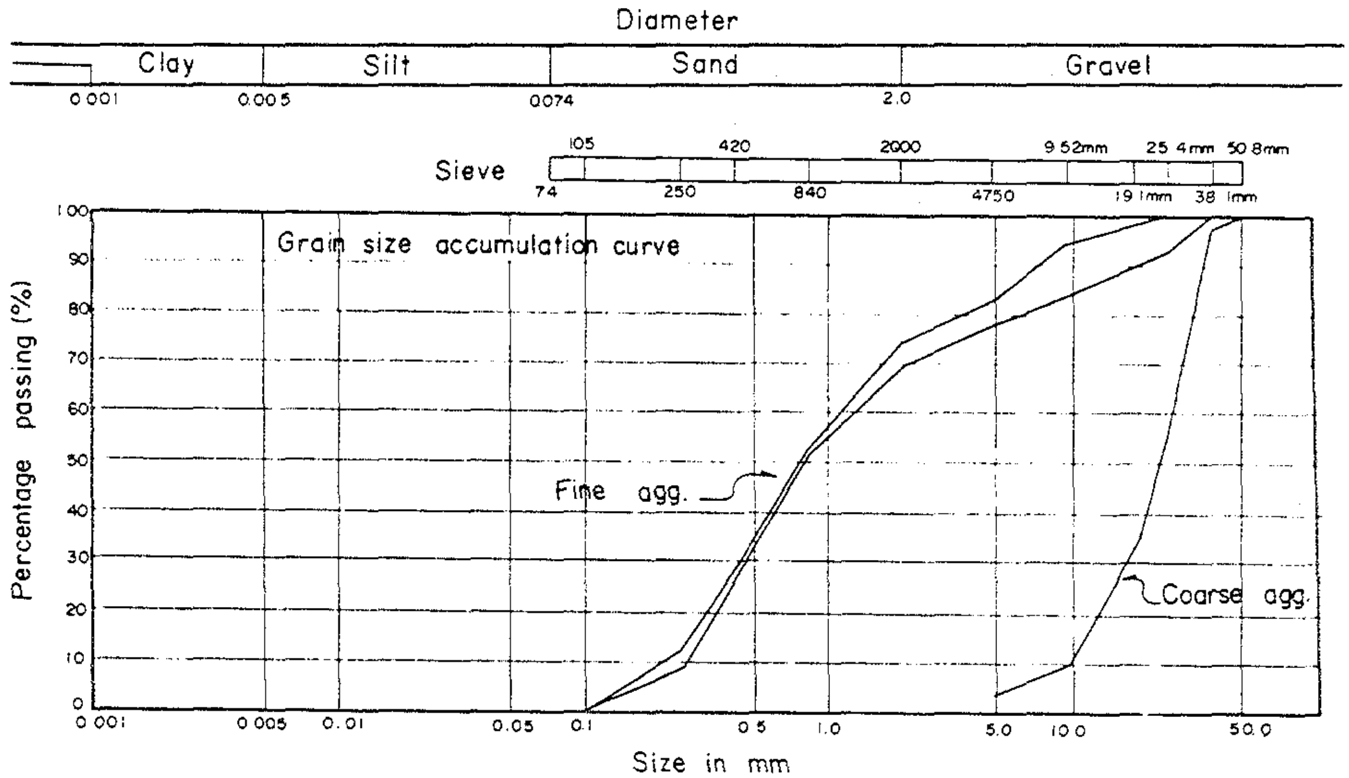
Alluviam

Pleistocene

Tertiary

Quaternary volcanic products

Fig. 3. 24 LOCATION MAP OF RIVER GRAVEL DEPOSIT



Note : — Natural river bed sand used for concret aggregate
— Coarse aggregate mode by manual crushing

Fig. 3.25 GRAIN SIZE OF CONCRETE AGGREGATE

4. HYDROLOGY

4.1 General

The Madiun river takes its origin on the mountain area of the Southern part of Java Island at the west border of the East Java Province. It flows along the Madiun valley which extend from south to north and meets the Bengawan Solo at ngawi. It drains around 3.800 km^2 of land and joins various tributaries before it comes to the confluence with the Solo river.

The river basin is culminated by mount Lawu, 3,265 meters in altitude in the west. The east of the basin is also flanked by the high mountains Dorowati, 2,362 meters and Wilis, 2,169 meters in altitudes.

The profile of the Madiun river has rather gentle slope ranging from 1 : 1.500 to 1 : 3.000. Most of the tributaries, however, have steep slopes; as steep as 1 to 200 even at their most down stretches.

The climate in the river basin is tropical and is dominated by the tropical monsoons.

The north-west wind prevails in the period from November to April and brings wet season in the basin. Whereas the south-east is the dominant wind during the dry season, from May to October.

The average annual rainfall is around 1,900 mm and almost eighty percent of it is concentrated in the wet season.

However heavy rainfall is have been experienced in the transition period from wet to dry season from time to time. And several remarkable floods have occurred in May or June.

Rainfall of high intencity in the steep sloped mountain area are concentrated into the gently sloped Madiun river channel in a short period and tend to cause inundation some sides of the river. A certain portion of the sediment transported in the tributaries are deposited in the Madiun river channel and result in the decrease of the channel capacity.

In the project area, the river stretch between the confluence with the Catur river and one with Jerowan river meander heavily. The decrease of channel slope and accordingly the decrease of channel capacity subsequent to the meander. And it is not seldom that the discharge from the upstream exceed the river channel capacity and inundate some part of the Madiun city.

Large floods in the project area have been occurred in the cases that the heavy rainfall in the Catur river basin and/or the Gandong river basin took place simultaneously with the high water stage in the upper reach of the Madiun river.

No remarkable tributary joins to the Madiun river in the downstream from the Gandong river until it come across the Jerowan river.

The Jerowan river has large catchment area of 453 km^2. But the flood discharge from the Jerowan river is not considered to amplify the flood in the Madiun river seriously because its channel slope is gentle and the peak discharge from the river is presumed to be small. And because the flat area near the junction formed a retardation area of flood runoff.

In the downstream from the junction with the Jerowan the slope of the land become flat. The average river channel slope of the Madiun river is less than 1 to 33000 in this portion. The channel capacity decreased to the range of 200 to 300 m^3/sec. The exceeding discharge from the channel capacity spill out from the channel and widely spread inundating a considerable area.

The river flow in this portion is influenced by the back water from the Solo river.

4.2 Available Meteoro-Hydrologic Data

4.2.1 Rain gage

Rainfall data have been recorded by the PBS, the local irrigation offices and others in the river basin. The established rain gage stations have been located in the basin as exhibited in Fig. 4.1.

Generally the distribution densities of the rain gage stations is high in the flat low lands and it may be able to estimate the areal rainfall within a certain accuracy. But the sparsity of the established rain gages in the mountain areas make it difficult to estimate the basin rainfall from the point rainfall recorded therein.

Average annual rainfall recorded at several gaging stations are shown in Table 4.1.

Most of the rain gages are the ordinary accumulation type. The accumulated rainfall have been observed and recorded once a day, at 6 O'clock in usual. The recorded data have been sent monthly or bi-monthly to the relevant offices. And the daily rainfall data are available for most of the gaging stations.

The automatic recording type gages have been installed in recent years in the existing gaging stations. The logged charts have been brought to the offices concerned and been interpreted into hourly rainfall thereby. Accordingly the hourly distribution of a rainfall is able to study for a point rainfall. However as the recording type gages were installed therein, are scarece, it is not practical to estimate the hourly rainfall in a certain catchment area.

4.2.2 Hydrologic gage

Several hydro-metering stations have been established in the Madiun river basin.

some were constructed as a part of the intake weir structures in the tributaries. Most of these stations are managed by the local irrigation offices. The observations thereof are carried out only in the low flow periods. The observed is limitted to the water level and almost no discharge measurement has been done. However the flood discharge are to be approximated by hydraulic calculation because most of the gages were installed at the crest of the weirs.

In this river basin, some temporary gages were installed mainly in the Madiun river itself. The gages were brought and observed in order to obtain the data for the specific hydrologic studies. They were abandoned and no observation is continued after the studies were terminated.

The most useful hydrologic gaging stations for the study are ones established in the Madiun river main stem as follows;

Sekayu gaging station; Automatic recording type gage was installed near the Ponorogo town. The river channel is kept natural.

Dam Jati gaging station; Located on the rectangle spillway of the Jati intake weir. The crest length is 41 meters. The weir is designed not to be submerged by flood. The weir was constructed to control the intake water level and no storage reservoir is afforded thereby.

The installed is the ordinary staff gage and observation has been done three times a day in low flow period. Hourly observation has been carried out by the staff of the irrigation office in high water period.

Mangunbarjo Gaging station; Located at just upstream from the town of the Madiun city. The automatic recording type gage was installed and has been maintained well. The river bed at the site, however, is variable because a aggregate production company has exploited considerable volume of sand from the river bed. And the data recorded at the station are deemed to be available only for the confirmation.

A. Yani gaging station; Located in the Madiun city. The ordinary staff gage was installed. The hourly water level records are available. Discharge measurement have been carried out periodically at the site. The cross sectional survey carried out for the discharge measurement prove the stability of the river bed at the site.

The recorded data at the site are to be adopted as the basic hydrologic data for the study.

Dungus gaging station; Located near Ngawi city. The ordinary staff gage was installed and have been observed by the local irrigation office.

The hourly water level have been recorded in high water period.

Discharge measurement have been carried out and once water stage-discharge rating curve was developed. But site is located near the confluence with the Solo river and is seriously influenced by the back water therefrom. Consequently the relationship between the water stage and the discharge disturbed to some extent.

The data recorded at the Kajangan and the Karang Nongko gaging stations situated in the Solo river provide the useful information to the hydrologic study for the Madiun river improvement project.

The principal features of the gaging stations are summarized in Table 4.2.

4.2.3 Discharge measurement at A. Yani

The discharge measurement at the A. Yani gaging station was commenced on February 3, 1971 by the Institute of Hydraulics, Bandung. The measurement work was also carried out in 1973 by the OTCA, Japan; the study team of the Bengawan Solo Basin Master Planning. The measured results were studied and a water stage-discharge curve was developed.

In 1975, the remarkable flood occurred in the Madiun river and the substantial part of the Madiun city was submerged by the inundated flood runoff. The PBS constructed dykes on the both river bank for 15 km up and downstream of the Madiun city. And the gaging station was left the outside of the dykes, the river channel side.

As the dykes were constructed, the river cross section and other hydraulic conditions were changed. The PBS performed the discharge measurement several times a year since 1977 and the new rating curve was established.

The actual discharge measurement were conducted at the Winongo Railway bridge site by measuring the flow velocity with the price meter for each subsection. The discharge were obtained as usual by the products of the velocity and the flow area.

The rating curves were developed correlating the discharge obtained as above and the simultaneously observed water level at the A. Yani gaging station.

The results of the discharge measurement were listed in Table 4.3 and the developed rating curves were shown in Fig. 4.2.

The new rating curve developed by the PBS is separated into two curves at the height of 5.60 meters due to the base line of the constructed dykes. This separation reflects well the actual discharge measurement data as can be seen in the figure.

4.2.4 Hydrographic data

The hourly water level have been recorded and the rating curves were developed for the Sekayu, Dam Jati, A. Yani and Dungus gaging stations. And the flood hydrographs drawn thereby are available.

The flood occurred in the upstream area from the Sekayu gaging station inundate in and around the Ponorogo city. The hydrograph of the site show rather flat wave due to the retardation in the inundation area. The average time of concentration is estimated to be in the range of 9 to 10 hours.

The inflow to the Dam Jati is raised its water level by the intake weir. The gradient of the water surface slope is decreased and the flood water flows in a wide river channel with large channel storage capacity. Accordingly the recorded flood, outflow from the weir, have small peak and long duration at this site.

The river channel become narrow in the portion a little up-and downstream of the A. Yani gaging station. The river banks were arranged by dykes. And there are no significant retarding area in the upper reach of the station. Consequently the flood hydrograph developed from the flood data recorded at the A. Yani gaging station have sharp peak compare with ones at the Sekayu and the Dam Jati gaging station.

No typical flood hydrograph can be obtained from the flood data at the Dungus gaging station due to the heavy influence of the water level of the Solo river.

The typical flood hydrograph are shown in Fig. 4.3.

The dimensionless hydrograph was established for the flood hydrograph at the Dam Jati by P2AT, Madiun, the Madiun Ground Water Development Project. In this study the ordinate and the abscissa of the hydrograph are normalized by the following manner;

Ordinate (y) : $T_p \cdot V_x / V_T$
Abscissa (x) : t / T_p

where,

T_p : Lag time between the beginning of flood and the peak.
V_x : Momentary flood volume correspond to abscissa (x).
V_T : Total flood volume, integration of V_x.
t : Time after the beginning of the flood.

The same was applied to the hydrograph of the flood at the Sekayu and the A. Yani gaging stations and the dimensionless hydrograph of the stations were developed.

The obtained ordinates and abscissas for the dimensionless hydrograph are shown in Table 4.4 together with ones for the Dam Jati.

The hydrograph at the A. Yani gaging station was generated by applying the ordinate and abscissa to the recorded peak discharge of 790 m^3/sec, T_p of 10 hours and the base flow of 230 m^3/sec on June 9, 1977. The calculation of the generation was listed in Table 4.5 and the generated is shown in Fig. 4.4.

4.2.5 Probable flood

Several studies has been conducted to analyse the flood of the Madiun river. The probable flood were estimated for the Dam Jati and Karang Nongko gaging stations by the Madiun Ground Water Development Project, P2AT in 1976 and for the Sekayu gaging station by the Wonogiri Multipurpose Dam Project, PBS in 1978.

The Gumbel's method was applied to the estimation described above and the estimated result are as follows;

	Return period in Year				
Gaging Station	2	5	10	20	50
Sekayu (PBS)	320	360	385	405	445
Dam Jati (P2AT)	440	520	630	740	840
Karang Nongko	1,550	1,790	1,950	2,100	2,300

4.3 Estimation of Annual Maximum Discharge at A. Yani

4.3.1 Flood occurred before 1977

The hourly water level record since 1963 were kept in the data table in the gaging station. And the annual maximum water level were retrieved from the original data table.

The annual maximum water level have occurred in March and April frequently.

The momentary peak water level were converted to discharge applying the rating curves. The rating curve established by the OTCA and the DPMA was used to estimate the discharge occurred before 1975 and one by the PBS was used to estimate the annual maximum momentary peak discharge in 1976 and 1977.

The obtained peak discharges were summarized in Table 4.6 together with ones occurred in 1978 and 1979.

4.3.2 Flood occurred in 1978 and 1979

Flood occurred in 1978 and 1979 were remarkable ones. These flood over-topped from the crest of the right dyke near Patihan around two km downstream from the A. Yani gaging station. The dyke was breached at the over-topped portions for 30 to 40 meters to the foundations of the embankment.

There remain no exact information concerning the breakdown of the dyke. But it is considered that the dyke was breached to the foundation when the peak discharge arrived the A. Yani gaging station. Because the water level at the gaging station kept its highest water level for five hours in both cases.

The water surface slopes might become steeper than the usual because the water level at Patihan became low due to the breakdown of the dyke. And the discharge capacity of the river channel at A. Yani was to be increased. Accordingly the following hydraulic approach was examined to estimate the peak discharge occurred in 1978 and 1979 instead of simple application of the rating curve.

The procedure of the hydraulic approach is as follows;

1) Water level at Patihan was assumed.

2) Water surface slope between A. Yani and Patihan was obtained corresponding to the water level in (1) because the water level at A. Yani was fixed to the maximum.

3) The channel discharges at A. Yani and Patihan were estimated for each water slope in (2). The Manning's Uniform flow formula was used with the coefficient of roughness of 0.03.

4) The discharge from the broken dyke were calculated for each water level at Patihan in (1). Rectangle sections and the critical flow condition were assumed for the calculation.

5) The total discharge at Patihan was obtained as the summation of the channel discharge and one from the rent in dyke.

6) A kind of trial calculation was applied to find the water levels which bring the same amounts of discharge for A. Yani and Patihan. The discharge thus obtained was adopted as the maximum peak discharge.

In actual calculation, the following data and assumption were adopted.

For flood in 1978: Water level at A. Yani 9 : 3. Rectangle rent in the dyke of 40 meters width at the section No. HI.127.

For flood in 1979: Water level at A. Yani 9 : 3. Rectangle rent in the dyke of 40 meters width at the section No. HI.129.

The annual maximum peak discharge of 1,200 m^3/sec and 1,010 m^3/sec were obtained in this manner for the year of 1978 and 1979 respectively. The procedures of the estimation are illustrated in Fig. 4.5 and Fig. 4.6.

The estimated results were confirmed through the daily rainfall in the remnant catchment area between the Dam Jati and the A. Yani gaging stations. The average rainfall of 100 mm and 18 mm were received in the remnant catchment area in 1978 and 1979 respectively.

The statistics of the records bring the estimated remnant discharge of 443 m^3/sec for the rainfall of 100 mm and 161 m^3/sec for 18 mm. The peak discharges of 1,220 m^3/sec and 1,041 m^3/sec in 1978 and 1979 were obtained because the discharges at the Dam Jati gaging station in 1978 and 1979 were 780 m^3/sec and 880 m^3/sec respectively.

Consequently the flood discharges discussed above were adopted as the annual maximum discharges in 1978 and 1979.

4.4 Estimation of Probable Flood Discharge

The probable peak discharge at the A. Yani gaging station site were estimated applying the Gumbel's method to the estimated annual maximum peak discharge. The annual maximum peak discharge for 17 years from 1963 to 1979 bring the following probable peak discharge.

Return period in Year	Peak discharge in m3/second
2	757
10	1,110
20	1,245
50	1,419
100	1,550

The regression curve of the return period to the peak discharge was developed and shown in Fig. 4.7.

The analysed results show the recorded maximum peak discharge of 1,200 m^3/sec has the return period of around 17 years. And the flood of the peak discharge more than 760 m^3/sec may occurs every two years.

The estimated probable peak discharge at the A. Yani gaging site seems unnaturally high comparing ones at the Sekayu and the Dam Jati gaging stations. However the correlation between the catchment area and the flood volume obtained from the estimated probable peak discharge show convex curves as shown in Fig. 4.8. Thus the estimated probable peak discharge were proved to maintain the hydrological consistency.

4.5 Estimation of Runoff from the Tributaries

4.5.1 Method of estimation

Several tributaries such as the Ashin, the Gonggang rivers and others join the Madiun river in the portion between the Sekayu and the Dam Jati gaging station. The total of the remnant catchment area in this portion amounts 658 km^2.

The Catur and the Gandong rivers are the substantial tributaries which come into the Madiun main stem between the Dam Jati and the A. Yani gaging stations. The catchment area of 580 km^2 remains between both stations.

The flood discharge from these tributaries influence the flood in the Madiun river.

The maximum flood discharge of 1,200 m^3/sec was recorded at the A. Yani gaging station on April 18, 1975. The corresponding peak discharge at the Sekayu gaging station was 244 m^3/sec and at the Dam Jati gaging station, 830 m^3/sec. The difference of these figures are considered to be brought by the flood from the tributaries.

The differences between the peak discharge recorded at the different stations were studied for the remarkable flood occurred in 1975, 1978 and 1979.

The corresponding peak discharge recorded at three gaging stations are as follows:

Gaging Station	Catchment area (km2)	Peak Discharge 1975 (m3/sec)	1978 (m3/sec)	1979 (m3/sec)
Sekayu	1,056	244	414	450
Dam Jati	1,714	830	820	880
A. Yani	2,294	1,200	1,200	1,010

The contribution of the local inflow from the tributaries to the flood peak of the Madiun river were estimated as follows;

Remaining Basin	Catchment area (km2)	Local inflow 1975 (m3/sec)	1978 (m3/sec)	1979 (m3/sec)
Sekayu - Dam Jati	658	586	406	430
Dam Jati - A. Yani	580	370	380	130

The average contribution of the remaining basin between Sekayu and Dam Jati was equivalent to the specific discharge of 0.76 $m^3/sec/km^2$.

Whereas one from the basin between Dam Jati and A. Yani was 0.52 $m^3/sec/km^2$. The average of one from the remaining basin between Sekayu and A. Yani was 0.64 $m^3/sec/km^2$.

On the other hand, the recorded maximum flood at the A. Yani gaging station of 1,200 m^3/sec has the 17-year return period. The respective probable flood with same return period at the Sekayu gaging station and the Dam Jati gaging station are 404 m^3/sec and 713 m^3/sec. In these cases the differences between the flood peak magnitudes are 309 m^3/sec for the Sekayu and the Dam Jati gaging stations and 487 m^3/sec for the Dam Jati and the A. Yani gaging stations. The respective specific discharge are 0.44 and 0.84 $m^3/sec/km^2$ and the mean of them is 0.64 $m^3/second/km^2$.

In this respect the specific discharge of the tributaries was assumed 0.64 $m^3/sec/km^2$ in case the flood discharge at the A. Yani gaging station is 1,200 m^3/sec.

4.5.2 Allocation of discharge

As described before the major tributaries, the flood therefrom influence the peak discharge of the Madiun main stem, are the Catur, the Gandong and the Jerowan rivers in this project area.

The flood peak of 1,200 m^3/sec at the A. Yani gaging station reflect the influence of the local inflow from the Catur and the Gandong rivers. There is no significant local inflow until the flood reaches the confluence with the Jerowan river. At the confluence the flood discharge is affected by the local inflow from the Jerowan river.

The influence of the local inflow is evaluated by the specific discharge of 0.64 $m^3/sec/km^2$ in case the peak discharge at the A. Yani gaging station is 1,200 m^3/sec. For the flood other than the one of 1,200 m^3/sec, the influence is to be evaluated by the specific discharge modified from 0.64 $m^3/sec/km^2$ in proportion to the magnitude of the peak discharge at the A. Yani gaging station.

The allocated discharge for the several peak discharge at the A. Yani gaging station were summarized in Table 4.7.

4.6 Discharge in the Downstream from the Confluence with the Jerowan River

4.6.1 Jerowan inundation area

A dyke was constructed around 500 m upstream from the confluence of the Jerowan river on the left side bank of the Madiun river. But the right bank dyke was constructed only 2 km upstream from the confluence and the land is exposed to the inundation. Especially the triangle land located between the Madiun and the Jerowan rivers is comparatively low in its altitude and have been submerged even by the usual high water.

The area forms a natural retardation area in the river stretch. The flood from the Madiun river and/or the Jerowan river flow into the area and flow out to the downstream after decreasing their peak.

No hydrologic gage were installed in the Jerowan area and no actual record were available to evaluate the effect of the area on the flood discharge.

In this respect the storage volume curve of the inundation area and the discharge rating curve were developed from the topographic map and the non-uniform flow calculation as shown in Fig. 4.9 and 4.10.

The regulating effects of the retarding area were studied on a few probable floods applying the storage volume curve and the discharge rating curve as shown in Fig. 4.11, 12 and 13.

The peak inflow of 1,490 m^3/sec to the Jerowan inundation area is estimated from the peak discharge of 1,200 m^3/sec at the A. Yani gaging station since the additional inflow from the Jerowan river will amplify the peak by 290 m^3/sec as described in the Table 4.7. The peak outflow from the inundation area is estimated to be 1,190 m^3/sec through the regulation calculation applying the storage volume curve and the discharge rating curve.

4.6.2 Runoff in the downstream reach

Several floods recorded at the A. Yani and the Dungus gaging stations were correlated.

The flood of 1,200 m^3/sec occurred in 1975 dispersed in the vast area from the Madiun city to the downstream of the Jerowan confluence and caused the flood of 890 m^3/sec at the Dungus gaging station. Whereas the flood of the same magnitude at the A. Yani station occurred in 1978 caused the flood of 675 m^3/sec at the Dungus nevertheless the flood inundated only the downstream from the Madiun city.

The smaller flood of 1,010 m^3/sec occurred in 1979 also inundated the area downstream from the Madiun city due to the dyke breach at Patihan. The flood caused larger flood of 930 m^3/sec at the Dungus gaging station.

The inconsistecies of the flood at the Dungus gaging station might be caused by the local inflow from the remaining catchment area, the inundation along the river and the back water from the Solo river.

A few tributaries join the Madiun river in the downstream reach from the confluence with the Jerowan river. The influence of the discharge from the tributaries to the one of the Madiun main stem, however, is not able to estimate by applying the specific discharge of 0.64 m^3/sec per km^2 likewise the tributaries in the upstream. Because the river bed slope of the tributaries are flat and a flood from a tributary itself inundate in its catchment and is assumed to have a small peak and a long duration. Accordingly the influences of the tributaries in this portion deemed far less than ones in the upstream.

In this portion the flood discharge of the Madiun river spillout from the river banks and inundate the vast land especially in the left side. The retardation of the flood runoff in the inundation area is one of the most important element among the various ones which affect the flood discharge in the downstream from the Jerowan confluence. The fact described above, however, imply that the inundation in the upstream stretch from the Jerowan confluence has rather small influence and the inundation in the downstream area from the confluence including the Jerowan inundation area have the major influence to the flood discharge in the downstream.

The peak outflow from the Jerowan inundation area was estimated to have been 1,190 m^3/sec for the flood in 1975 based on the flood of 1,200 m^3/sec at the A. Yani gaging station. The flood magnitude decreased to 900 m^3/sec at the Dungus gaging station in spite of the remnant catchment area of 900 km^2. No hydrological data are available in the portion between the A. Yani and the Dungus gaging stations. And it is difficult to say the decreasing tendency of the discharge from place to place in the river reach. However, the discharge may be assumed to have decreased from 1,190 m^3/sec to 900 m^3/sec in proportion to the distance from the Jerowan inundation area because there are no remarkable retarding area and remarkable tributary in this portion.

The flood at the Dungus gaging station is actually influenced by the water level of the Solo river as described before. However the available data are too scarece to discuss the combined probability of the floods in the Madiun river and the Solo river. And the flood at the Dungus gaging station were considered to be obtainable by the estimation from the flood magnitude of around 900 m^3/sec recorded in 1975 and 1979 in proportion to the magnitude of the outflow flood from the Jerowan inundation area.

Table 4.1 Average Annual Rainfall

No.	Station no.		Name	Elevation	Observation years during the period 1941 - 1975	Mean annual rainfall (mm)
1	Mediun	6	Kendel	300	28	2,722
2	"	13	Poncol	875	26	2,116
3	"	16	Hgawi	50	25	2,002
4	"	27	Purwodadi	75	25	1,919
5	"	29	Bogem	130	27	1,718
6	"	32	Lembean	125	27	1,850
7	"	33	Sungkur	110	29	1,723
8	"	34	Kwadungen	50	26	1,832
9	"	38	Kanigoro	70	28	1,801
10	"	43a	Gombal	140	28	1,750
11	"	44	Ponorogo	92	26	1,720
12	"	46	Slahung	65	25	1,952
13	"	50	Tulung	90	26	1,782
14	"	52b	Wates	145	24	1,828
15	"	55	Gondosuli	600	20	2,255
16	"	59	Pudak	1,250	26	2,224
17	"	60a	Kesugihan	360	24	2,074
18	"	61	Sawo	100	25	1,724

Table 4.2 Principal Features of Gaging Station

Name of Gaging Station	Catchment Area (km^2)	Elevation of Zero Point	Type of Equipment	Managed by	Starting Year of Operation
Madiun river					
Sekayu	1,056	86,588	A.R [1]	P.B.S. [3]	1975
Dam Jati	1,714	73,750	S.G [2]	I.O. [4]	1952
A. Yani	2,294	52,800	S.G	I.O.	1952
Mangunharjo	2,294	57,851	A.R	D.P.M.A. [5]	1970
Dungus	3,755	33,410	S.G	I.O.	1970
Solo river					
Kajangan	5,463		A.R	P.B.S.	1975
Karang Nongko	10,037	0	S.G	I.O.	1952

Remarks
1) Automatic recording type gage
2) Staff gage
3) The Project Bengawan Solo
4) Local irrigation office
5) Institute of Hydraulics, Bandung

Table 4.3 Discharge Measurement Data

Water level (m)	Velocity (m/sec)	Discharge (m3/sec)	Date	Carried out by
5.25	0.77	42.5	Feb. 3,'71	LPMA
6.17	0.83	110.8	Feb.20,'73	OTCA
6.83	2.05	220.9	Mar.14,'73	OTCA
5.72	0.81	63.5	Mar.20,'73	OTCA
8.09	1.76	355.3	Mar.28,'73	OTCA
5.74	1.38	140.5	Feb. 4,'77	PBS
4.80	0.51	22.9	Mar.20,'77	PBS
6.33	1.18	184.1	Jul. 5,'78	PBS
7.23	1.39	295.2	Jul. 6,'78	PBS
7.65	1.58	377.2	Jul. 6,'78	PBS
4.02	0.44	6.6	Dec. 7,'78	PBS
7.10	1.44	276.5	Mar.29,'79	PBS
8.35	1.69	548.8	Jan.22,'80	PBS
7.95	1.56	430.4	Jan.22,'80	PBS
6.25	0.98	170.0	Jan.23,'80	PBS

Table 4.4 Dimensionless Hydrograph

Absoissa x	Ordinate							
	Sekayu $T_p = 9$		Dam Jati #) $T_p = 7$		A. Yani $T_p = 7$		Karang Nongko #) $T_p = 75$	
	t	y	t	y	t	y	t	y
0	0	0	0	0	0	0	0	0
0.3	3	0.20	2	0.16	2	0.34	22.5	0.13
0.5	4.5	0.40	3.5	-	3.5	0.45	37.5	-
0.7	6	0.55	5	0.38	5.0	0.55	52.5	0.50
1.0	9	0.64	7	0.48	7	0.68	75	0.84
1.3	12	0.60	9	-	9	0.57	97.5	-
1.5	13.5	0.48	10.5	0.4	10.5	-	112.5	0.49
1.7	15	0.40	12.0	-	12.0	0.38	127.5	-
2.0	18	0.27	14	0.29	14	0.2	150.0	0.28
2.5	22.5	0.12	17.5	0.18	17.5	0.07	187.5	0.16
3.0	27	0.0	21	0.12	21	0	225.0	0.09
4.0	36	-	28	0	28	-	300.0	0.0

Note #) : quoted from the report prepared by P2AT, Madiun.

Table 4.5 Generated Flood Hydrograph

T_p = 10,0
Q_p = 790 (June 9, 1977)

Base flow = 230 (m^3/sec)
V = (790 - 230) = 560 (T_p = 10.0 , X = 1.0)
Y = 0.68 (T_p = 10.0 , X = 1.0)

Thus $V_T = T_p \cdot V/Y$ = 8,235

X	t	Y	$V = \frac{V_T}{T_p} \cdot Y$	Q
0	0	0	0	230
0,3	3	0,34	280	510
0,5	5	0,45	371	601
0,7	7	0,55	453	683
1,0	10	0,68	560	790
1,3	13	0,57	469	699
1,7	17	0,38	313	543
2,0	20	0,20	165	395
2,5	25	0,07	58	288

Table 4.6 Annual Maximum Water Level and Discharge

(A. Yani)

Date		Water Level (m)	Discharge (m3/sec)
1)	Jan 6, 1963	8.30	750
1)	Mar 3, 1964	8.60	620
1)	Apr 9, 1965	8.60	620
1)	Mar 16, 1966	8.85	800
1)	Jan 26, 1967	8.50	570
1)	Mar 26, 1968	8.80	750
1)	Apr 3, 1969	8.95	880
1)	Feb 12, 1970	8.45	560
1)	Mar 26, 1971	8.60	620
1)	Mar 29, 1972	8.80	750
1)	Mar 28, 1973	8.70	680
1)	May 6, 1974	8.90	850
1)	Apr 19, 1975	9.30	1,200
2)	Dec 1, 1976	8.80	700
2)	Jan 9, 1977	9.00	790
3)	Jun 30, 1978	9.50	1,200
3)	May 5, 1979	9.25	1,010

Note 1) : Rating curve established by OTCA and LPMA was used.

2) : Rating curve established by PBS was used.

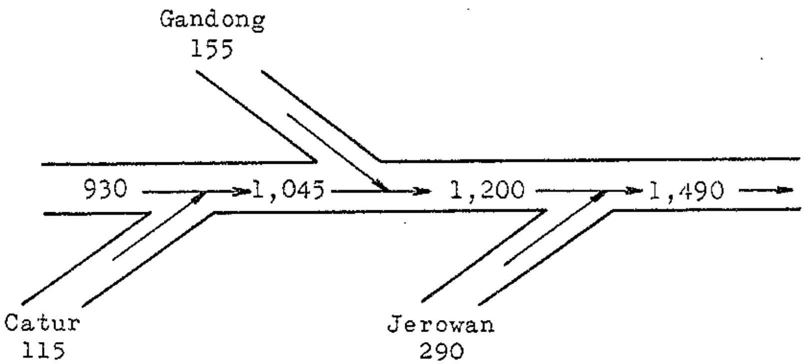
3) : Estimated by hydraulic calculation.

Table 4.7 Allocated Discharge

	Probable flood at A. Yani (m^3/sec)	Specific discharge ($m^3/sec/km^2$)	Local inflow Catur (m^3/sec)	Gandong & others (m^3/sec)	Jerowan (m^3/sec)
Catchment area (Km^2)	-	-	180	240	453
Return period					
17	1,200	0.64	115	155	290
10	1,110	0.59	105	140	270
5	969	0.52	95	125	235
2	757	0.40	70	95	180

Schematic Diagram

(A. Yani 1,200 m^3/sec)



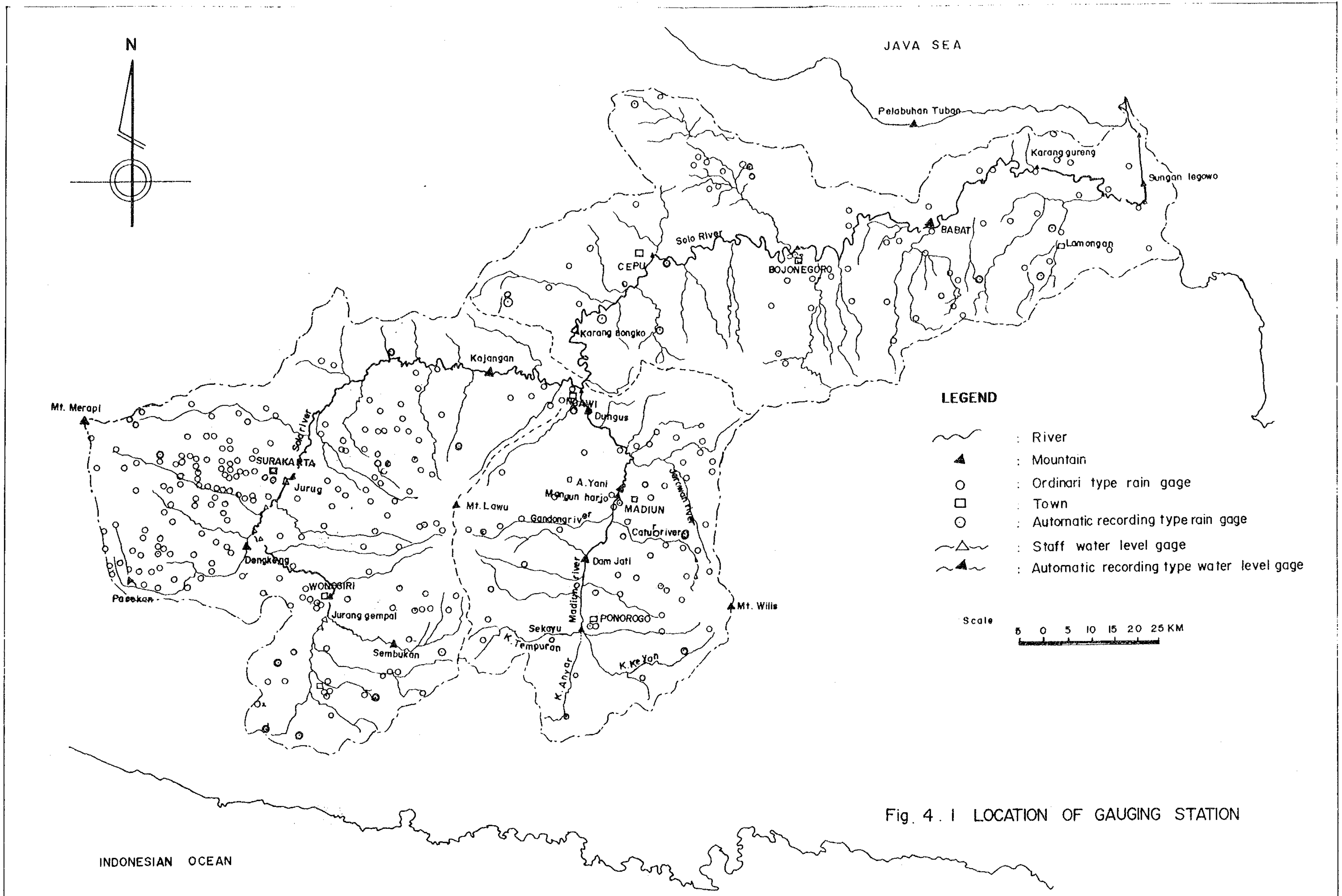
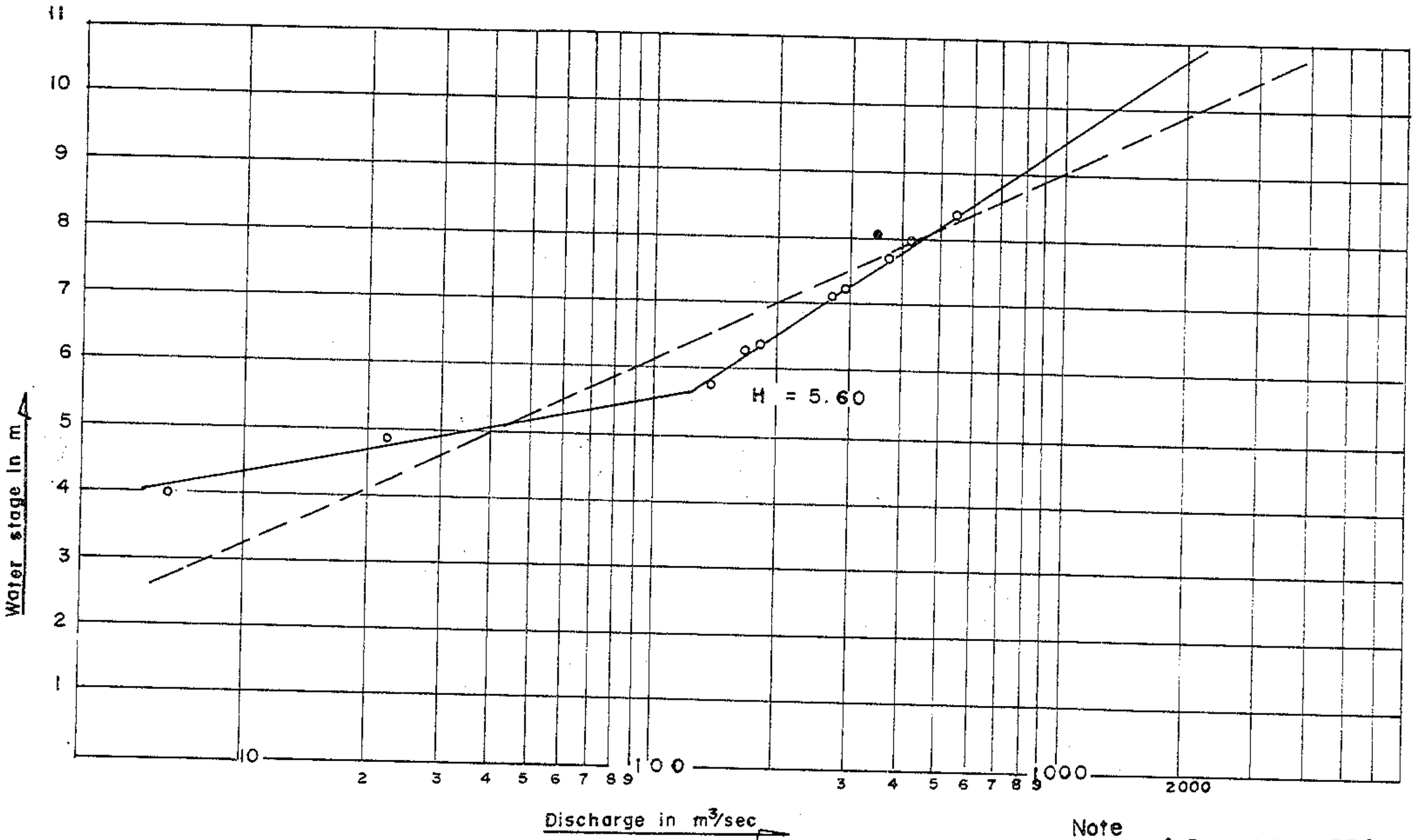
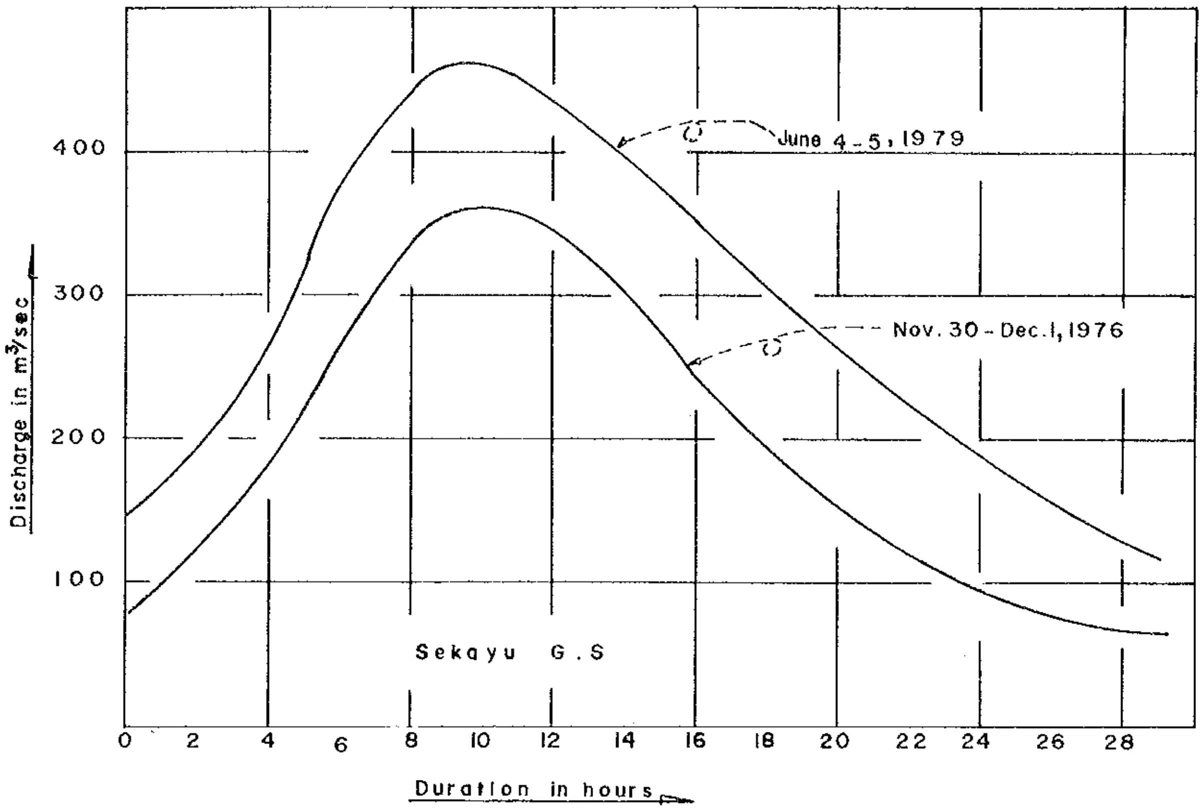


Fig. 4. I LOCATION OF GAUGING STATION



Note
o : Recorded by PBS
• : " LPMA, OTCA
– – – – : Rating curve for before 1975
——— : " " often 1976

Fig. 4.2 DISCHARGE RATING CURVE



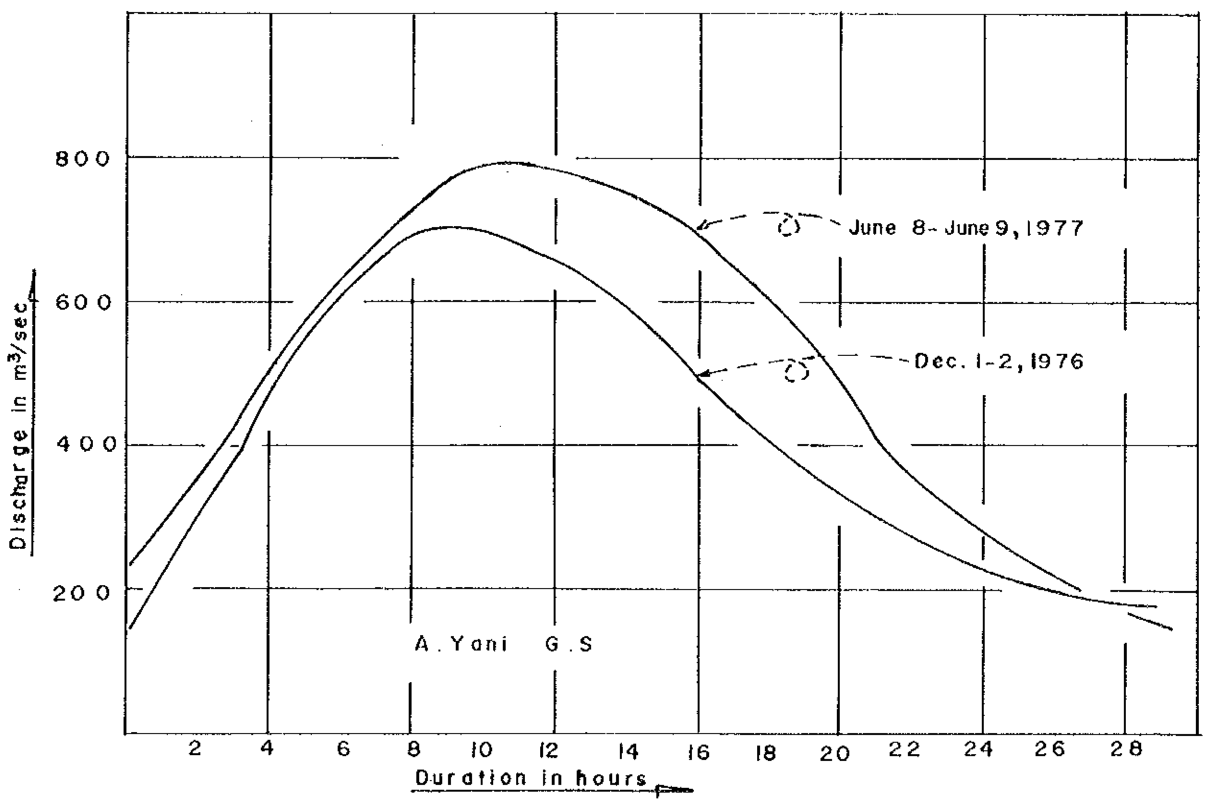


Fig. 4.3 TYPICAL FLOOD HYDROGRAPH

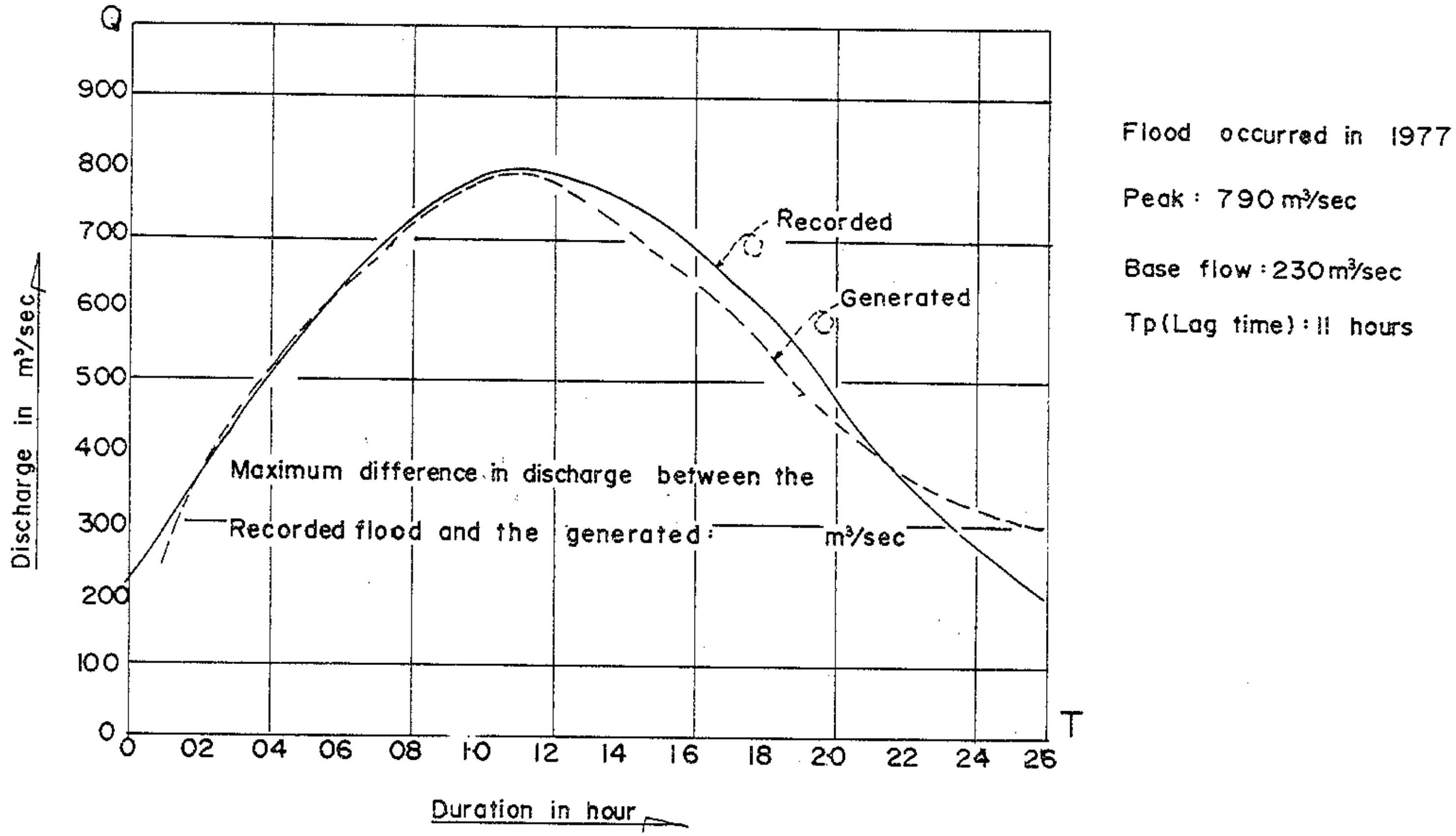


Fig.4.4 GENERATED HYDROGRAPH

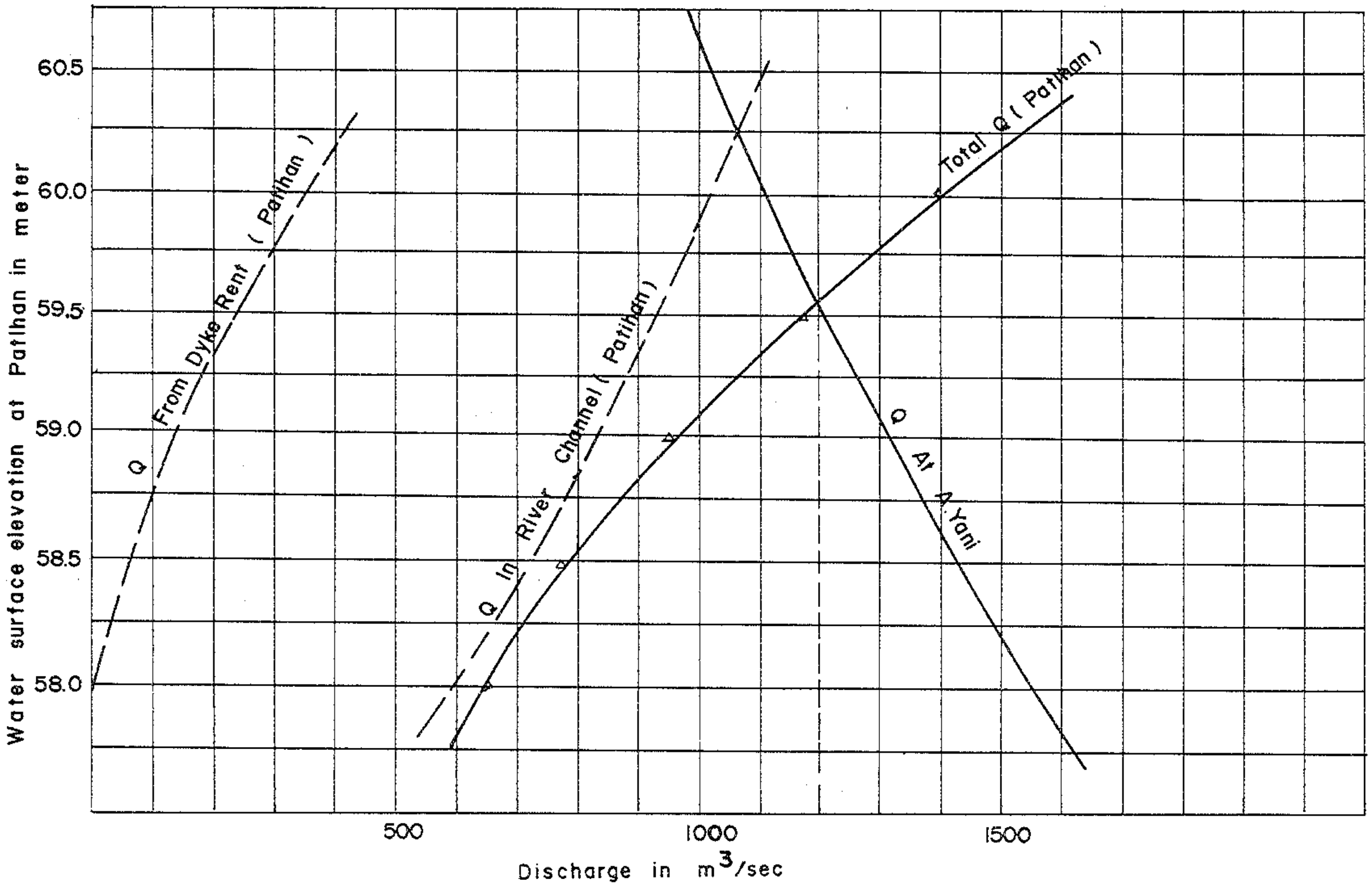


Fig. 4.5 ESTIMATION OF PEAK DISCHARGE IN 1978

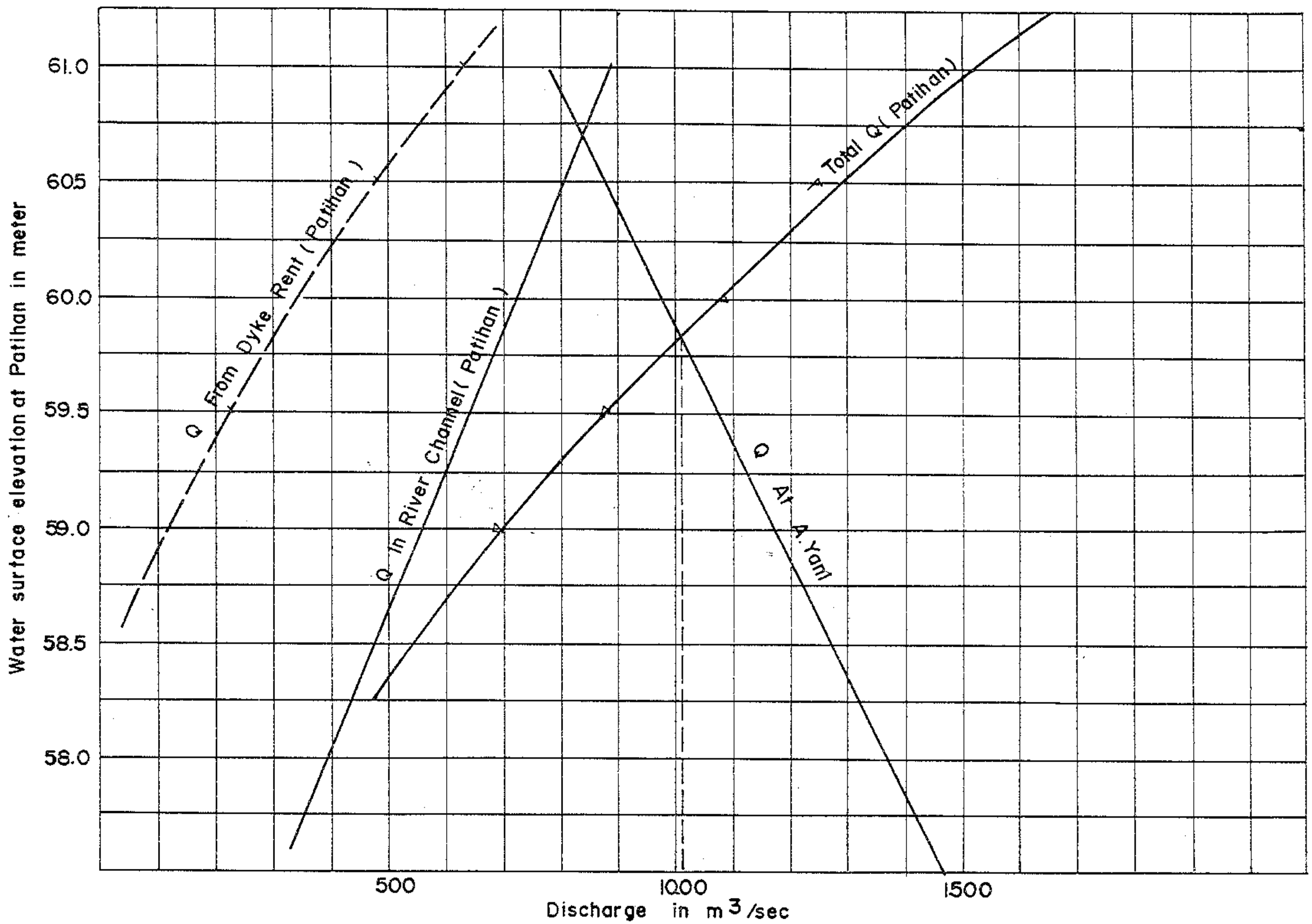


Fig. 4.6 ESTIMATION OF PEAK DISCHARGE IN 1979

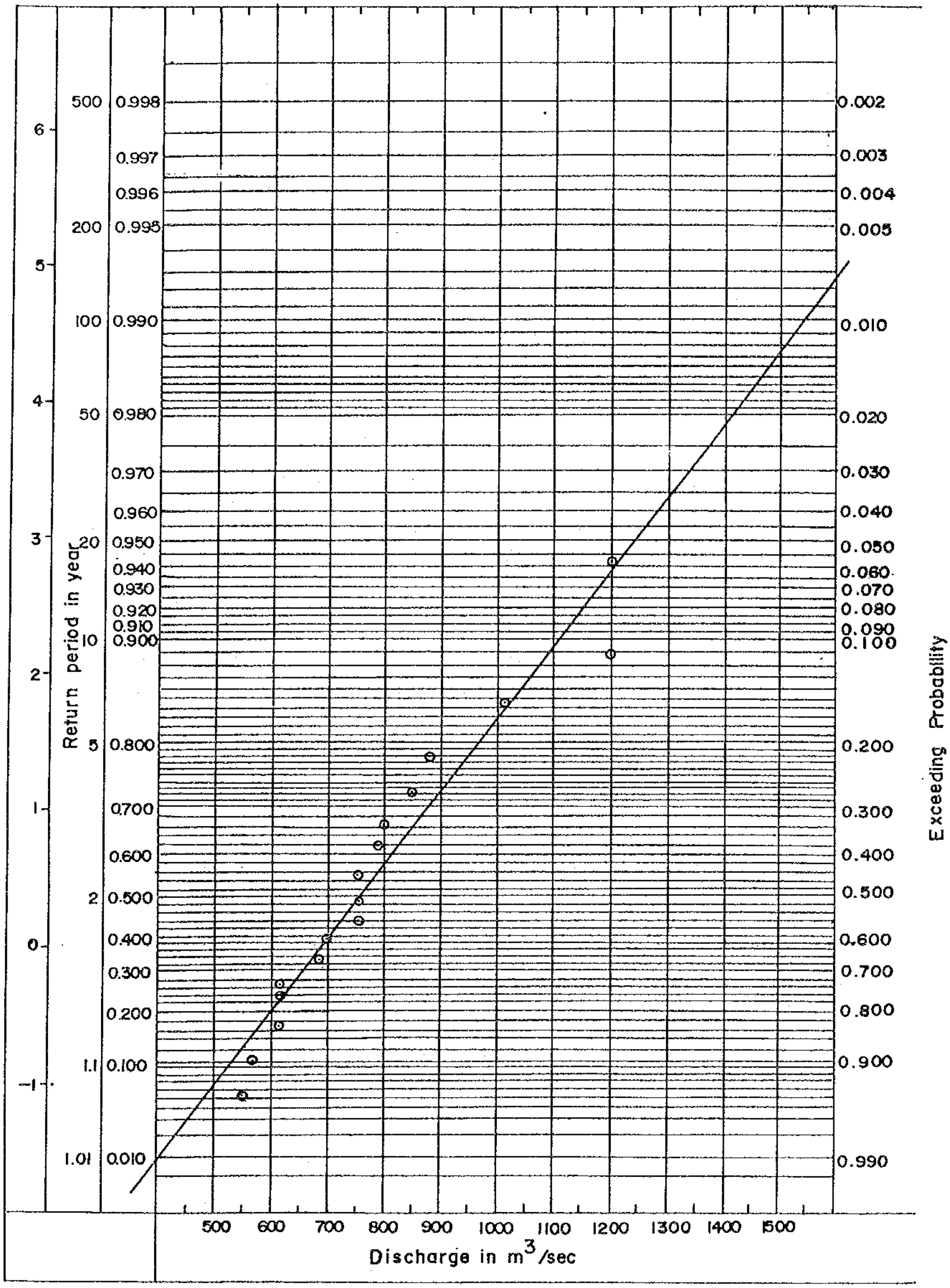


Fig . 4 . 7 PROBABLE PEAK DISCHARGE

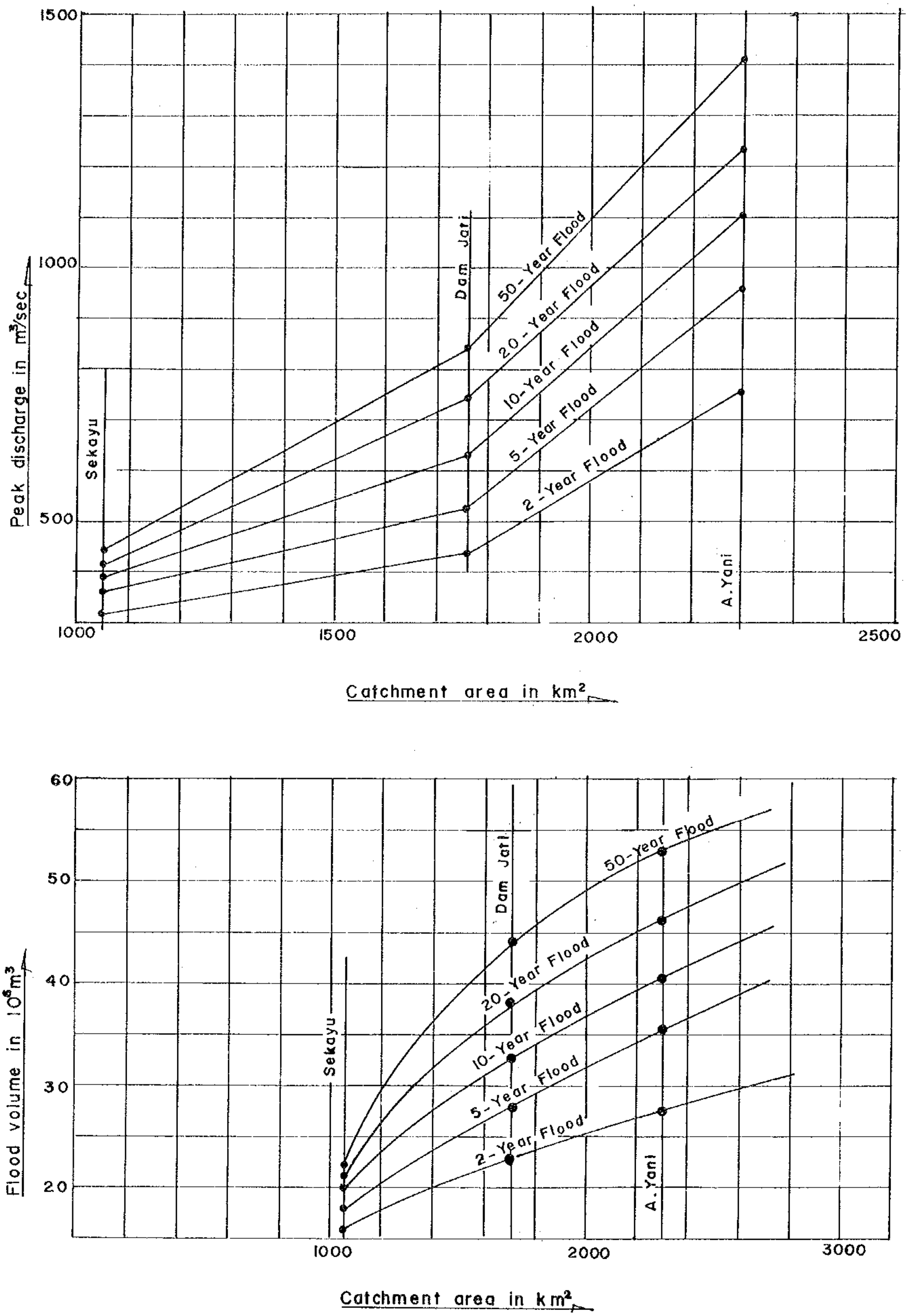
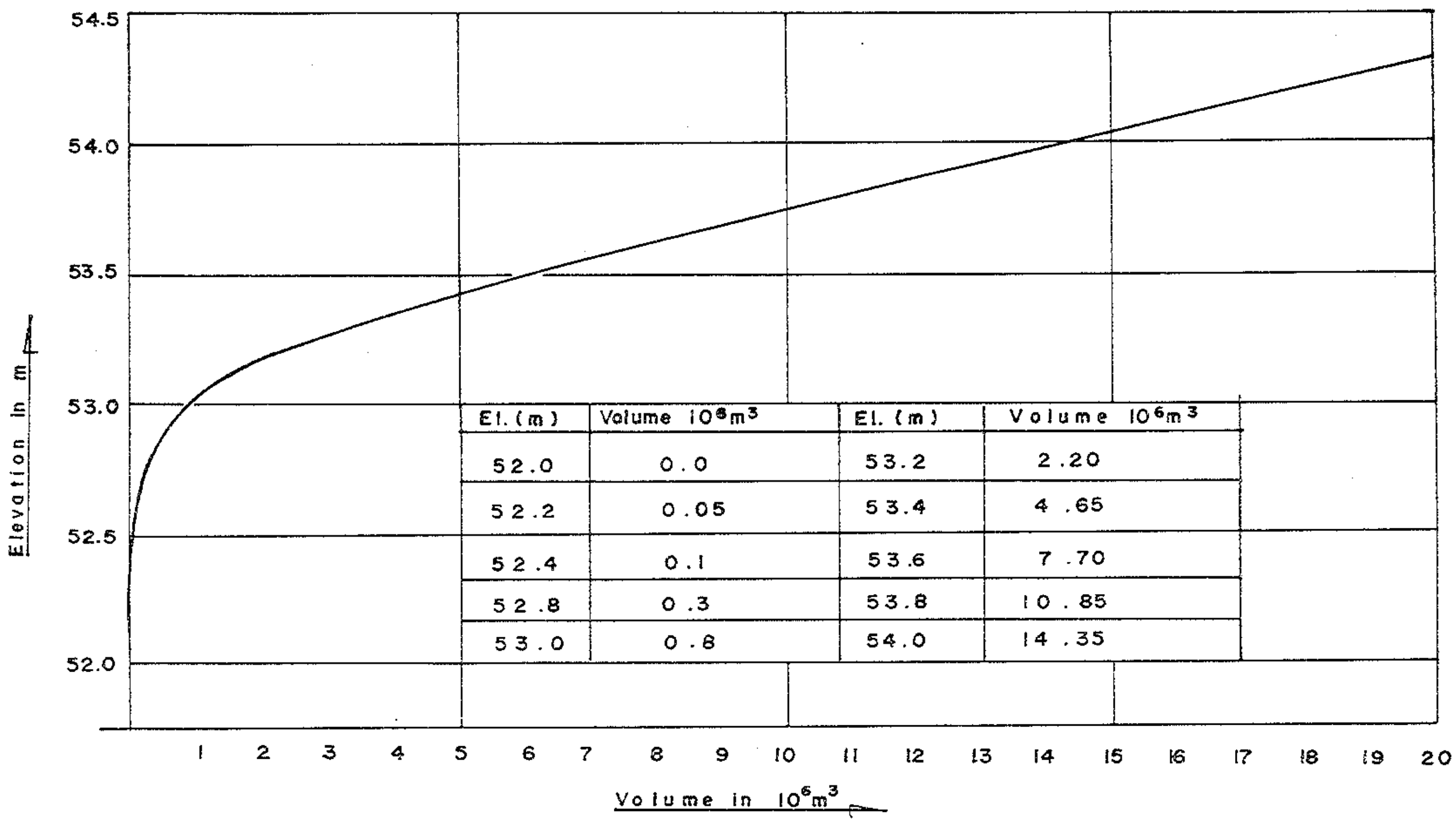
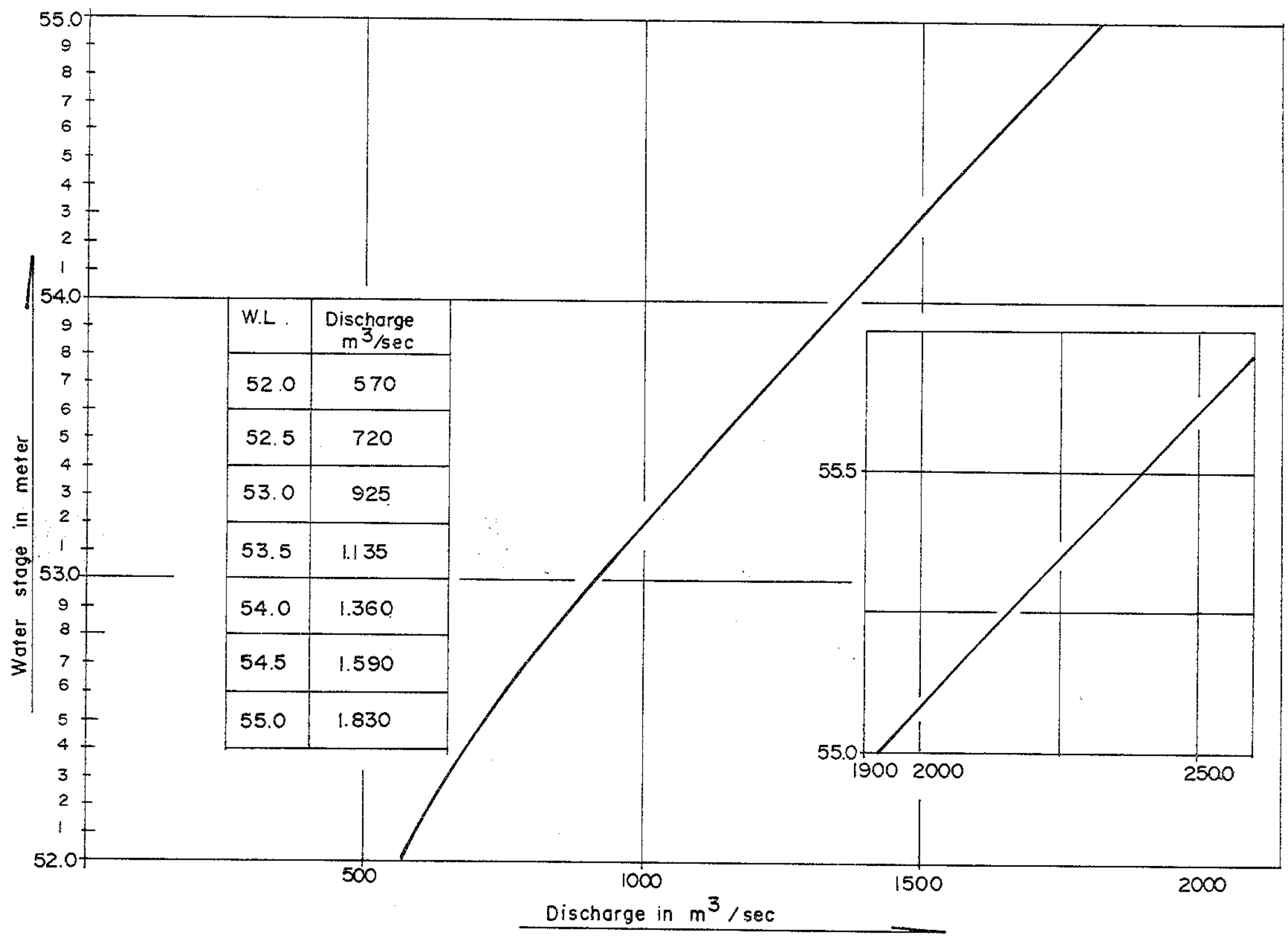


Fig.4.8 CATCHMENT AREA, PEAK DISCHARGE AND FLOOD VOLUME



El. (m)	Volume $10^6 m^3$	El. (m)	Volume $10^6 m^3$
52.0	0.0	53.2	2.20
52.2	0.05	53.4	4.65
52.4	0.1	53.6	7.70
52.8	0.3	53.8	10.85
53.0	0.8	54.0	14.35

Fig.4.9 STORAGE VOLUME CURVE OF JEROWAN INUNDATION AREA



W.L.	Discharge m³/sec
52.0	570
52.5	720
53.0	925
53.5	1.135
54.0	1.360
54.5	1.590
55.0	1.830

Fig.4.10 WATER STAGE – DISCHARGE RATING CURVE (Jerowan inundation area)

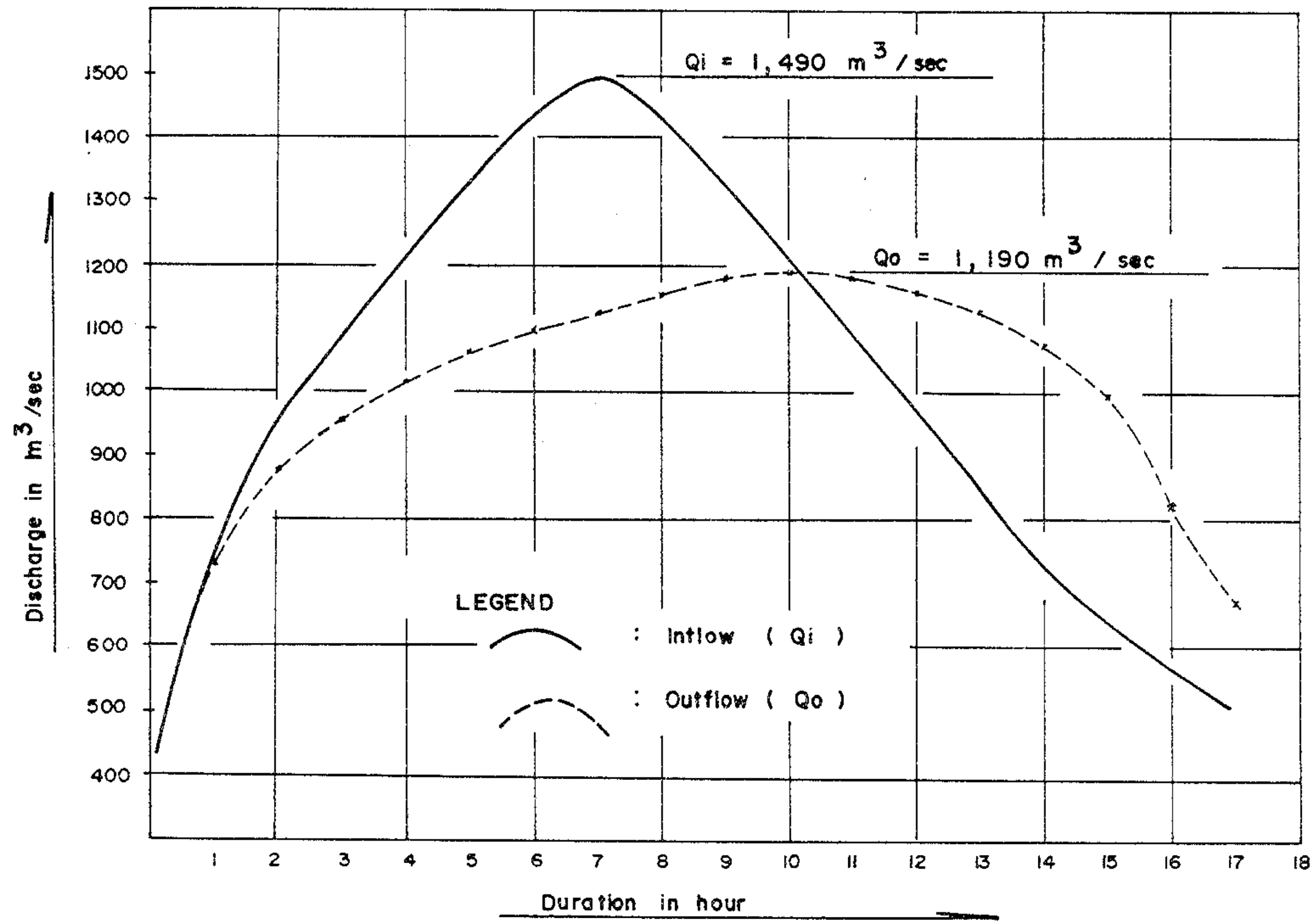


Fig.4.11 INFLOW AND OUTFLOW FROM JEROWAN INUNDATION AREA

(17 — Year return period)

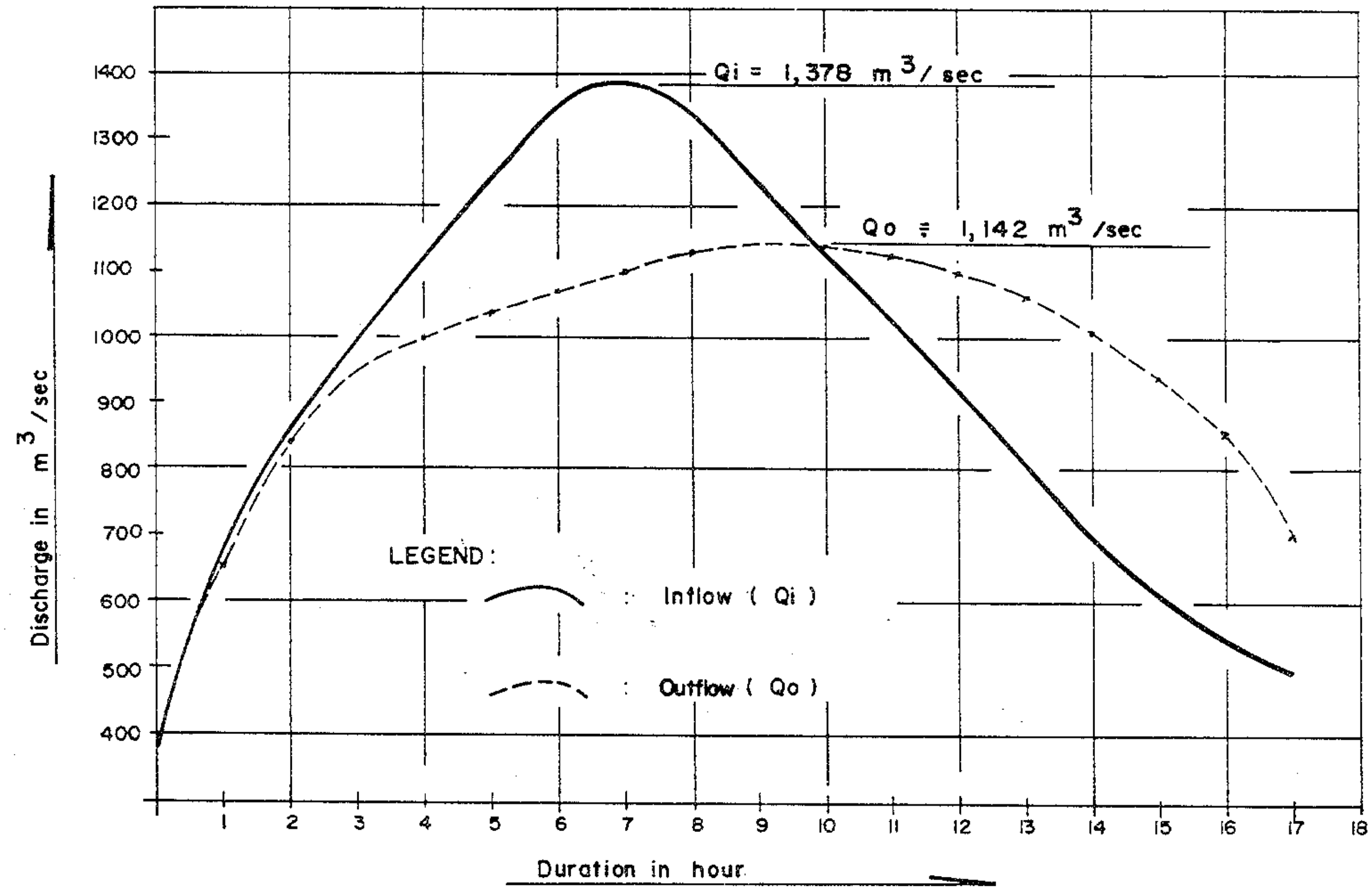


Fig .4.12 INFLOW AND OUTFOW FROM JEROWAN INUNDATION AREA

(10 — Year return period)

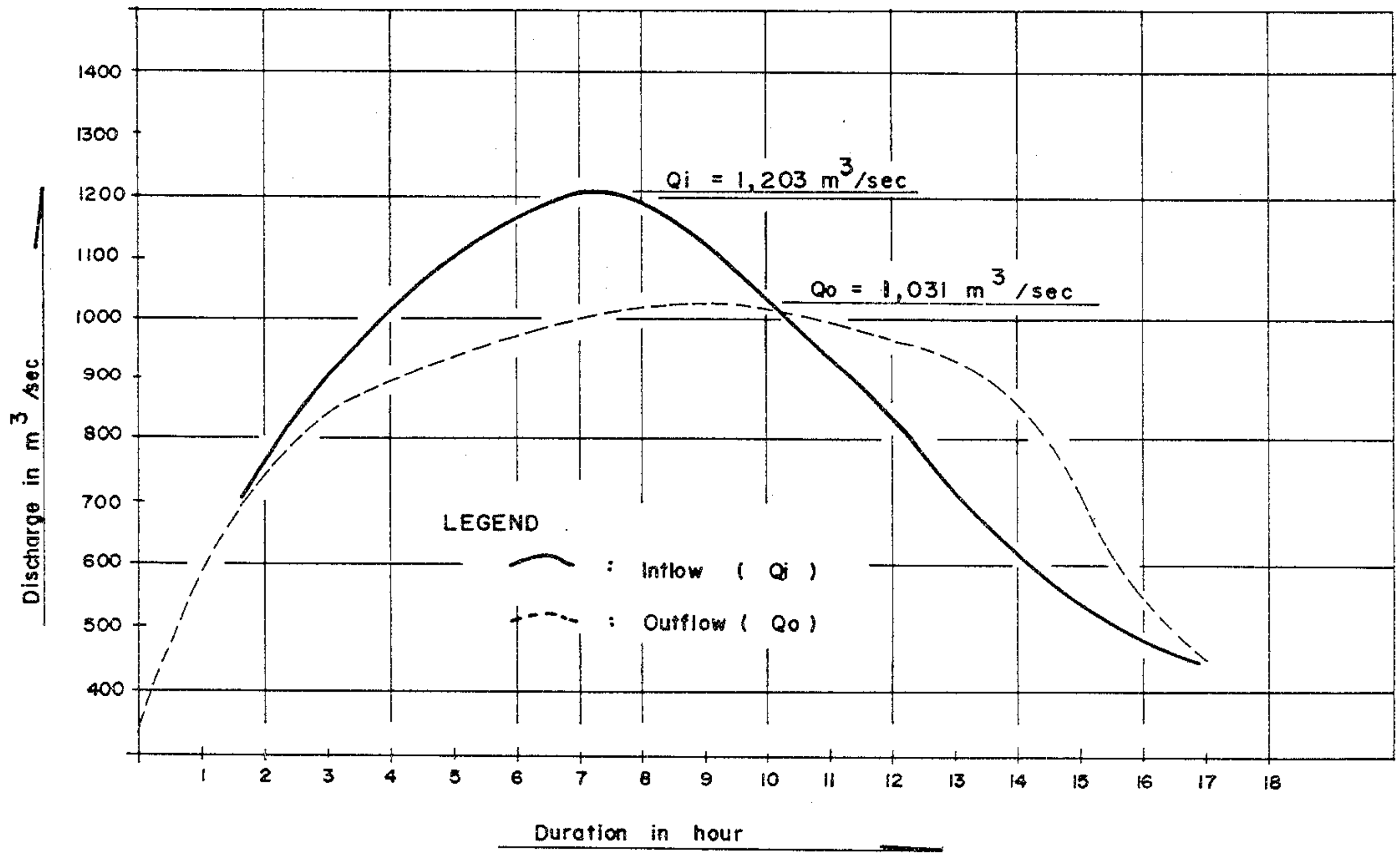


Fig.4.13 INFLOW AND OUTFLOW FROM JEROWAN INUNDATION AREA

(5 — Year return period)

5. TECHNICAL ASSESSMENT

5.1 Effect to the Downstream Area

5.1.1 Increase of discharge

The discharge in the downstream reach was studied by the non-uniform flow method. The increase of the water level in the downstream reach due to the river improvement in the project area was estimated around 20 centimeters on an average for the design discharge. No significant increase in the inundation area could be identified because the increase in water level was too small comparing the accuracy of the available topographic map.

The effect was again examined from the view point of discharge. The storage function of the downstream area was developed applying the inflow and outflow of the area. The estimated discharge from the Jerowan inundation area were assumed as the inflow to the downstream area. The corresponding flood records at the Dungus were assumed as the outflow from the downstream area.

The recorded lag time were distributed in the wide range, from 7 to 24 hours. The recorded lag time were considered to be affected by the local inflow and the back water from the Bengawan Solo river. And the actual lag time was estimated 7 to 10 hours if the noises mentioned above were eliminated.

On the other hand the estimated mean flow velocity was 0.62 m/sec for the design discharge. Accordingly the travelling velocity of the flood wave was estimated 1.03 m/sec by the Kleiz-Seddon's method. And the lag time of 8 hours was obtained because the distance between the Jerowan confluence and Dungus is about 30 km.

In this respect the lag time of 8 hours was adopted in this study. And the storage function was developed as shown in Fig. 5.1.

G and F for the storage function method were estimated working out the following equations;

$G = S/Dt + q/2$

$F = S/Dt - q/2$

Where S : Storage volume

Dt : One hour (3,600 second)

q : Outflow

Assuming the base flow of 300 m^3/sec, the relation between q, G and F was estimated and shown in Fig. 5.2.

The design discharge was applied to the storage function and the outflow was generated. The maximum discharge of 975 m^3/sec was obtained for the design discharge as summarized in Table 5.1. According to the recorded flood, the discharge at the Dungus gaging station of around 900 m^3/sec corresponds to the discharge of 1,200 m^3/sec at the A. Yani gaging station. Thus the increasure of 75 m^3/sec was estimated as the effect of the improvement by the difference between 975 and 900 m^3/sec.

The said increase of discharge will bring the increase of water level of 50 cm according to the rating curve. This figure coincide with the estimated increase in water level of around 20 cm in the inundation area.

The increasures mentioned above were considered to be within the range of usual fluctuation. Because the rating curve has rather wide range of errors brought by the various hydrologic fluctuation such as the local inflow, the back water from the Solo river and so on.

And this urgent river improvement plan was judged to be harmless to the downstream area and also to the Bengawan Solo river.

5.1.2 Conceivable measure

The river channel capacity in the downstream reach is reduced extremely due to the small riverbed slope and meandering channel. The meandering alignment was considered to be improved by constructing shortcut channel in this area.

The lag time is to be shorten and the stored volume in Fig. 5.1 is to be decreased. Thus the inundation will be mitigated. On the other hand the curves for G and F in Fig. 5.2 become flat. Consequently the horizontal distance between G and F is to be enlarged. Thus the hydrograph at Dungus will be modified to have shorter duration and higher peak.

Fig 7.3b in the Main Report shows the heavy meandering channel between the section BM41 and HI655. The shortcut in this portion was considered to be effective to mitigate the influence of the urgent river improvement plan. The discharge from the improved channel will be flushed out in the upstream area from section BM41. Said shortcut will contribute to reduce the water level therein.

5.2 Riverbed Evolution

5.2.1 Present riverbed

The riverbed of the Madiun was considered to have been aggradated considerably by deposit of sediment load according to the river cross sectional data at the old Mangunharjo bridge site. But aggradating has been retarded and no significant riverbed evolution has occurred since 1975 at the bridge site and the A. Yani gaging station site.

The PBS planned to restrain the aggradating condition mentioned above and have constructed the check dams in the tributaries since 1975. The constructed check dams were considered to have functioned as expected so far because the aggradating tendency have been retarded.

The apparent riverbed stability was confirmed by theoretical approach. The formula for sediment runoff established by Sato, Kikkawa and Ashida was applied using the estimated coefficients and variables. Said formula is expressed as follows;

$$q_B = f \cdot F(T/Tc)(T/c)^{1.5}/(S/c - 1)g$$

Where
- q_B : Sediment load per unit width in m^3/sec
- f : 0.623 if the coefficient of roughness is larger than 0.025
- Tc : Critical tractive force
- T : Tractive force
- C : Density of water (= 1.0)
- S : Density of bed material (= 2.6)

The value of F(T/Tc) was assumed to be unity as is usual. Since T is expressed by

$$T = C \cdot g \cdot H \cdot Ie$$

following formula was introduced;

$$Q_B = 1.219\ H^{1.5}\ Ie^{1.5}\ B$$

Where Q_B : Sediment load at a section

H : Average water depth

Ie : Energy gradient

B : Channel width

Q_B for the case of the flood with two years reccurrence interval were estimated for each section. In this estimation H, Ie and B were obtained from the results of the non-uniform flow calculation. The estimated results were plotted and shown in Fig. 5.3. As can be seen in the figure, the Q_B for the present river channel is in the range of 18×10^{-3} and 10×10^{-3} m^3/sec through out the river course. In this manner the stability of the riverbed was confirmed.

5.2.2 Effect of shortcut

In the urgent river improvement plan, shortcut works were proposed at the meandering portions. The present riverbed is stable as described before. However shortcut work make the riverbed slope steep and the tractive force in the upper reach will be enlarged. Accordingly the degradation of riverbed will be brought subsequent to the implementation of the plan. The degradation was estimated as follows;

The original channel length of the proposed shortcut portion is 4,593 m wheras the shortcut channel length is 2,930 m. On the other hand, the elevation of the upstream end is 55.2 m and the downstream end is 53.0 m in this portion. The original riverbed slope is 4.79×10^{-4}. Accordingly the riverbed is expected to become stable when the elevation of the upstream end reduced to 54.4 m. In this manner the degradation of 0.8 m was predicted for the upstream end of the shortcut portion.

On the other hand, by rearranging the bedload equation by Peter-Muller-Meyer to the following form

$$D = 5.26 \; . \; I \; . \; H \; . \; K^{1.5}$$

The minimum transportable or armoring size material can be computed. Where D, I, H are armoring size, channel slope and mean channel depth respectively and K is expressed

$$K = n^{1.5}/d^{0.25}$$

where n is the coefficient of roughness and d is the 90 percent passing particle diameter. Then the depth of degradation is obtained by equation

$$Yd = Ya\left(\frac{1}{P} - 1\right)$$

Where	Yd :	Depth of degradation
	Ya :	Thickness of armoring layer
	P :	Decimal percentage of material larger than the armoring size

The depth, Ya, necessary to armor the bed varies with the size particle needed to form the armoring layer. A rough guide of 3 armoring particle diameters or 0.5 foot, whichever smaller, has generally been used as the depth of Ya.

The depth of the degradation was estimated for the flood of two years return period for the present river channel. D of 10.22 mm and Yd of 1,196 mm were obtained at just upstream site of the shortcut. And it is considered that the bed material equivalent to obtained Yd, 1,196 mm, are supplied to the site from the upstream reach because the present riverbed is stable and no degradation is observed.

D and Yd for the case of with the shortcut channel were predicted by the same method. The respective obtained figures for D and Yd were 17.82 mm and 2,620 mm. And the difference of 1,424 mm between 2,620 and 1,196 was considered to be the actual depth of degradation.

As the conclusion, 1.2 meters, the approximate mean of estimated degradations, was adopted for the maximum depth of degradation for the flood with two years return period.

According to the examined results, the influence of the shortcut was diminished around 2,000 meters upstream from the upper end of shortcut. And no influence was traced in the downstream reach from shortcut channel. However the shortcut channel may be scored to some extent accompanying the degradation in the upstream reach. Thus the degradation was assumed in proportion to the distance from the upper end of the shortcut channel in the reach of 2,500 meters in upstream reach and 1,500 meters in the downstream reach.

The bedload transportation, Q_B, were estimated for the case just after the shortcut channel was constructed and the case after the assumed degradation occurred. The estimated Q_B were plotted and shown in Fig. 5.3. The maximum Q_B of 30×10^{-3} m^3/sec was obtained at the just upper end of the shortcut channel and the unstable situation of the riverbed is explicit. Whereas most of Q_B for the degraded channel were within the narrow range between 15×10^{-3} and 20×10^{-3} m^3/sec. And the riverbed was expected to recover its stability.

5.2.3 Monitoring system

The riverbed evolution was predicted as described in the former section. However, the available data were limited and the decisive theory is not developed yet. And it is recommendable to establish the monitoring system for the riverbed evolution and carry out the following works;

(1) River cross sectional survey once every year in the end of flood season with interval of 500 meters from the site HI96 for 5 km upstream and 2 km downstream.

(2) Sampling, particle size analysis and property test of riverbed material at each site described above. Two samples in both high and low water channels.

(3) Trace of the maximum degradation during the flooding period. Coloured sand or mixture of sand and the powder of lime stone are to be filled in the bore holes with diameter of 50 mm before a flood. After the flood the exactly same points were to be bored with diameter of 100 mm. The coloured sand in the boring core indicate the maximum degradation.

Table 5.1 Estimation of Outflow

t	It - 300	Ft	Gt	qt
0	0	0	0	0
1	450	220	225	20
2	650	750	770	50
3	790	1,350	1,470	90
4	920	2,000	2,205	145
5	1,020	2,670	2,970	210
6	1,130	3,700	4,045	320
7	1,190 (*)	4,400	4,860	405
8	1,130	5,050	5,560	490
9	1,020	5,550	6,125	560
10	905	5,950	6,512	615
11	785	6,200	6,795	655
12	650	6,350	6,918	670
13	540	6,400	6,945	675 (*)
14	430	6,300	6,830	670
15	330	5,800	6,300	590
16	270	5,300	5,800	520
17	200	4,800	5,300	460

Note:

t : Time

It: Inflow

Ft: S/Dt - 1/2 q

Gt: S/Dt + 1/2 q

qt: Outflow

Base flow: 300 m^3/sec

(*): Peak

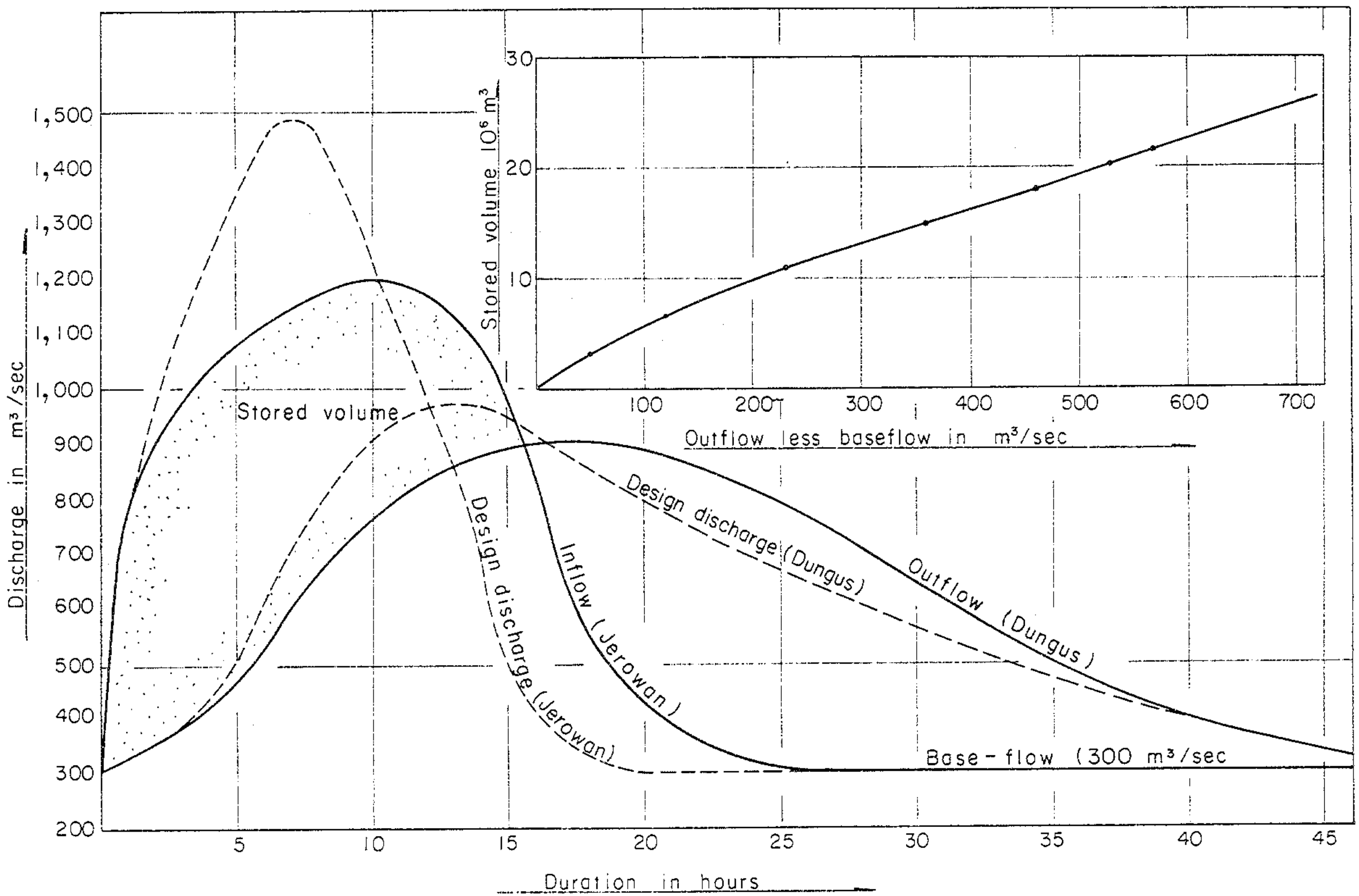


Fig. 5.1 INFLOW, OUTFLOW AND STORAGE FUNCTION

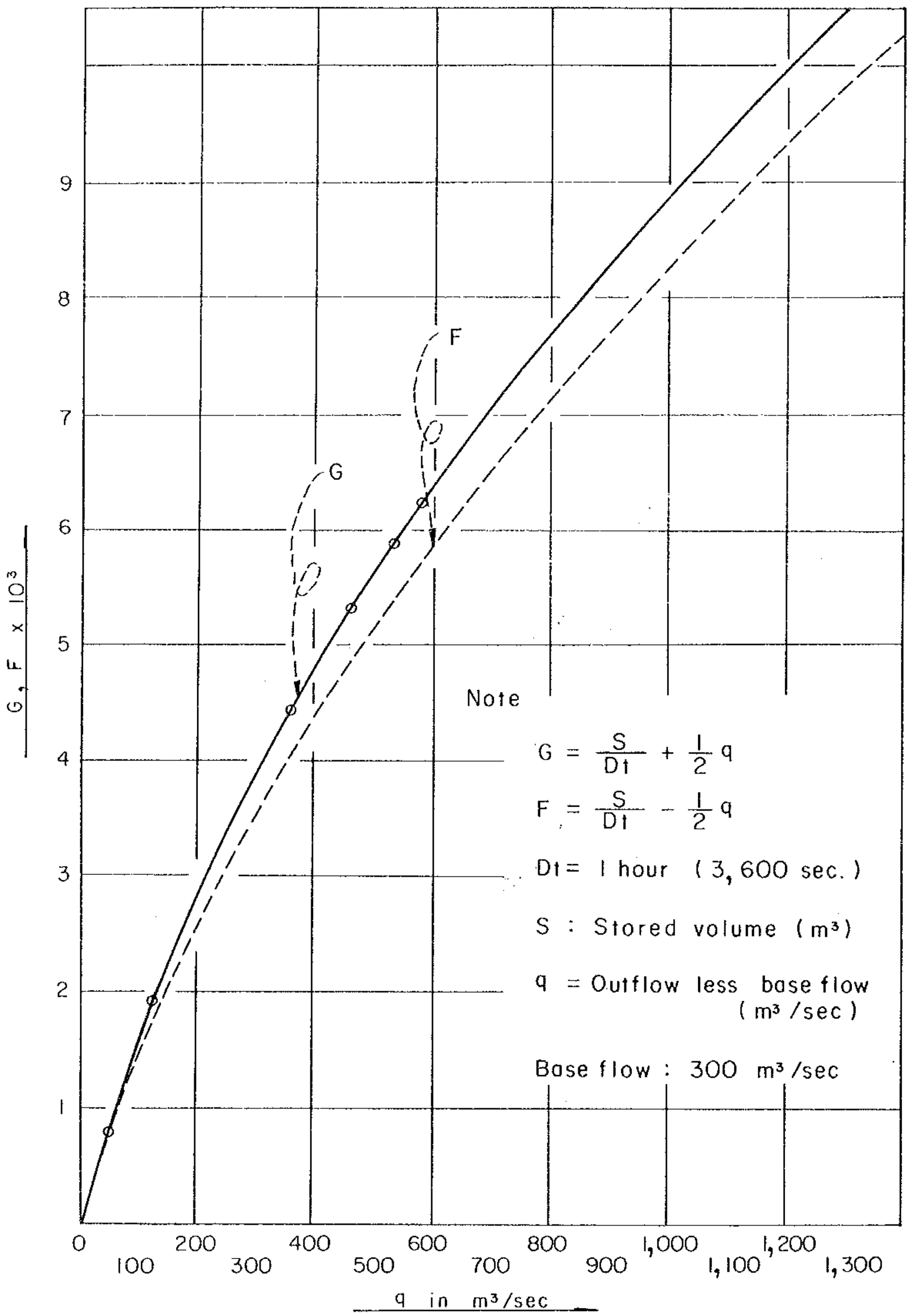


Fig. 5. 2 q , G AND F

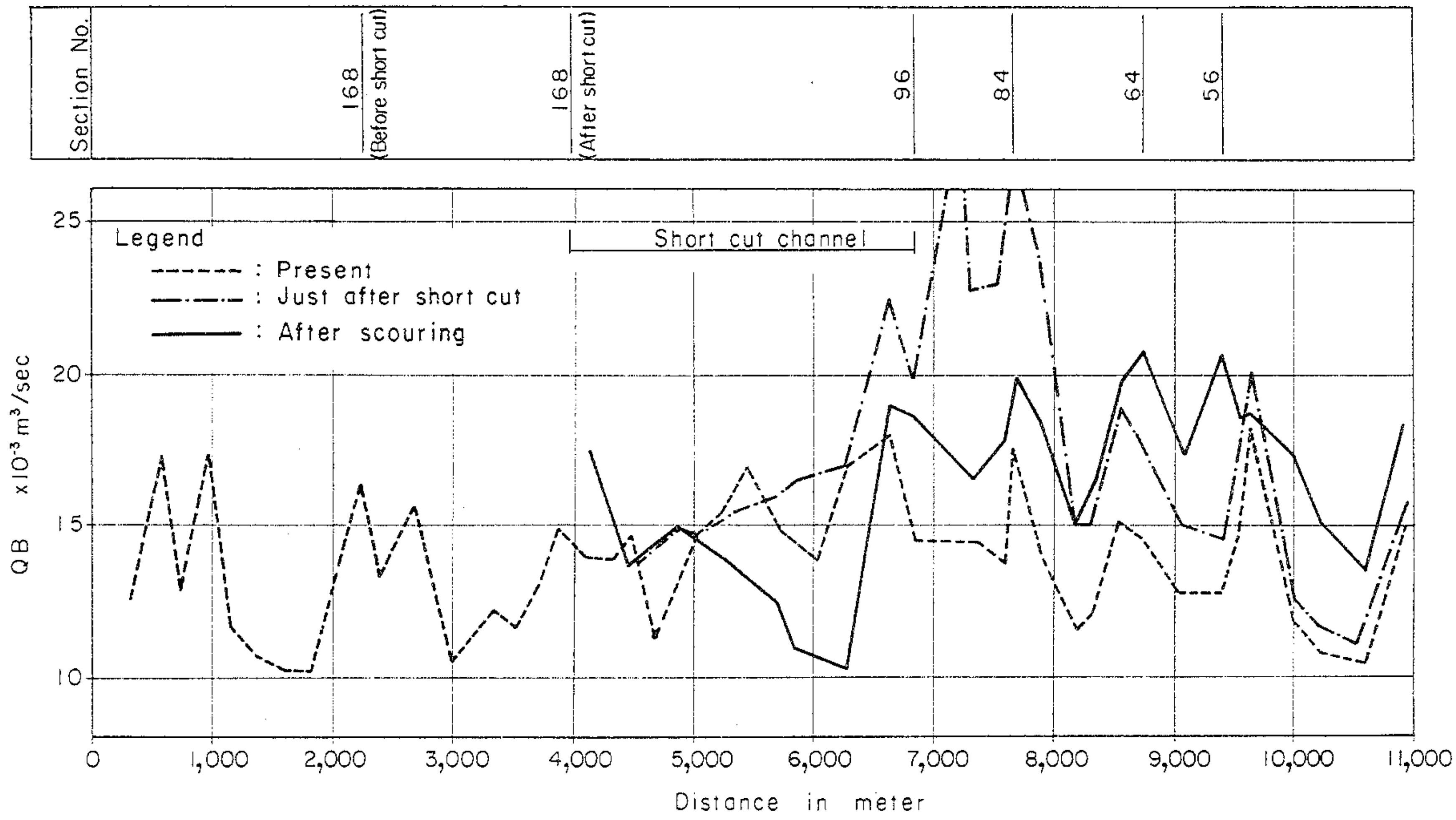


Fig. 5. 3 ESTIMATED SEDIMENT TRANSPORT

6. CONSTRUCTION PLAN

6.1 General

The construction works for the Madiun River Urgent Improvement Project comprise preparatory works, survey and investigations, compensation, embankment of levee, construction of shortcut, revetment, new bridges and gates, modification works and treatment of spoil bank.

All of the works are scheduled to be completed during four years including 1,5 years of detailed design and preparation of tender documents.

The implementation of the construction works for the Project is to be undertaken by the Bengawan Solo Project office of the Directorate of Rivers of Directorate General of Water Resources Development (DGWRD).

The construction works will be executed mechanically using mainly the construction equipment to be procured by the Government.

Preparatory works will be executed in 1982/1983 fiscal year under a separate local contract basis.

Main civil works, new bridges and gates, modification works and treatment of spoil bank will be executed from 1983/1984 to 1984/1985 fiscal year under a local contract basis. The contractors will be selected through the local selective competative bidding.

The contractors will be furnished with the construction equipment, spare parts, and steel materials for metal works, by the Government. The project operation is assisted by selected consultant and minimum guidance personnel specialized in engineering and technical field operation.

6.2 Construction Work Quantity

The work quantities estimated based on the results of investigation are summarized as follows;

(i) Preparatory works

(a)	Access road	:	20 km
(b)	Site office	:	400 m^2
(c)	Repair shop and motor pool	:	1.800 m^2
(d)	Store house	:	400 m^2
(e)	Temporary store house	:	2.000 m^2

(ii) Survey and investigations

(a)	Topo and land survey	:	L.S.
(b)	River survey	:	L.S.
(c)	Route survey	:	L.S.
(d)	Soil test	:	L.S.
(e)	Concrete test	:	L.S.

(iii) Compensation

(a)	Land to be purchased	:	88 ha
(b)	Land to be hired	:	93 ha
(c)	House	:	454 P.C.

(iv) Embankment of levee : 1,308,000 m^3

(v) Shortcut : 525.000 m^3

(vi) Revetment

(a)	Wet masonry	:	44.000 m^2
(b)	Parapet wall	:	4.200 m^3
(c)	Groyne	:	240 m

(vii) New bridges and gates

(a)	Bridges	:	3 sets
(b)	Gates	:	4 sets

(viii) Modification works

(a)	Bridges	:	2 sets
(b)	Irrigation canal	:	L.S.
(c)	Public road	:	L.S.

(ix) Treatment of spoil bank : 210.000 m^3

6.3 Construction Time Schedule

The construction time schedule of the Madiun River Urgent Improvement Project is shown in Fig. 6.1.

The construction of all works are scheduled to be completed during four years including about 1,5 years of detailed design and preparation of tender documents.

The compensation and preparation works will be made during 1982/1983 fiscal year.

The works comprising the embankment of levee, construction of shortcut, wet masonry, parapet wall, groyne, new bridges and gates, modification and treatment of spoil bank will be constructed in two years from 1983/1984 to 1984/1985 fiscal year.

6.4 Basic Conditions for Construction Works

6.4.1 Workable day

The workable days for construction equipment were assumed to be 159 days based on the 28 years average rainfall records in the Project area. The suspended days of work by the rainfall is estimated based on the following criteria.

Rainfall (mm)	Suspended Days
0 - 5	0
5 - 10	0.5
10 - 30	1.0
30 over	2.0

The estimated suspended days by the rainfall were adjusted considering sunday and national holidays. Sunday and national holidays are not included in the workable days.

6.4.2 Workable hour

The works will be executed by one shift under the condition that the workable hour is nine hours. The actual workable hour is assumed at seven hours considering the work efficiency and meal time.

6.4.3 Swell factor for earth material

The swell factor of earth material to be applied for the Project are determined as follows based on the results of investigation and tests.

Kind of Material		Conditions		
		Bank	Loose	Compacted
1.	Clay	1.0	1.30	0.90
2.	Sand	1.0	1.10	0.85
3.	Gravel & Cobble	1.0	1.10	0.95
4.	Rock	1.0	1.55	1.20

6.4.4 Spoil bank for excavated material

The spoil bank yard could not be found in the Project area so far since riparian areas along the stretch of the Madiun River are densely populated zone. Thus the spoil bank yard is selected in the meandering portion of the river where is scheduled to construct the short cut. The spoil materials to be transported to the spoil bank will be commenced after the river diversion work is finished.

6.4.5 Borrow pit

The total required volume is 1,308,000 m^3 comprising the excavation from short cut portion and excavation from farm land where is confined within one km from embankment site.

The volume excavated from short cut is estimated at 525,000 m^3, and remaining volume of 783,000 m^3 is scheduled to be supplied from excavation of farm land. The acreage of the farm land to be required is about 783,000 m^2 with an average depth of one meter.

6.4.6 Aggregate site

For the execution of the works such as pavement, revetment, concrete, etc, the following aggregate sites are selected after result of investigation.

	River Name		Quantity (m3)
1.	K. Catur	more than	100,000
2.	K. Gandong	more than	80,000
3.	K. Jerowan	more than	50,000

Aggregate materials are classified into boulder, gravel and sand at the river side, and are transported to the respective work site.

6.5 Construction Method

6.5.1 Preparatory works

(a) Access road

The access roads are to be used for transportation of soil material for levee embankment. The access roads are scheduled to be provided by enlarging the existing road or constructing newly on the farm land.

The width of the access road is seven meters and thickness of the road embankment is 30 cm at least above the existing ground surface.

Average distance on the access roads is estimated at two km from the borrow pits.

(b) Site office

One site office having floor area of about 400 m^2 is scheduled to be constructed at the job site.

(c) Repair shop and motor pool

One repair shop having floor area of about 1,800 m^2 comprising facilities for washing place, saw mill, inspection pit, store house of spare parts and motor pool is scheduled to be provided at the job site.

(d) Store house

One store house having floor area of about 400 m^2 for the storage of cement, reinforcement bar and etc is scheduled to be provided at the job site.

(e) Temporary store house

One temporary store house having floor area of about 20 m^2 to store small tools and miscellaneous materials is to be constructed at the riverine sites at the interval of about 200 m. After the work is finished, they will be transferred to other sites to use their materials twice.

6.5.2 Survey and investigation

The survey and investigation comprising the works for topographic surveys and laboratory test are scheduled to be carried on every construction year as the routine works.

The topographic surveys consist of the survey for land compensation areas, longitudinal and cross sectional surveys to be carried out for 20 km along the river stretch, and route survey for the access road.

The laboratory tests which comprise physical quantity and mechanical property tests for soil and aggregate and fresh concrete tests for concrete.

6.5.3 Compensation

The works for compensation comprise borrowing of the land to be used for stock pile and borrow pit along river stretch and purchasing of the land to be used for short cut and levee embankment as well as the compensation of house located in the proposed short cut and embankment areas.

It is scheduled that these compensation works are carried out during 1982/1983 fiscal year before execution of the physical works.

6.5.4 Embankment of levee

The embankment of levee comprise the embankment to reinforce the existing levee and embankment to provide new levee.

The embankment works will be executed from 1983/1984 to 1984/1985 fiscal year.

Borrow pits

Earth material for levee embankment will be executed from the proposed borrow pits.

Moisture content suitable for levee embankment will be ensured at the borrow pit site in principle. To attain the above purpose, levee embankment work is scheduled to be executed in the dry season of seven months. Even in the dry season, if rainfall of more than 5 mm/hr occurs, excavation work of the earth material should be suspended. In case that earth material excavated at the borrow pit is in too dried up condition, adequate amount of water will be sprinkled to the earth material by the water tanker.

Excavation of earth material for levee embankment will be made by a combination of 1.2 m^3 class dozer shovel, 21 ton class bulldozer with ripper and six ton class dump truck.

Foundation treatment of levee

Surface soil of foundation on which new levee is embanked will be stripped by 11 ton class bulldozer. Depth of stripping of the surface soil will be confined within about ten centimeters.

In case that levee embankment is made to reinforce the existing levee, surface slope of the existing levee will be cut in a shape of step to contact firmly the new levee embankment and the existing levee.

Spreading of embankment material

The embankment material transported from the borrow pit by six ton class dump truck will be directly spread to the area to be embanked.

The embankment material will be spread horizontally using 11 ton class bulldozer. Depth of spreading will be 20 - 30 cm.

Compaction

The spread earth material will be compacted horizontally using 2.5 ton class vibrating roller. In case that width of the area to be compacted in the direction of cross section is less than one meter, the compaction will be made using soil compactor or tamper.

To cope with the settlement of the levee, extra banking will be executed.

Sodding

The side slope of levee will be protected by sod facing. The sodding work will be carried out selecting appropriate season for lawn growing.

6.5.5 Short cut

The short cut to be constructed in the left side comprises Part No.1 and Part No.2. These short cut will be carried out from 1983/1984 to 1984/1985 fiscal year.

Stripping of surface soil

Surface soil of short cut portion will be stripped by 11 ton class bulldozer. Depth of stripping of the surface soil will be confined within about ten centimeter.

Surface soil stripped will be transported by a combination of 1.2 m^3 class dozer shovel and six ton class dump truck to stock pile.

Excavation of short cut

Earth materials to be excavated in the short cut will be transported to the respective levee embankment site.

Excavation of earth materials will be made by a combination of 1.2 m^3 class dozer shovel, 21 ton class bulldozer with ripper and six ton class dump truck.

6.5.6 Revetment

The revetment works comprise the works for the wet masonry, parapet wall and groyne. The revetment works will be executed from 1983/1984 to 1984/1985 fiscal year.

(a) Wet masonry

The construction of wet masonry will be executed in order of coffering and unwatering, foot protection and stone masonry including backfilling.

Coffering and unwatering

Prior to the execution of the wet masonry work, cofferdam will be provided surrounding the foot protection to be executed.

The cofferdam will be constructed by driving the pile in two rows, stretching bamboo panel inside the driven piles and packing sand between the space of two rows of piles. The space between the piles and height of the cofferdam are two meters respectively.

Unwatering will be carried out using submergible pump during the execution of foot protection work. After the foot protection work is finished, all of the cofferdam will be removed.

Foot protection

The river bed will be transformed horizontally to some extent. Some river bed will decrease its altitude by scouring, some will be raised by silting of sediment.

If there are possibility of scouring in the river banks the toe of the bank will be protected with gabion mattress.

Prior to the pile driving, gabion mattress will be prepared at the site where the revetment is installed. Size of the gabion mattress is 0.5 m in depth, 1.5 m in width and 3 m in length.

The wooden piles will be driven through the empty frame of the mattress. The packing work of the boulder into the frame will be performed at installing site. After the boulder is packed inside the frame, upper portion of the frame will be closed.

<u>Wet masonry</u>
Side slope of the levee, where the wet masonry is constructed, will be trimmed to meet the designed slope.

In case that the side slope of the levee is formed by backfilling, surface treatment of the existing levee and succeeding backfilling will be made in the same manner as adopted to the levee embankment.

The boulder with 10 - 20 cm in diameter will be placed on the trimmed surface of the levee. Void between the boulders will be filled with sand and gravel with good quality.

Base concrete and coping concrete will be placed on the boulder foundation.

The side slope of the levee will be protected by the wet masonry to the place where high water level reaches.

Cobble to be used for the wet masonry is 20 - 30 cm in diameter. The cobble will be placed closely on the base concrete and void between the cobble will be fully filled with mortal. Construction joint will be provided. The type of the wet masonry is shown in Fig. 4.14, Main Report.

(b) Parapet wall
The existing parapet wall comprises gravity type wall, pile type wall and buttress type wall. These existing parapet wall are to be reinforced by the following method.

The gravity type wall made by brick and mortal will be reinforced by the reinforced concrete covering all over the existing wall.

The pile type wall made by brick will be replaced by the reinforced concrete.

The buttress type wall made by concrete will be heightened by the reinforced concrete connecting the existing wall.

(c) Groyne

The concrete piles will be driven in the cross sectional direction. Prior to the pile driving, empty frame of the gabion mattress will be set along the existing river bed.

The concrete piles will be driven through the empty frame of the mattress. Top of the piles should be coincided with that driven at the deepest river bed.

Empty frame of the gabion mattress will be set in one layer on the existing river bed. After the boulder is packed inside the frame, top of the piles will be connected by the longitudinal concrete frame.

6.5.7 New bridges and gates

Three bridges and four gates are to be newly constructed.

The construction of new bridges and gates will be executed from 1983/1984 to 1984/1985 fiscal year.

(a) Bridges

The type of the bridges to be newly constructed is the metal girder bridge. The locations of the proposed bridges are Slajen, Patihan and Krandang.

The works of bridge comprise the foundation, substructure and errection of superstructure. The Government is to supply the materials thereof.

The access from the existing road to the river bed will be constructed. Cofferdam constructed by driving the wooden piles in two rows and packing the river bed materials in the space between the two pile rows will be provided surrounding the foundation of the bridge.

After the river water stagnated inside the cofferdam is drained by the submergible pump, the river bed portion will be excavated to the specified elevation.

The steel pipe piles will be driven at the foundation by using the diesel pile hammer. Base concrete will be placed on the boulder foundation after bottom surface is trimmed.

Concrete works for pier portion will be executed following to the base concrete. After all of the concrete works for bridge structure are completed, backfilling will be carried out using the tamper.

The erection of superstructure will be carried out by using the 30 ton class hydraulic crane. The existing bridges will be removed after the completion of new bridges.

(b) Gates

The type of the gate to be newly constructed is the sluice gate.

The works of gate construction comprise the foundation, inlet structure and errection of the sluice gate. The Government is to supply the gate materials.

Prior to the execution of works, the cofferdam constructed by driving the wooden piles in two rows and packing the river bed materials in the space between the pile rows will be provided surrounding the gate site.

After the river water stagnated inside the cofferdam is drained by the submergible pump, the river bed portion and existing levee portion will be excavated to the specified elevation.

Base concrete will be placed on the boulder foundation after the bottom surface is trimmed. Concrete works for inlet structure will be executed following to the base concrete.

After all of the concrete works for gate structure are completed, backfilling will be carried out using the tamper. The errection of the gate will be carried out using the 30 ton class hydraulic crane.

6.5.8 Modification works

The modification works comprise the bridges, irrigation canal and public road. The works of modification will be executed from 1983/1984 to 1984/1985 fiscal year.

(a) Bridges

The Madigondo and Gandong bridges are to be modified.

The Madigondo bridge crossed over the Madiun river is mainly used to transport the sugar cane by using the locomotive. The transportation of sugar cane is scheduled to change from the locomotive to the cargo truck during several months of modification work. The passer will be ensured using the existing bridge even in the during of modification work.

The steel support made by H-steel will be installed between girder of the bridge and the river bed. Four numbers of hydraulic jack will be set on the steel support, and the bridge portion inside the pier will be jacked up carefully until the specified elevation. The pier portion will be heightened by placing of the reinforced concrete on the existing pier after finishing of surface treatment of the pier. After finishing of concrete works, the lifted bridge portion will be jacked down on the reinforced pier.

In case of the Gandong bridge crossed over the tributary, the temporary bridges will be provided during the modification work. The same work will be applied to the modification.

(b) Irrigation canal

The existing irrigation canal located in the short cut portion will be removed to the appropriate place along the short cut. The works of modification comprise the irrigation canal and siphon.

Surface soil of irrigation canal is stripped by 11 ton class bulldozer. Depth of stripping of the surface soil will be confined within about ten centimeter. Excavation of irrigation canal will be made by using the 0.6 m^3 class back hoe. The earth materials excavated will be spread horizontally using the 11 ton class bulldozer. The spread earth materials will be compacted horizontally using 0.5 ton class vibrating roller.

The foundation of siphon to be installed in the short cut portion will be excavated to the specified elevation. Base concrete will be placed on the boulder foundation after bottom surface is trimmed. Concrete works for inlet structure, conduit and outlet structure will be executed following to the base concrete. After all the concrete works are completed, backfilling will be carried out using the tamper.

(c) Public road

The existing public road located in the short cut portion will be removed to the appropriate place along the short cut, and also the access to the new bridges and the bridges to be mofified will be contemplated.

The public road is scheduled to be provided by embanking the soil materials and paving the asphalt macadam. The works of embankment will be applied by a same method mentioned in Paragraph 6.5.4.

The macadam will be set in a layer on the subgrade of embankment and be compacted by the road roller after the void between macadam is filled by the sand and gravel. The surface of the macadam will be paved by the asphalt as a surface cource.

6.6 Treatment of Spoil Bank

After finishing of all works, the spoil bank will be backfilled by the spoil materials piled in the stock pile and removing the existing levee material.

6.7 Possibility of Transferable Construction Equipment

The construction period of other projects were compared to one of the Madiun River Urgent Improvement Project to find the possibility to transfer the construction equipment used in other project to the project.

The on-going project to be contemplated for checking of possibility of transferable equipment is Wonogiri Dam Project.

The Wonogiri Dam Project is being implemented and the primal works are to be completed before 1981. All of construction equipment employed in this project are newly procured by OECF's loan. However, it is scheduled that majority of these equipment are transfered to Wonogiri Irrigation Project.

The Wonogiri Irrigation Project will implemented from 1980 to 1983.

Thus, to transfer the construction equipment from other project is practically impossible.

6.8 Availability of Contractor

In consideration of the work items, work volume and construction period of the Madiun River Urgent Improvement Project, the contractor will be required to have sufficient technical experience and financial capacity.

According to the results of prequalification and tendering for Wonogiri Irrigation Project and Way Rarem Irrigation Project, it is judged that some of contractor have sufficient experience, personnel, equipment and financial capacity to carry out the works of the Project.

6.9 Required Equipment

The kind of the construction equipment and their numbers to be required for the implementation of construction works were estimated based on the construction method mentioned in paragraph 6.5.

The results of the estimation of construction equipment and their numbers are shown in Table 6.1. The equipment use schedule in each field of works are shown in Table 6.2.

The calculation sheet of construction equipment and the hourly production of construction equipment are shown in Table 6.3 and 6.4 respectively.

Table 6.1 Number of Equipment to be Required (1/2)

No.	Name of Equipment	Capacity	Q'ty	Remarks
1	Bulldozer w/Ripper	21 t	9	
2	Bulldozer	11 t	14	
3	Swamp Bulldozer	13 t	2	
4	Back Hoe	0.6 m^3	1	
5	Back Hoe	0.35 m^3	1	
6	Wheel Loader	2 m^3	1	
7	Dozer Shonel	1.2 m^3	14	
8	Dump Truck, 4 x 4	6 t	79	
9	Truck w/Crane	6 t	4	
10	Water Tanker	5 m^3	6	
11	Fuel Tanker	5 m^3	4	
12	Grease Car	6 t	2	
13	Maintenance Car	6 t	1	
14	Service Car	2 t	2	
15	Hydraulic Crane	30 t	2	
16	Hydraulic Crane	20 t	2	
17	Motor Grader	11 t	2	
18	Vibrating Roller	2.5 t	17	
19	Vibrating Roller	0.5 t	12	
20	Road Roller	8/10 t	2	
21	Tamper	80 kg	5	
22	Soil Compactor	90 kg	11	
23	Port Belt Conveyor	7 m	30	
24	Port Concrete Mixer	0.5 m^3	3	
25	Port Concrete Mixer	0.2 m^3	5	
26	Concrete Vibrator	45 mm	20	
27	Port Air Compressor	6 m^3	8	
28	Pick Hammer	1 m^3/min	9	
29	Engine Driven Winch	3 ps	2	
30	Submergible Pump	6 inch	4	

(to be continued)

Table 6.1 Number of Equipment to be Required (2/2)

(Continuation)

No.	Name of Equipment	Capacity	Q'ty	Remarks
31	Submergible Pump	4 inch	10	
32	Submergible Pump	3 inch	10	
33	Engine Driven Welder	200 A	4	
34	Trailer	30 t	1	
35	Generator	175 kVA	2	
36	Generator	60 kVA	3	
37	Generator	15 kVA	2	
38	Diesel Pile Hammer	2.5 t	1	
39	Inspection Car	6 persons	10	
40	Ambulance Car	1 Bed	1	
41	Repair Shop Tool & Crane		L.S.	

Table 6.2 Equipment Use Schedule (1/3)

No.	Work Items	Q'ty	Kind of Equipment to be Used	Cap	Construction Period 82/83	83/84	84/85	Remarks
1.	Access Road	20 km						
			Bulldozer w/Ripper	21 t	1	-	-	
			Bulldozer	11 t	1	-	-	
			Dozer Shovel	1.2 m^3	1	-	-	
			Dump Truck	6 t	6	-	-	
			Motor Grader	11 t	1	-	-	
			Road Roller	8/10 t	1	-	-	
			Vibrating Roller	2.5 t	2	-	-	
2.	Embankment of Levee	1,380,000 m^3						
			Bulldozer w/Ripper	21 t	-	6	6	
			Bulldozer	11 t	-	10	10	
			Dozer Shovel	1.2 m^3	-	10	10	
			Dump Truck	6 t	-	53	53	
			Water Tanker	5 m^3	-	5	5	
			Vibrating Roller	2.5 t	-	17	17	
			Vibrating Roller	0.5 t	-	8	8	
			Soil Compactor	90 kg	-	7	7	
3.	Short Cut	525,000 m^3						
			Bulldozer w/Ripper	21 t	-	3	3	
			Bulldozer	11 t	-	3	3	
			Dozer Shovel	1.2 m^3	-	4	4	
			Dump Truck	6 t	-	23	23	
4.	Wet Masonry	41,000 m^2						
			Bulldozer	11 t	-	1	1	
			Dozer Shovel	1.2 m^3	-	1	1	
			Submergible Pump	6 inch	-	2	2	
			Submergible Pump	4 inch	-	6	6	
			Submergible Pump	3 inch	-	6	6	
			Diesel Generator	60 kVA	-	2	2	
			Engine Driven Winch	3 PS	-	2	2	
			Tamper	80 kg	-	1	1	
5.	Parapet Wall	4,200 m^3						
			Concrete Mixer	0.2 m^3	-	2	2	
			Concrete Vibrator	45 mm	-	4	4	
			Pick Hammer	1 m^3	-	6	6	
			Air Compressor	6 m^3	-	2	2	
			Belt Conveyor	7 m	-	4	4	
6.	Groyne	240 m						
			Diesel Pile Hammer	2.5 t	-	1	-	
			Truck Crane	20 t	-	1	-	
			Truck w/Crane	6 t	-	1	-	

Table 6.2 Equipment Use Schedule (2/3)

No.	Work Items	Q'ty	Kind of Equipment to be Used	Cap	Construction Period			Remarks
					82/83	83/84	84/85	
7.	New Bridges	3 sets						
	(Slajen, Patihan &		Bulldozer w/Ripper	21 t	-	1	1	
	Krandang Bridge)		Bulldozer	11 t	-	2	2	
			Dozer Shovel	1.2 m^3	-	1	1	
			Back Hoe	0.6 m^3	-	1	1	
			Dump Truck	6 t	-	6	6	
			Truck w/Crane	6 t	-	2	2	
			Motor Grader	11 t	-	1	1	
			Road Roller	8/10 t	-	1	1	
			Vibrating Roller	2.5 t	-	1	1	
				2.5 t	-	1	1	
			Truck Crane	30 t	-	2	2	
			Welder	200 A	-	1	1	
			Tamper	80 kg	-	2	2	
			Submergible Pump	6 inch	-	2	2	
			Submergible Pump	4 inch	-	2	2	
			Submergible Pump	3 inch	-	2	2	
			Diesel Generator	60 kVA	-	1	1	
			Concrete Mixer	0.5 m^3	-	2	2	
			Concrete Vibrator	45 mm	-	6	6	
			Belt Conveyor	7 m	-	10	10	
			Air Compressor	6 m^3	-	2	2	
8.	Gates	4 sets						
	(WG 1, 2, 3, & 4)		Bulldozer w/Ripper	21 t	-	1	1	
			Bulldozer	11 t	-	1	1	
			Back Hoe	0.35 m^3	-	1	1	
			Truck w/ Crane	6 t	-	2	2	
			Truck Crane	30 t	-	1	1	
			Truck Crane	20 t	-	1	1	
			Submergible Pump	4 inch	-	2	2	
			Submergible Pump	3 inch	-	2	2	
			Generator	175 kVA	-	2	2	
			Generator	15 kVA	-	2	2	
			Tamper	80 kg	-	2	2	
			Concrete Mixer	0.5 m^3	-	1	1	
			Concrete Mixer	0.2 m^3	-	1	1	
			Concrete Vibrator	45 mm	-	6	6	
			Belt Conveyor	7 m	-	10	10	
			Air Compressor	6 m^3	-	2	2	
			Welder	200 A	-	2	2	

Table 6.2 Equipment Use Schedule (3/3)

No.	Work Items	Q'ty	Kind of Equipment to be Used	Cap	Construction Period 82/83	83/84	84/85	Remarks
9.	Modification	L.S						
	(Bridges, canal,		Bulldozer w/Ripper	21 t	-	1	1	
	siphon & public road)		Bulldozer	11 t	-	1	1	
			Dozer Shovel	1.2 m^3	-	1	1	
			Dump Truck	6 t	-	6	6	
			Motor Grader	11 t	-	1	1	
			Road Roller	8/10 t	-	1	1	
			Vibrating Roller	2.5 t	-	1	1	
			Vibrating Roller	0.5 t	-	4	4	
			Back Hoe	0.6 m^3	-	1	1	
			Concrete Mixer	0.2 m^3	-	2	2	
			Concrete Vibrator	45 mm	-	4	4	
			Belt Conveyor	7 m	-	6	6	
			Air Compressor	6 m^3	-	2	2	
			Pick Hammer	1 m^3	-	3	3	
			Soil Compactor	90 kg	-	4	4	
			Tamper	80 kg	-	2	2	
			Diesel Pile Hammer	2.5 t	-	1	1	
			Truck w/Crane	6 t	-	1	1	
			Truck Crane	20 t	-	1	1	
			Welder	200 A	-	1	1	
10.	Spoil Bank	L.S						
			Bulldozer	21 t	-	-	10	
			Bulldozer	11 t	-	-	2	

Table 6.3 Calculation Sheet of Construction Equipment

1. Access Road

1.1 Land Levelling

Q'ty : 20 km x 7 m x 0.1 m = 14,000 m^3

Period : 148 days

Hourly Construction:

$$\frac{14,000\ m^3}{148\ days\ x\ 9\ hrs} = 13.5\ m^3/h$$

Kind of Equipment:

11 ton Bulldozer

$$\frac{13.5\ m^3}{33.0\ m^3} = 0.41$$ 1 unit

11 ton Motor Grader

$$\frac{13.5\ m^3}{153\ m^3} = 0.09$$ 1 unit

8/10 ton Road Roller

$$\frac{13.5\ m^3}{45\ m^3} = 0.3$$ 1 unit

1.2 Excavation of Earth Materials

Q'ty : 20 km x 7 m x 0.3 m = 42,000 m^3

42,000 m^3 ÷ 0.9 = 47,000 m^3

Period : 148 days

Hourly Construction:

$$\frac{47,000\ m^3}{148\ days\ x\ 7\ hrs} = 45\ m^3/h$$

Kind of Equipment:

21 ton Bulldozer W/Ripper

$$\frac{45\ m^3}{73\ m^3} = 0.62$$ 1 unit

1.2 m^3 Dozer Shovel

$$\frac{45\ m^3}{54\ m^3} = 0.83$$ 1 unit

6 ton Dump Truck (D = 2.0 km)

$$\frac{45 \text{ m}^3}{8.2 \text{ m}^3} = 5.5$$ <u>6 units</u>

1.3 Spreading of Earth Materials

Q'ty : 47,000 m^3

Period : 148 days

Hourly Construction:

$$\frac{47,000 \text{ m}^3}{148 \text{ days x } 7 \text{ hrs}} = 45 \text{ m}^3/\text{h}$$

Kind of Equipment:

11 ton Bulldozer

$$\frac{45 \text{ m}^3}{73 \text{ m}^3} = 0.62$$ <u>1 unit</u>

1.4 Compaction of Earth Materials

Q'ty : 47,000 m^3

Period : 148 days

Hourly Construction:

$$\frac{47,000 \text{ m}^3}{148 \text{ days x } 7 \text{ hrs}} = 45 \text{ m}^3/\text{h}$$

Kind of Equipment:

2.5 ton Vibrating Roller

$$\frac{45 \text{ m}^3}{28 \text{ m}^3} = 1.6$$ <u>2 units</u>

2. Embankment of Levee

2.1 Foundation Treatment of Levee

Q'ty : 42,000 m^3

Period : 318 days

Hourly Construction:

$$\frac{42{,}000\ m^3}{318\ \text{days} \times 7\ \text{hrs}} = 19\ m^3/h$$

Kind of Equipment:

11 ton Bulldozer

$$\frac{19\ m^3}{33\ m^3} = 0.57$$ <u>1 unit</u>

1.2 m^3 Dozer Shovel

$$\frac{19\ m^3}{54\ m^3} = 0.35$$ <u>1 unit</u>

6 ton Dump Truck <u>3 units</u>

D_1 = 2 km

$$\frac{19\ m^3 \times 7.7\%}{8.2\ m^3} = 0.18$$

D_2 = 3.6 km

$$\frac{19\ m^3 \times 5.0\%}{5.2\ m^3} = 0.18$$

D_3 = 0.6 km

$$\frac{19\ m^3 \times 26.2\%}{17.0\ m^3} = 0.29$$

D_4 = 7.7 km

$$\frac{19\ m^3 \times 5\%}{2.7\ m^3} = 0.35$$

D_5 = 2.0 km

$$\frac{19\ m^3 \times 56.1\%}{8.2\ m^3} = 1.30$$

11 ton Bulldozer (Stock Pile)

$$\frac{19\ m^3}{73\ m^3} = 0.26$$ <u>1 unit</u>

2.2 Stripping of Top Soil, Borrow Pit

Q'ty : 114,300 m^3

36,000 m^3 : Hauling to Stock Pile

78,300 m^3 : Backfilling of Farm Land

Period : 318 days

Hourly Construction:

$$\frac{36,000\ m^3}{318\ \text{days} \times 7\ \text{hrs}} = 16.2\ m^3/h$$

$$\frac{78,300\ m^3}{318\ \text{days} \times 7\ \text{hrs}} = 35.1\ m^3/h$$

Kind of Equipment:

11 ton Bulldozer

$$\frac{16.2\ m^3 + 35.1\ m^3}{33\ m^3} = 1.55 \qquad \underline{2\ \text{units}}$$

1.2 m^3 Dozer Shovel

$$\frac{16.2\ m^3}{54\ m^3} = 0.3 \qquad \underline{1\ \text{unit}}$$

6 ton Dump Truck (D = 0.6 km)

$$\frac{16.2\ m^3}{17.0\ m^3} = 0.95 \qquad \underline{1\ \text{unit}}$$

11 ton Bulldozer (Stock Pile & Farm Land)

$$\frac{16.2\ m^3 + 35.1\ m^3}{73\ m^3} = 0.7 \qquad \underline{1\ \text{unit}}$$

2.3 Excavation of Earth Materials

Q'ty : 1,308,000 m^3

Farm Land : 783,000 m^3 (60%)

Short Cut : 525,000 m^3 (40%)

Period : 318 days

Hourly Construction:

$$\frac{1,308,000\ m^3}{318\ \text{days} \times 7\ \text{hrs}} = 588\ m^3/h$$

Kind of Equipment:

21 ton Bulldozer w/Ripper — 8 units

Farm Land

$$\frac{588\ m^3 \times 60\%}{73\ m^3} = 4.83$$

Short Cut

$$\frac{588\ m^3 \times 40\%}{73\ m^3} = 3.22$$

1.2 m^3 Dozer Shovel — 11 units

Farm Land

$$\frac{588\ m^3 \times 60\%}{54\ m^3} = 6.53$$

Short Cut

$$\frac{588\ m^3 \times 40\%}{54\ m^3} = 4.36$$

6 ton Dump Truck — 69 units

D_1 = 2.0 km (Farm Land)

$$\frac{588\ m^3 \times 8\%}{8.2\ m^3} = 5.74$$

D_2 = 3.3 km (Short Cut)

$$\frac{588\ m^3 \times 4.7\%}{5.6\ m^3} = 4.94$$

D_3 = 0.3 km (Short Cut)

$$\frac{588\ m^3 \times 24.6\%}{22\ m^3} = 6.57$$

D_4 = 7.4 km (Short Cut)

$$\frac{588\ m^3 \times 4.7\%}{2.8\ m^3} = 9.87$$

D_5 = 2.0 km (Farm Land)

$$\frac{588\ m^3 \times 58\%}{8.2\ m^3} = 41.59$$

2.4 Spreading of Earth Materials

Q'ty : 1,308,000 m^3

Period : 318 days

Hourly Construction:

$$\frac{1,308,000 \ m^3}{318 \text{ days} \times 7 \text{ hrs}} = 588 \ m^3/h$$

Kind of Equipment:

11 ton Bulldozer

$$\frac{588 \ m^3}{73 \ m^3} = 8.05$$ 8 units

2.5 Compaction of Earth Materials

Q'ty : 1,308,000 m^3

Period : 318 days

Hourly Construction:

$$\frac{1,308,000 \ m^3}{318 \text{ days} \times 7 \text{ hrs}} = 588 \ m^3/h$$

Kind of Equipment:

5 m^3 Water Tanker 4 units

Sprinkling of Water

Materials from Short Cut:		6.66 t
Materials from Farm Land:		34.93 t
	Total :	41.59 t

Hourly capacity:

Cycletime 30 min

$$\frac{60 \times 5t}{30 \text{ min.}} = 10 \ t/h$$

Equipment to be Required:

$$\frac{41.59t}{10t} = 4.16$$

2.5 ton Vibrating Roller (75%)

$$\frac{588 \ m^3 \times 75\%}{28 \ m^3} = 15.75$$ 16 units

0.5 ton Vibrating Roller (15%)

$$\frac{588\ m^3 \times 15\%}{12\ m^3} = 7.35 \qquad \underline{8\ \text{units}}$$

90 kg Soil Compactor (10%)

$$\frac{588\ m^3 \times 10\%}{9.3\ m^3} = 6.33 \qquad \underline{7\ \text{units}}$$

3. Wet Masonry

3.1 Coffering & Unwatering

Q'ty : (H) 2 m x (W) 2 m x (L) 2,770 m = 11,080 m^3

Period : 60 days (30 days x 2 years)

Hourly Construction :

$$\frac{11,080\ m^3}{60\ \text{days} \times 7\ \text{hrs}} = 26\ m^3/h$$

Kind of Equipment :

11 ton Bulldozer

$$\frac{26\ m^3}{33\ m^3} = 0.79$$ <u>1 unit</u>

1.2 m^3 Dozer Shovel

$$\frac{26\ m^3}{54\ m^3} = 0.48$$ <u>1 unit</u>

∅ 6" Submergible Pump	<u>2 units</u>
∅ 4" "	<u>6 units</u>
∅ 3" "	<u>6 units</u>
60 kVA Diesel Generator	<u>2 units</u>

3.2 Foot Protection

Q'ty : 2,770 m

Period : 318 days

Daily Construction :

$$\frac{2,770\ m}{318\ \text{days}} = 8.7\ m\ /\text{day}$$

Kind of Equipment :

3 PS Engine Driven Winch <u>2 units</u>

Number of Wooden Pile to be Driven :

$$\frac{8.7\ m}{0.4\ m} = 22\ \text{pcs.}$$

Daily Capacity : 14 pcs/day

Equipment to be Required :

$$\frac{22\ \text{pcs}}{14\ \text{pcs}} = 1.57$$

3.3 Backfilling

Q'ty : $43,360\ m^2 \times 0.3\ m = 13,008\ m^3$

Period : 318 days

Daily Construction :

$$\frac{13,008\ m^3}{318\ days} = 40.9\ m^3/day$$

Kind of Equipment :

80 kg Tamper

$$\frac{40.9\ m^3}{44\ m^3} = 0.93$$ <u>1 unit</u>

4. Parapet Wall

Q'ty : $4,200\ m^3$

Period : 318 days

Hourly Construction :

$$\frac{4,200\ m^3}{318\ days \times 7\ hrs} = 1.9\ m^3/h$$

Kind of Equipment

$0.2\ m^3$	Concrete Mixer	(Left & Right Side)	<u>2 units</u>
$\phi45$	Concrete Vibrator	(")	<u>4 units</u>
*$1\ m^3$	Pick Hammer	(")	<u>6 units</u>
$6\ m^3$	Air Compressor	(")	<u>2 units</u>
7 m	Belt Conveyor	(")	<u>4 units</u>

* Pick Hammer will be used for the breaking of existing wall.

5. Groyne

Q'ty : 240 m

Concrete Pile 408 pcs.

Period : 30 days

Daily Construction :

$$\frac{408\ pcs}{30\ days} = 13.6\ pcs/day$$

Kind of Equipment :

2.5 ton Diesel Pile Hammer

$$\frac{13.6 \text{ pcs}}{14 \text{ pcs}} = 0.97$$ 1 unit

20 ton Hydraulic Crane	1 unit
6 ton Truck	1 unit

6. New Bridges

Bridges : Slajen, Patihan & Krandang

Period : 2 years

Kind of work :

(1)	Access to the River Bed	L.S
(2)	Coffer Dam & Unwatering	L.S
(3)	Excavation of Bridge Foundation	L.S
(4)	Driving of Steel Pipe Pile	L.S.
(5)	Treatment of Bridge Foundation	L.S
(6)	Placing of Boulder	L.S
(7)	Placing of Base & Pier Concrete	L.S.
(8)	Back filling	L.S
(9)	Erection of Superstructure	L.S
(10)	Placing of Floor Slub	L.S
(11)	Asphalt Paving	L.S
(12)	Approach Road	L.S
(13)	Installation of Rail & Accessories	L.S.
(14)	Removal of Existing Bridges	L.S

Kind of Equipment :

21 ton Bulldozer W/Ripper	1 unit
11 ton Bulldozer	1 unit
1.2 m^3 Dozer Shovel	1 unit
0.6 m^3 Back Hoe	1 unit
6 ton Dump Truck	6 units
6 ton Truck w/Crane	2 units
11 ton Motor Grader	1 unit
8/10 ton Road Roller	1 unit
2.5 ton Vibrating Roller	1 unit

2.5 ton	Diesel Pile Hammer	1 unit
30 ton	Truck Crane	2 units
200 A	Welder	1 unit
80 kg	Tamper	2 units
6 inch	Submergible Pump	2 units
4 inch	"	2 units
3 inch	"	2 units
60 kVA	Diesel Generator	1 unit
0.5 m^3	Concrete Mixer	2 units
45 mm	Concrete Vibrator	6 units
7 m	Belt Conveyor	10 units
6 m^3	Air Compressor	2 units

7. Gates

Gates : WG 1, 2, 3 & 4

Period : 2 years

Kind of Work :

(1)	Access to the River Bed	L.S
(2)	Coffer Dam & Unwatering	L.S
(3)	Excavation of Gate Foundation	L.S
(4)	Treatment of Gate Foundation	L.S
(5)	Placing of Boulder	L.S
(6)	Placing of Base & Inlet Structure Concrete	L.S
(7)	Backfilling	L.S
(8)	Erection of Gate	L.S

Kind of Equipment :

21 ton Bulldozer w/Ripper	1 unit
11 ton Bulldozer	1 unit
0.35 m^3 Back Hoe	1 unit
6 ton Truck w/Crane	2 units
30 ton Truck Crane	1 unit
20 ton "	1 unit
4 inch Submergible Pump	2 units
3 inch "	2 units
175 kVA Generator	2 units
15 kVA "	2 units

80 kg	Tamper	2 units
0.5 m^3	Concrete Mixer	1 unit
0.2 m^3	"	1 unit
45 mm	Concrete Vibrator	6 units
7 m	Belt Conveyor	10 units
6 m^3	Air Compressor	2 units
200 A	Welder	2 units

8. Modification

Modification : Bridges, Canal, Siphon & Public Road

Period : 2 years

Kind of Work :

Bridges

(1)	Access to the River Bed	L.S
(2)	Assembling of Staging	L.S
(3)	Driving of Concrete Pile	L.S
(4)	Treatment of Pier	L.S
(5)	Breaking of Floor Slub	L.S
(6)	Placing of Pier Concrete	L.S
(7)	Placing of Floor Slub	L.S
(8)	Asphalt Paving	L.S
(9)	Installation of Rail & Accessories	L.S
(19)	Approach Road	L.S

Canal

(1)	Stripping of Top Soil	L.S
(2)	Excavation of Canal	L.S
(3)	Embankment of Canal	L.S

Siphon

(1)	Excavation of Siphon Foundation	L.S
(2)	Treatment of Siphon Foundation	L.S
(3)	Placing of Boulder	L.S
(4)	Placing of Base, Inlet, Conduit & Outlet Concrete	L.S
(5)	Backfilling	L.S

Public Road

(1) Land Levelling L.S

(2) Compaction L.S

Kind of Equipment :

21 ton	Bulldozer w/Ripper	1 unit
11 ton	Bulldozer	1 unit
1.2 m^3	Dozer Shovel	1 unit
6 ton	Dump Truck	6 units
11 ton	Motor Grader	1 unit
8/10 ton	Road Roller	1 unit
2.5 ton	Vibrating Roller	1 unit
0.5 ton	Vibrating Roller	4 units
0.6 m^3	Back Hoe	1 unit
0.2 m^3	Concrete Mixer	2 units
45 mm	Concrete Vibrator	4 units
7 m	Belt Conveyer	6 units
6 m^3	Air Compressor	2 units
1.0 m^3	Pick Hammer	3 units
90 kg	Soil Compactor	4 units
80 kg	Tamper	2 units
2.5 ton	Diesel Pile Hammer	1 unit
6 ton	Truck w/Crane	1 unit
20 ton	Truck Crane	1 unit
200 A	Welder	1 unit

9. Treatment of Spoil Bank

Q'ty : 210,000 m^3

Period : 61 days

Hourly Construction :

$$\frac{210,000\ m^3}{61\ \text{days} \times 7\ \text{hrs}} = 492\ m^3/h$$

Kind of Equipment :

Excavation of Soil Materials

21 ton Bulldozer

$$\frac{492\ m^3}{73\ m^3} = 6.74$$ 7 units

Spreading of Soil Materials

21 ton Bulldozer

$$\frac{492\ m^3 \times 70\ \%}{119\ m^3} = 2.89 \qquad \underline{3\ \text{units}}$$

11 ton Bulldozer

$$\frac{492\ m^3 \times 30\ \%}{73\ m^3} = 2.02 \qquad \underline{2\ \text{units}}$$

Table 6.4 Hourly Production of Construction Equipment

1. Bulldozer

1.1 Excavation Work (Bank Measure)

$$Q = \frac{3,600 \times q \times E}{2.2D + 15} \quad (m^3/h)$$

Where,

Q: Hourly Production

q: Blade Capacity

11 t = 1.24 m^3
21 t = 2.74 m^3
Swamp 13 t - 1.32 m^3

E: Working Efficiency

Bulldozer	-	Normal	0.6
Swamp	-	Bad	0.4

D: Distance 30 m

Then,

11 ton Bulldozer	33 m^3/h
21 ton Bulldozer	73 m^3/h
13 ton Swamp Bulldozer	23 m^3/h

1.2 Compaction Work (Bank Measure)

$$Q = \frac{V \times W \times D \times E \times f}{N} \quad (m^3/h)$$

Where,

Q: Hourly Production

V: Compacting Speed

W: Compacting Width

Bulldozer	W (m)	V (m/h)
11 ton	0.6	3,500
21 ton	0.9	3,500

D: Compacting Depth 0.2 m

E: Working Efficiency 0.7

N: Number of Compaction 5

f: Swell Factor 1.1

Then,

11 ton Bulldozer	65 m^3/h
21 ton Bulldozer	97 m^3/h

1.3 Spreading Work (Bank Measure)

11 ton Bulldozer

$$Q = 10E\,(11D + 8)\,f \qquad (m^3/h)$$

21 ton Bulldozer

$$Q = 10E\,(18D + 13)\,f \qquad (m^3/h)$$

Where,

Q:	Hourly Production	
D:	Compacting Depth	0.2 m
E:	Working Efficiency	0.65
f:	Swell Factor	1.1

Then,

11 ton Bulldozer	<u>73 m^3/h</u>
21 ton Bulldozer	<u>119 m^3/h</u>

1.4 Spreading & Compacting Work (Bank Measure)

$$Q = \frac{Q_1 \times Q_2}{Q_1 + Q_2} \qquad (m^3/h)$$

Where,

Q :	Hourly Production
Q_1:	Hourly Compacting
Q_2:	Hourly Spreading

Then,

11 ton Bulldozer	<u>34 m^3/h</u>
21 ton Bulldozer	<u>53 m^3/h</u>

2. Back Hoe (Bank Measure)

$$Q = \frac{3,600 \times q \times E}{Cm} \quad (m^3/h)$$

Where,

Q : Hourly Production

q : Bucket Capacity

$0.35\ m^3$ class - $0.34\ m^3$
$0.6\ m^3$ class - $0.59\ m^3$

E : Working Efficiency

Normal 0.7

Cm: Cycletime

$0.35\ m^3$ (135°) - 31 sec
$0.6\ m^3$ (135°) - 33 sec

Then,

$0.35\ m^3$ Back Hoe	$28\ m^3/h$
$0.6\ m^3$ Back Hoe	$45\ m^3/h$

3. Dozer Shovel (Bank Measure)

$$Q = \frac{3,600 \times q \times K \times f \times E}{Cm} \quad (m^3/h)$$

Where,

Q:	Hourly Production	
q:	Bucket Capacity	Heap - $1.4\ m^3$
K:	Coefficient of Bucket	Normal - 0.85
f:	Swell Factor	0.77
E:	Working Efficiency	0.7

Cm: Cycletime

$Cm = m \cdot \ell + t_1 + t_2$

m: Coefficient of under Carriage
Crawler Type - 1.8 (sec/m)
ℓ: Hauling Distance Normal - 8 m
t_1: Loading Time Crawler Type - 8 sec
t_2: Gear Change, Setting & Waiting
V Type Loading - 20 sec

$Cm = 42.4$ sec

Then,

$1.2\ m^3$ Dozer Shovel	$54\ m^3/h$

4. Dump Truck (Bank Measure)

$$Q = \frac{60 \times C \times f \times Et}{Cmt} \quad (m^3/h)$$

Where,

Q : Hourly Production

C : Vessel Capacity

$$C = \frac{T}{\gamma t} \times L$$

T: Loading Weight 6 t

γt: Unit Weight 1.8 t/m^3

L: Swell Factor 1.3

$$C = \frac{6}{1.8} \times 1.3 = 4.3 \ m^3$$

f : Swell Factor 0.77

Et: Working Efficiency

Normal - 0.9

Cmt: Cycletime

$$Cmt = \frac{Cms \times n}{60 \times Es} + T_1 + T_2 + t_1 + t_2$$

Cms: Cycletime of Loader 42.4 sec

n: Number of Loading

$$n = \frac{C}{q \cdot k}$$

q: Bucket Capacity 1.4 m^3

k: Coefficient of Bucket 0.85

$$n = \frac{4.3}{1.4 \times 0.85} \doteqdot 4$$

T_1: Hauling

T_2: Returning

$$T_1 = T_2 = \frac{D}{v}$$

Distance (km)	$T_1 = T_2$ (min)
$D_1 = 0.3$	$\frac{D \times 60}{15} = 1.2$
$D_2 = 0.6$	" $= 2.4$
$D_3 = 2.0$	" $= 8.0$
$D_4 = 3.3$	" $= 13.2$
$D_5 = 3.6$	" $= 14.4$
$D_6 = 7.4$	" $= 29.6$
$D_7 = 7.7$	" $= 30.8$
$D_8 = 15.0$	$\frac{D \times 60}{25} = 36.0$

t_1: Unloading 1.0 min

t_2: Waiting & Others 0.7 min

Then,

6 ton Dump Truck

Distance (km)	Cycletime (min)	Hourly Production (m^3/h)
$D_1 = 0.3$	8.14	22.0
$D_2 = 0.6$	10.54	17.0
$D_3 = 2.0$	21.74	8.2
$D_4 = 3.3$	32.14	5.6
$D_5 = 3.6$	34.54	5.2
$D_6 = 7.4$	64.94	2.8
$D_7 = 7.7$	67.34	2.7
$D_8 = 15.0$	77.74	2.3

5. Ripper Work (Bank Measure)

$$Q = \frac{60 \times A \times \ell \times f \times E}{Cm} \quad (m^3/h)$$

Where,

Q : Hourly Production

A : Area of Ripper

21 ton Bulldozer - Three Tooth 0.3 m^2

ℓ : Distance 20 m

f : Swell Factor 0.77

E : Working Efficiency 0.65

Cm : Cycletime

$Cm = 0.05\ \ell + 0.33$

$= 1.23$ min

Then,

21 ton Bulldozer w/Ripper <u>146 m^3/h</u>

6. Motor Grader (Bank Measure)

$$Q = \frac{60 \times W \times L \times H \times f \times E}{P \times Cm} \quad (m^3/h)$$

Where,

Q : Hourly Production

W : Width of Blade

$W = \ell \cdot \mathrm{Sin}\ 60^{\circ} - 0.3$

ℓ : Length of Blade 3.7 m

$W = 3.7 \times \mathrm{Sin}\ 60^{\circ} - 0.3$

$= 2.9$ m

L : Length of Grading 200 m

H : Depth of Layer 0.3 m

f : Swell Factor 0.77

E : Working Efficiency 0.4

P : Number of Grading 3

Cm : Cycletime

$$Cm = \frac{L}{V_1} + \frac{L}{V_2} + 2t$$

V_1 : Working Speed 66.7 m/min

V_2 : Reverse Speed 100 m/min

t : Gear Change & Others 1.0 min

Cm = 7.0 min

Then,

11 ton Motor Grader <u>153 m^3/h</u>

7. Compacting Equipment (Bank Measure)

7.1 Road Roller

$$Q = \frac{W \times V \times H \times f \times E}{P} \quad (m^3/h)$$

Where,

Q : Hourly Production

W : Width of Compaction

W = Width of Roller - 0.3

= 1.5 - 0.3

= 1.2 m

V : Working Speed 1,500 m/hr

H : Depth of Layer 0.3 m

f : Swell Factor 0.77

E : Working Efficiency 0.6

P : Number of Compaction 6

Then,

8/10 ton Road Roller <u>45 m^3/h</u>

7.2 Vibrating Roller

$$Q = \frac{W \times V \times H \times f \times E}{P} \quad (m^3/h)$$

Where,

Q : Hourly Production

W : Width of Compaction

2.5 ton

$1.0 - 0.2 = 0.8$ m

0.5 ton

$0.6 - 0.2 = 0.4$ m

V : Working Speed

2.5 ton - 1,500 m/h

0.5 ton - 1,300 m/h

H : Depth of Layer 0.3 m

f : Swell Factor 0.77

E : Working Efficiency 0.6

P : Number of Compaction 6

Then,

2.5 ton Vibrating Roller	28 m^3/h
0.5 ton Vibrating Roller	12 m^3/h

7.3 Tamper & Soil Compactor

$$Q = \frac{W \times V \times H \times f \times E}{P} \quad (m^3/h)$$

Where,

Q : Hourly Production

W : Width of Compaction

Tamper - 0.3 m

Soil Compactor - 0.4 m

V : Working Speed

Tamper - 450 m/h

Soil Compactor - 1,000 m/h

H : Depth of Layer 0.3 m

f : Swell Factor 0.77

E : Working Efficiency 0.6

P : Number of Compaction

Tamper - 3

Soil Compactor - 6

Then,

80 kg Tamper $\underline{6\ m^3/h}$

90 kg Soil Compactor $\underline{9\ m^3/h}$

8. Concrete Mixer

$$Q = \frac{60 \times q \times E}{Cm} \quad (m^3/h)$$

Where,

Q : Hourly Production

q : Mixing Capacity

$0.2\ m^3$

$0.5\ m^3$

E : Working Efficiency 0.4

Cm : Cycletime 4 min

Then,

$0.2\ m^3$ Concrete Mixer $\underline{1.2\ m^3/h}$

$0.5\ m^3$ Concrete Mixer $\underline{3\ m^3/h}$

No	DESCRIPTION	Q'ty	REMARKS
1	Detail Design & Preparation of Tender Documents	L. S.	
2	Preparatory Works		
	2.1 Access road	20 km	
	2.2 Site office	400 m^2	
	2.3 Repair shop & motor pool	1.800 m^2	
	2.4 Store house	400 m^2	
	2.5 Temporary Store house	2.000 m^2	
3	Survey & Investigations	L. S.	
4	Compensation		
	4.1 Land to be purcahased	89 ha	
	4.2 Land to be hired	100 ha	
	4.3 Houses	454 p.c	
5	Main Works		
	5.1 Embankment of levee	1.308.000 m^3	
	5.2 Short cut	525.000 m^3	
	5.3 Revetment		
	5.3.1 Wet masonry	41.000 m^2	
	5.3.2 Parapet wall	4.200 m^3	
	5.3.3 Groyne	240 m	
6	New Bridges & Gates		
	6.1 Bridges	3 sets	
	6.2 Gates	4 sets	
7	Modification Works		
	7.1 Bridges	2 sets	
	7.2 Irrigation canal	L.S	
	7.3 Public road	L.S	
8	Treatment of Spoil Bank	210.000 m^3	

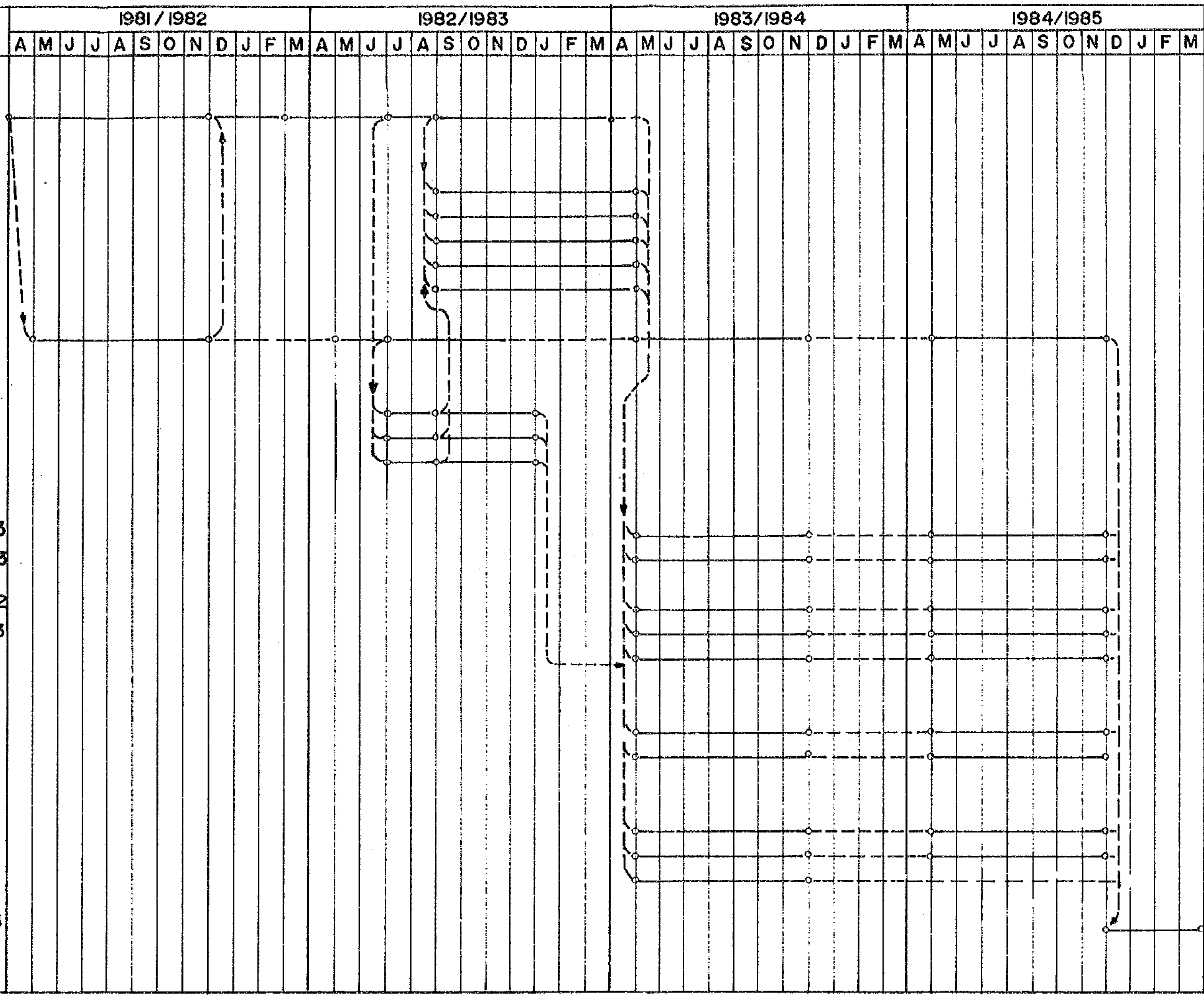


Fig. 6.1 CONSTRUCTION TIME SCHEDULE
(Alternative II — 2)